Computers and Geotechnics 57 (2014) 53–64

Contents lists available at ScienceDirect

Computers and Geotechnics

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

How the response spectrum of non-liquefied loose-to-medium sand deposits is affected by the groundwater level

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ARTICLE INFO

Article history:
Received 6 June 2013
Received in revised form 25 November 2013
Accepted 11 January 2014
Available online 28 January 2014

Keywords:
Seismic site effects
Ground water level
Elastoplastic soil behaviour

ABSTRACT

This study examines the water level influence on the seismic response of sand deposits. The seismic ground response of a sand deposit for two cases was computed: groundwater level near the surface and at great depth. The ratio of soil factors computed for both cases evaluates the influence of having the water table near the surface. This parameter is small when the full range of periods is considered, but rises to ~1.15 when narrowing the range of periods to 0.2–0.6 s, corresponding to the plateau of Eurocode 8 ground type C response spectrum. So, the water level effect is of the same magnitude of the existing soil factor in seismic codes.

1. Introduction

Earthquakes of the last decades have clearly shown the influence of local site conditions on the characteristics of the seismic ground motion. Site response analysis has become a very important issue in earthquake engineering, and it is usually the first step for the analysis of soil-structure interaction problems.

In the last decades a significant effort has been devoted to understanding and predicting the response of soil deposits to strong earthquake motion.

The so-called dynamic properties of soils mainly depend on soil density, effective confining pressures, soil plasticity and strain amplitude. In particular, for sands the influence of the mean effective stress can be quite important at low effective confining pressure conditions (e.g. [1,2]).

In a sand deposit, the effective stress is strongly affected by the hydrostatic pressure, related to the presence of a water level (WL), and by the excess pore pressures that result from seismically induced plastic deformations in the sand skeleton corresponding to undrained or poorly drained conditions. In addition, in this soil type it is expected that the ground water level naturally fluctuates.

Groundwater may play an important role in seismic analysis. The decrease in the shear strength and/or stiffness caused by the increase in pore water pressures in saturated cohesionless materials during earthquake ground motion, may give rise to significant permanent deformations or even to a condition of near-zero effective stress in the soil [3]. Modern seismic codes (e.g. Eurocode 8 [3] and the US code ICC [4]) require a separate analysis for potential liquefaction hazard and define a special ground type for deposits of liquefiable soils. Non-liquefiable soils are included in the other ground types.

This paper is focused in examining the influence of the water level on the seismic response of non-liquefied sand deposits.

The effect introduced by the water level close to the surface it is not usually taken into account when strong-motion records are associated to a given site characteristics. This occurs because (i) generally the ground water level during past seismic event was not determined (e.g. [5,6]), and (ii) it is not possible to isolate the water level effect for a given recorded ground motion, as different seismic events recorded at the same site where the water level fluctuates significantly have different seismogenetic characteristics and, thus, are not directly comparable.

Hence, the assessment of the influence of water level by numerical simulation arises as the main alternative.

The purpose of this work is to quantify the influence of the water level on the seismic response for two extreme cases (WL near the surface vs. WL at great depth) and to call attention to its effects, encouraging discussion. These results provide insight into the relative importance of having information on ground water level when predicting ground motions.

The quantification is done computing amplification factors as the ones computed for Eurocode 8 [3]. Amplification factors represent the ratio of a ground motion intensity measure for a specified site condition to the value of the parameter that would have been expected for a reference site condition. Reference site conditions are typically rock or weathered rock.

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http://dx.doi.org/10.1016/j.compgeo.2014.01.006

In this work, the influence of the water level position on the seismic response of a sand deposit (thicknesses modelled: 15.5 m and 30 m) overlying an elastic half-space is examined. Both models were used to conduct analyses where:

(1) WL great depth – the water level is at great depth, the sand is admitted to be unsaturated and the response is fully drained; for this case a pure mechanical analysis is performed.
(2) WL near surface – the water level is near the ground surface (2 m deep): below the water level, the sand is admitted to be fully saturated and the response is partially drained; for this case a coupled two-phase formulation is used; above the water level, the sand is admitted to be unsaturated and the response is drained; the suction's effect is not considered and a pure mechanical analysis is performed.

Because an advanced constitutive model is required to simulate the effects of the water level on the seismic response of a sand deposit, the advanced multi-mechanism elastoplastic model developed at École Centrale de Paris, ECP, [7,8] was used. This model takes into account important factors that affect soil behaviour, such as the strain level and the stress conditions, while the influence of other factors which control the stiffness degradation, such as the plasticity index and the initial state (OCR, void ratio, stress state, etc.) are considered via the model parameters. In this work the behaviour of Toyoura sand was simulated.

The ECP's model has been widely used to simulate soil behaviour under static and cyclic loading (e.g. [9,10]) and is implemented in the general purpose finite element code Gefdyn [11]. This code was choosen because is particularly suitable for modelling the cyclic behaviour of soils and soil-structure interaction, and it has been successfully used in the past to study the behaviour of geotechnical structures performing pure mechanical single-phase analysis and/or coupled two-phase analysis (e.g. [12–14]).

A set of 45 horizontal seismic records compatible with seismic action type 1 of Eurocode 8 [3] was used. A total of 180 dynamic analyses were performed in time domain. The ground response of the tested models is analyzed.

