Soil Dynamics and Earthquake Engineering 106 (2018) 231–242

Contents lists available at ScienceDirect

Soil Dynamics and Earthquake Engineering

journal homepage: www.elsevier.com/locate/soildyn

Effects of compaction on the seismic performance of embankments built with gravel

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ARTICLE INFO

Keywords:
Gravel
Embankment
Compaction
Seismic response
Collapse

ABSTRACT

The effects of compaction on the seismic performance of gravelly embankments is investigated transferring the precision of very accurate laboratory tests on the constituent material into the prediction of the large scale response. The stress-strain response observed upon monotonic and cyclic tests is captured by a critical state elastic-plastic model used as a virtual simulator of experiments to determine the influence of the initial state on the soil response. This dependency is introduced in equivalent linear elastic-perfectly plastic models as function of the state variable ψ to study the propagation of shear waves through horizontally layered landfills and embankments. A parametric analysis is performed assigning systematically variable seismic inputs, soil density and geometry to see the role of these factors on the soil response and the sliding resistance of the embankment. The study reveals an exceptionally high resistance of the gravel that enables embankment with steep abutments to resist against severe earthquakes.

1. Introduction

Thanks to their excellent mechanical properties, gravely soils are used from ancient ages [1] in different civil engineering applications. Earth and rockfill dams, highway and railway embankments, landfills in dry or marine environment are just a few examples of the large variety of possible applications. From the beginning, noticeable importance was placed on compaction, as a significant role of the density on the mechanical response of materials was empirically felt. However only in the first half of nineteenth century a rational methodology, based on systematic experimental investigations carried out on site and in the laboratory, was introduced to quantify and control the effects of compaction in the construction of large dams [2]. This scientific approach led on one side to improve the performance of earth structures, on the other side to develop more powerful compaction machineries [3]. Nowadays a greater part of the energy and cost supported in embankment construction is spent to place the material with roller compactors, and cost effectiveness relies on the optimization of thickness of strata and number of roller passes. However, in spite of these technical progresses, the constitutive models implemented for gravels are rarely able to simulate the effects of compaction with sufficient accuracy, as they do not possess the same level of generality used for other soil types (e.g., Strahler et al. [4]). The adopted schemes typically reproduce specific testing conditions, without attempting to quantify the mechanical characteristics into a general and consistent framework. The surprising difference with the level of sophistication adopted for finer soils stems from a large confidence placed on the capacity of gravely materials to resist heavy loading conditions, but also on the experimental difficulties to manage uncommonly big laboratory equipment. More complete analytical tools, capable of simulating the influence of compaction on the static and dynamic response of gravels may on the contrary lead to a more cost-effective design of the placement.

The first extensive studies on rockfill were carried out by Marsal [5] who performed triaxial tests on samples of coarse grained materials (size up to 20 cm) having diameters equal to 113 cm. These tests were conducted at very high confining stresses (up to 2500 kPa) willing to simulate the response of soil used for the construction of tall dams. The observed decrease with confining stress of the angle of internal friction ($\phi = arcsin[(\sigma_1 - \sigma_3)/(\sigma_1 + \sigma_3)_{max}]$) was interpreted as an effect of particle breakage [2,6]. Subsequently, Charles and Watts [7] and Houlsby [8] focused on the curvature of the Mohr failure envelope at low confining stresses, i.e. where particle breakage is unlikely, concluding that many rockfill embankments could be built with slopes steeper than currently given without significantly losing their margin of safety against instability.

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https://doi.org/10.1016/j.soildyn.2018.01.005
Received 19 September 2016; Received in revised form 16 August 2017; Accepted 1 January 2018

Later on the response of gravels under cyclic loading was extensively observed by Seed et al. [9] who performed a series of cyclic tests carried out with variable strain amplitudes on materials compacted at different relative densities and recorded the variation of secant shear modulus and damping ratio. In this way, the authors inferred a dependency of the parameters of an equivalent viscous-elastic model for dynamic analyses on the soil density and on the initial stress. Similar studies were conducted by Rollins et al. [10] and more recently by Aghaei Araei et al. [11].

With particular reference to the small strain response (less than $10^{-3\%}$), Park and Tatsuoka [12] observed a dependency of normal stiffness on the relative orientation between loading direction and plane of soil deposition. A further factor of anisotropy was acknowledged from the dependency of the stiffness moduli on the different stress components. Jang et al. [13], Modoni et al. [14], and Flora et al. [15] performed small strain cyclic triaxial and pulse wave transmission tests on a gravel compacted at different initial density. All these authors agree in quantifying the role of void ratio on the small strain stiffness with the function defined for sands by Hardin and Richart [16] and subsequently used herein (see Eq. 3.g).

One of the very few attempts to simulate the response of gravel at larger strain levels was conducted by Balakrishnaiyer and Koseki [17], who interpreted the results of their large amplitude cyclic tests with an elastic-plastic model previously defined for sands by Masuda et al. [18]. A drag and scaling rule was introduced to relate the unloading and reloading stress strain curves to the primary loading backbone curve. However, the dependency of soil response on the density is not taken into account. Later Modoni et al. [19] formulated a critical state multiple hardening elastic-plastic model to simulate the stress strain response of a gravel from small strain to failure. This model, valid for monotonic and cyclic loading paths, consists of an adaptation to gravels of models previously introduced for sands [20,21], whose main concept is the combined dependency of soil response on the current void ratio (e) and mean effective stress (p′) via the state variable ψ [22], expressing the current distance from the critical state line in the e-p′ plane. In particular, Modoni et al. [19] simulated the cyclic loading with a hierarchy of yield surfaces, each having its hardening functions and flow rule.

