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**On the soil response of a coastal sandy slope subjected to
tsunami-like solitary wave**

Jian Leng^a, Guanlin Ye^a, Chencong Liao^{a,*}, Dongsheng Jeng^{a,b}

a Collaborative Innovation Center for Advanced Ship and Deep-Sea Exploration,
State Key Laboratory of Ocean Engineering, Department of Civil Engineering,
Shanghai Jiao Tong University, Shanghai, 200240, China

b Griffith School of Engineering, Griffith University Gold Coast Campus, Queensland,
QLD 4222, Australia

**Correspondence information: Chencong Liao, Shanghai Jiao Tong University,
Shanghai, 200240, China; billaday@sjtu.edu.cn**

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1 **Abstract**

2 This study proposes a two-dimensional coupled approach to examine dynamic
3 response of a sloping beach due to tsunami-like solitary wave. Wave motion is
4 governed by volume-averaged Reynolds-averaged Navier–Stokes (VARANS)
5 equations, while the beach response is described with the poro-elastoplastic theory.
6 The wave module and beach module are strongly integrated, resulting in a profound
7 investigation of the solitary wave-induced soil response. Validation against the
8 experimental demonstrates the applicability of the present approach. Results show
9 that the excess pore water pressure ratio (EPWPR) is significant in the shallow soil.
10 Distribution of EPWPR along the soil depth direction shows a decreasing trend. In
11 addition, the principal axes of soil element on the shoreline rotated considerably under
12 the solitary wave loading. When wave draws down from the slope, both shear stress
13 and mean effective stress decrease compared with the run-up process. For engineering
14 practice, special attention is given to the effect of permeability and coast slope on the
15 soil response subjected to tsunami-like solitary waves.

16

17 **Keywords:** Solitary wave; Sandy slope; Porous media; Pore pressure; Effective stress

1. Introduction

Tsunami with huge amount of energy and fast traveling velocity brings serious damages to coastlines and coastal infrastructures in form of slope erosion, bed failure and sediment liquefaction. Although there exist several investigations of tsunami wave generation and propagation modelling, few researches take into account of the influence of beach response around coast slopes. During wave shoaling, breaking and run-up processes, sloping beach undergoes the build-up of excess pore pressure in the area where tsunami could reach. Once the sediment liquefied, it behaves like a heavy fluid that can be taken away by currents, resulting in collapse of structures built on it. Hence, it is of great significance for coastal and geotechnical engineering to estimate the coast slope response under tsunami waves involved in the design of marine structures.

Numerous studies of fluid-soil interactions have been conducted since the 1970s, involving the assessment of pore pressures, effective stresses, and displacements (Yamamoto et al. 1978; Madsen 1978; Okusa 1985; Hsu and Jeng 1994). The analytical solutions for response of an isotropic poro-elastic seabed, including inertial effects of the soil particle or pore fluid, also have been developed by many researchers (Jeng and Rahman 2000; Ulker and Rahman 2009). In recent years, more and more attentions are paid to the fluid-soil interaction around marine structures using numerical approximations (Zhang et al. 2013; Jeng et al. 2013; Ye et al. 2016). However, all these investigations have been limited to the oscillatory response and momentary liquefaction of the seabed under the framework of elastic theory. Besides the oscillatory mechanism, laboratory experiments (Tzang 1998; Sumer et al. 2006; Sumer et al. 2012) reported the importance of wave-induced pore pressure build-up mechanism and the existence of residual liquefaction. Seed and Rahman (1978)

1 developed a simple one-dimensional finite element model for the earthquake-induced
2 liquefaction to describe the buildup of pore pressure under progressive wave. This
3 model has been further extended to examine the wave-induced residual soil response
4 analytically and numerically (McDougal et al. 1989; Cheng et al. 2001; Sumer and
5 Fredsøe 2002). The build-up of pore pressure under progressive and standing waves
6 were observed in the wave flume tests and centrifuge tests by Kirca et al. (2013) and
7 Sassa and Sekiguchi (1999). In most of these experimental studies, only the pore
8 pressures were measured by electronic transducers, while the other factors, such as
9 displacements, effective stress and stress-strain relationship cannot be captured due to
10 the limitation in the monitoring technology. However, these deficiencies can be
11 remedied by numerical analyses, which is vital in the assessment of the sediment
12 response.

13 For the sake of simplicity and similarity, solitary-type long waves have been
14 employed for decades to investigate the tsunami behavior (Liu et al. 1991; Synolakis
15 and Bernard 2006). Special attention was paid to the wave run-up on a uniform slope
16 (Carrier et al. 2003; Hsiao et al. 2008). To implement the tsunami-like solitary wave
17 generation process, it is necessary to build a wave-maker in the wave field where the
18 solitary wave is generated and propagates over a porous sloping beach. In this study, a
19 volume-averaged Reynolds-averaged Navier-Stokes (VARANS) solver (Masuoka and
20 Takatsu 1996; Nakayama and Kuwahara 1999; Hsu et al. 2002) with the volume of
21 fluid (VOF) method (Hirt and Nichols, 1981) and $k-\varepsilon$ turbulence model (Launder and
22 Spalding 1974) is developed for modeling wave propagation within FLOW3D.