The response spectra of all four cases are analyzed and compared. Amplification factors computed as the ones defined by Eurocode 8 [3] are presented and discussed. It is shown that the presence of the water level near the surface is of the same magnitude of the exiting soil factors in seismic codes.

2. Numerical model

2.1. Constitutive model for soil

2.1.1. Model description

The soil behaviour is simulated over a large range of strains with ECP's elastoplastic multi-mechanism model developed by Aubry et al. [7] and extended to cyclic behaviour by Hujeux [8].

The model is written in terms of effective stress and the representation of all irreversible phenomena is made by four coupled elementary plastic mechanisms: three plane-strain deviator plastic deformation mechanisms in three orthogonal planes and an isotropic one.

The elastic response is assumed to be isotropic and non-linear where the bulk, K, and shear moduli, G, are functions of the mean effective stress according to the relations:

$$K = K_{ref}\left(\frac{p'}{p'_{ref}}\right)^{n_e}; \quad G = G_{ref}\left(\frac{p'}{p'_{ref}}\right)^{n_e} \tag{1}$$

where K_{ref} and G_{ref} are respectively the bulk and shear modulus at the reference mean effective stress p'_{ref} and p' the mean effective stress. The degree of the non-linearity is controlled by the exponent n_e.

During monotonic loading, the primary yield function associated to the generic mechanism k has the expression:

$$f_k(q_k, p'_k, \varepsilon_v^p, r_k) = q_k - p'_k \cdot \sin\phi'_{pp} \cdot \left(1 - b \cdot \ln\left(\frac{p'_k}{p'_c}\right)\right) \cdot r_k \tag{2}$$

where the following variables have been introduced:

- $q_k = \sqrt{\left(\frac{\sigma'_{ii}-\sigma'_{jj}}{2}\right)^2 + (\sigma'_{ij})^2}$ is the radius of the Mohr circle in the plane of the generic deviatoric mechanism of normal $\vec{e}_k$. Here i, j, k {1,2,3}; $i = 1 + \mathrm{mod}(k,3)$, and $j = 1 + \mathrm{mod}(k+1,3)$, with $\mathrm{mod}(k,i)$ representing the residue of the division of k by i;
- $p'_k = \frac{\sigma'_{ii}+\sigma'_{jj}}{2}$ is the centre of the Mohr circle in the plane of the deviatoric mechanism of normal $\vec{e}_k$;
- p'_c is the critical pressure that is linked to the volumetric plastic strain ε_v^p by the relation $p'_c = p'_{co} exp(\beta\varepsilon_v^p)$, where p'_{co} represents the initial critical pressure, which is the critical mean effective stress that corresponds to the initial state defined by the initial void ratio, and β the plasticity compression modulus of the material in the isotropic plane ($\ln p'$, ε_v^p);
- ϕ'_{pp} is the friction angle at the critical state;
- b is a numerical parameter which controls the shape of the yield surface in the (p'_k, q_k) plane and varies from $b = 0$ to 1, passing from a Coulomb surface to Cam-Clay surface type;
- r_k is an internal variable that defines the degree of mobilized friction of the mechanism k and introduces the effect of shear hardening of the soil.

This last variable is linked to the plastic deviatoric strain, $\varepsilon^p_{d,k}$, according to the following hyperbolic function:

$$r_k = r^{el} + \frac{\int d\varepsilon^p_{d,k} dt}{a + \int d\varepsilon^p_{d,k} dt} \tag{3}$$

where a is a parameter which regulates the deviatoric hardening of the material. It varies between a_1 and a_2, such that:

$$a = a_1 + (a_2 - a_1)\alpha_k(r_k) \tag{4}$$

where the intermediate of the parameter $\alpha_k(r_k)$, integrates the decomposition of the behaviour domain into pseudo-elastic, hysteretic and mobilized domains, where:

$$\begin{aligned} &\alpha_k(r_k) = 0 && \text{if} \quad r^{el} < r_k < r^{hys} \\ &\alpha_k(r_k) = \left(\frac{r_k - r^{hys}}{r^{mob} - r^{hys}}\right)^m && \text{if} \quad r^{hys} < r_k < r^{mob} \\ &\alpha_k(r_k) = 1 && \text{if} \quad r^{mob} < r_k < 1 \end{aligned} \tag{5}$$

r^{el} defines the extent of the elastic domain and is the minimum value that r_k can take, while r^{hys} and r^{mob} designate the extent of the domain where hysteresis degradation occurs. In this last domain, the evolution of $\alpha_k(r_k)$ is controlled by the exponent m.

The evolution of the volumetric plastic strains $\varepsilon^p_{v,k}$ is controlled by a flow rule based on Roscoe-type dilatancy rule:

$$\partial\varepsilon^p_{v,k} = \partial\varepsilon^p_{d,k}\alpha_k(r_k) \cdot \left(\sin\psi - \frac{q_k}{p'_k}\right) \cdot \alpha_\psi \tag{6}$$

with α_ψ a constant parameter and ψ representing the characteristic angle defining the limit between dilating ($\partial\varepsilon^p_{v,k} < 0$) and contracting ($\partial\varepsilon^p_{v,k} > 0$) of the soil.

Whenever a stress reversal occurs, the primary yield function (2) is abandoned and the cyclic surface becomes active. A kinematic hardening based on the state variables at the last load reversal is used. Refer to [7,8,14] among others for further details about the ECP's model.