The approach herein followed to study the propagation of seismic waves through embankments of various shapes is similar to that originally suggested by Idriss and Seed [23]. In the performed analysis, the soil response is simulated with an equivalent linear elastic-perfectly plastic model, while the characteristic curves (G-γ and D-γ) and strength (friction and dilatancy angles) are obtained from a repetitive application of the previously recalled model from Modoni et al. [19]. The latter has been validated with experimental results on artificially reconstituted samples of a material made of crushed sandstone (Chiba gravel). In this practical way, the influence of soil density is taken into account without introducing the formal difficulty given by the implementation of the more complex model in the numerical codes.

2. The elastic - plastic model

The model [19], briefly described in the following, assumes the gravel equivalent to a continuum and computes deformation as composed of an elastic fraction and a plastic fraction:

$$\delta\varepsilon = \delta\varepsilon^{e} + \delta\varepsilon^{p} \tag{1}$$

The elastic response is modelled borrowing the cross-anisotropic model of Tatsuoka and Kohata [24], that adopts the following relations, where the vertical axis z coincides with the direction of soil deposition:

$$\begin{bmatrix}\varepsilon_x\\ \varepsilon_y\\ \varepsilon_z\\ \gamma_{xy}\\ \gamma_{yz}\\ \gamma_{zx}\end{bmatrix} = \begin{bmatrix} \frac{1}{E_h} & \frac{-\nu_{hh}}{E_h} & \frac{-\nu_{vh}}{E_v} & 0 & 0 & 0\\ \frac{-\nu_{hh}}{E_h} & \frac{1}{E_h} & \frac{-\nu_{vh}}{E_v} & 0 & 0 & 0\\ \frac{-\nu_{vh}}{E_v} & \frac{-\nu_{vh}}{E_v} & \frac{1}{E_v} & 0 & 0 & 0\\ 0 & 0 & 0 & \frac{(1+\nu_{hh})}{E_h} & 0 & 0\\ 0 & 0 & 0 & 0 & \frac{1}{2G_{vh}} & 0\\ 0 & 0 & 0 & 0 & 0 & \frac{1}{2G_{vh}} \end{bmatrix} \cdot \begin{bmatrix}\Delta\sigma'_x\\ \Delta\sigma'_y\\ \Delta\sigma'_z\\ \Delta\tau_{xy}\\ \Delta\tau_{yz}\\ \Delta\tau_{zx}\end{bmatrix} \tag{2}$$

The parameters in the above matrix are expressed as function of the stress components and soil void ratio by the following relations:

$$E_v = E_1 \cdot f(e) \cdot \sigma'_v{}^{m} \cdot p_r{}^{1-m} \tag{3.a}$$

$$E_h = E_1 \cdot (1-I_0) \cdot f(e) \cdot \sigma'_h{}^{m} \cdot p_r{}^{1-m} \tag{3.b}$$

$$\nu_{hh} = \nu_0 \tag{3.c}$$

$$\nu_{vh} = \nu_0 \cdot (1-I_0)^{-0.5} \cdot \left(\frac{\sigma'_v}{\sigma'_h}\right)^n \tag{3.d}$$

$$\nu_{hv} = \nu_0 \cdot (1-I_0)^{0.5} \cdot \left(\frac{\sigma'_h}{\sigma'_v}\right)^n \tag{3.e}$$

$$G_{vh} = \frac{E_1 \cdot (2 - I_0) \cdot f(e) \cdot (\sigma_v + \sigma_h)^m}{4 \cdot (1 + \nu_0)} \tag{3.f}$$

$$f(e) = \frac{(2.17-e)^2}{(1+e)} \tag{3.g}$$

where σ'_v and σ'_h are the vertical and horizontal effective stresses, respectively, p_r is a reference pressure (1 kPa), f(e) is the function of void ratio defined by Hardin and Richart [16], and I_o is a parameter defining inherent anisotropy. The parameters I_o, E_1, ν_o, m and n are soil parameters, fitted with experimental results.

The plastic strains are assumed to be induced only by deviator stress variation, as the experimental observations [25,26] exhibit an almost fully recoverable response when the mean stress is increased. Plastic strain are then calculated with a model based on the critical state, whose locus in the void-ratio – stress invariants space is defined by the following relations:

$$\frac{q}{p'} = M \tag{4.a}$$

$$e_{cs} = \Gamma - \lambda \cdot \ln p' \tag{4.b}$$

where p′ and q are the mean effective and deviator stress, Γ and λ define the projection of the critical state locus on the volumetric plane and M is the critical stress ratio. In particular, the stress-strain relation depends on the distance in the e-p′ plane from the critical state locus, expressed by the state variable ψ [22]:

$$\psi = e - e_{cs} \tag{5}$$

The dependency of the hardening functions on ψ for primary loading is defined by Eq. (6):

$$\frac{S}{S_{u.b.}} = I \frac{\varepsilon_q^p \cdot |\varepsilon_q^p|^{c-1}}{B + |\varepsilon_q^p|^c} \tag{6}$$

where ε_q^p represents the plastic distortional strain, S is a function of the stress invariants ratio ($\eta = q/p'$):

$$S = \frac{3\eta}{6+\eta} \tag{7}$$

$S_{u.b.}$ is an upper bound for S and is currently expressed as a function of the critical state friction angle ϕ'_{cs} and of the state variable ψ:

$$S_{u.b.} = \sin\phi'_{cs} \cdot (1 - k\psi) \tag{8}$$

B, c, k and I are soil parameters calibrated with experimental