23 To sum up, two mechanisms of pore pressure generation have been observed in the
24 laboratory and field measurement, i.e., oscillatory and residual mechanisms. Most
25 previous investigations have only considered one of them, rather than integrated both

mechanisms. Furthermore, previous investigations have been limited to semi-elastic simulation or non-cyclic plastic simulation concerning mainly pore water pressures. In this study, both oscillatory and residual mechanisms of the wave-induced pore pressure are considered by soil cyclic mobility. In section 2, the oscillatory mechanism is modeled with Biot u - p theory (Zienkiewicz et al. 1980), while the residual mechanism is modeled by plastic theory with cyclic mobility. Water wave motion is simulated using a momentum source function with volume of fraction (VOF) method. This model is validated with data from previous studies in Section 3. Detailed discussion of excess pore water pressure ratio (EPWPR) development, effective stress path and pore water pressures are presented in Section 4. Special attention is given to the influence of soil permeability coefficient and various coast slopes on the soil response subjected to tsunami-like solitary waves.

2. Theoretical Formulation

Two sub-modules are included in the proposed numerical approach: wave generation module and beach module. The wave module is established for generating waves and describing their propagation in a viscous fluid. The beach module is used to determine the beach responses to the waves, including EPWPR (excess pore water pressure ratio), mean effective stress and stress path.

2.1 Wave module

Based on mass and momentum conservation equations, the VARANS equations are adopted to describe the motion of incompressible viscous fluid (Masuoka and Takatsu 1996; Nakayama and Kuwahara 1999; Hsu et al. 2002). The governing equations can be expressed as:

$$\frac{\partial \langle u_{fi} \rangle}{\partial x_i} = 0 \quad (1)$$

$$\begin{aligned}
& \frac{\partial \rho_f \langle u_{fi} \rangle}{\partial t} + \frac{\partial \rho_f \langle u_{fi} \rangle \langle u_{fj} \rangle}{\partial x_j} = - \frac{\partial \langle p_f \rangle}{\partial x_i} + \frac{\partial}{\partial x_j} \left[\mu \left(\frac{\partial \langle u_{fi} \rangle}{\partial x_j} + \frac{\partial \langle u_{fj} \rangle}{\partial x_i} \right) \right] \\
& + \frac{\partial}{\partial x_j} \left(-\rho_f \langle u'_i u'_j \rangle \right) + \rho_f g_i
\end{aligned} \tag{2}$$

where x_i is the Cartesian coordinate, ρ_f is the density of fluid. $\langle u_{fi} \rangle$ is the ensemble mean velocity. $\langle p_f \rangle$ is the pressure of fluid, μ is the dynamic viscous, t is the time and g_i is the gravitation acceleration. The Reynolds stress term $-\rho_f \langle u'_i u'_j \rangle$, is modeled by the sophisticated 2-equations k - ε turbulence model (Launder and Spalding 1974; Rodi 1993) and then Reynolds stress term can be estimated by

$$-\rho_f \langle u'_i u'_j \rangle = \mu_t \left[\frac{\partial \langle u_{fi} \rangle}{\partial x_j} + \frac{\partial \langle u_{fj} \rangle}{\partial x_i} \right] - \frac{2}{3} \rho_f \delta_{ij} k \tag{1}$$

where μ_t is the turbulent viscosity, k is the turbulence kinetic energy and δ_{ij} is the Kronecker delta. Based on Eq. (3), Eq (2) can be rewritten as

$$\begin{aligned}
& \frac{\partial \rho_f \langle u_{fi} \rangle}{\partial t} + \frac{\partial \rho_f \langle u_{fi} \rangle \langle u_{fj} \rangle}{\partial x_j} = - \frac{\partial}{\partial x_i} \left[\langle \rho_f \rangle + \frac{2}{3} \rho_f k \right] \\
& + \frac{\partial}{\partial x_j} \left[\mu_{eff} \left(\frac{\partial \langle u_{fi} \rangle}{\partial x_j} + \frac{\partial \langle u_{fj} \rangle}{\partial x_i} \right) \right] + \rho_f g_i
\end{aligned} \tag{2}$$

in which $\mu_{eff} = \mu + \mu_t$ is the total effective viscosity.

A sophisticated and widely used equation for k - ε turbulence model which consists of two transport equations for the turbulent kinetic energy and its dissipation can be expressed as (Launder and Spalding 1974)

$$\frac{\partial \rho k}{\partial t} + \frac{\partial \rho \langle u_j \rangle k}{\partial x_j} = \frac{\partial}{\partial x_j} \left[\left(\mu + \frac{\mu_t}{\sigma_k} \right) \frac{\partial k}{\partial x_j} \right] + \rho P_k - \rho \varepsilon \tag{5}$$

$$\frac{\partial \rho \varepsilon}{\partial t} + \frac{\partial \rho \langle u_j \rangle \varepsilon}{\partial x_j} = \frac{\partial}{\partial x_j} \left[\left(\mu + \frac{\mu_t}{\sigma_\varepsilon} \right) \frac{\partial \varepsilon}{\partial x_j} \right] + \frac{\varepsilon}{k} (C_{\varepsilon 1} \rho P_k - C_{\varepsilon 2} \rho \varepsilon) \tag{6}$$

$$\mu_t = \rho C_\mu \frac{k^2}{\varepsilon} \quad (7)$$

in which ε is the rate of turbulent energy dissipation, μ_t is the turbulent kinematic viscosity, C , C_μ are the coefficients in approximated turbulent transport equations.

The VOF method (Hirt and Nichols, 1981) are used to track the free water surface, which consists of three main components: the definition of the volume of fluid function, a method to solve the VOF transport equation and setting the boundary conditions at the free surface. The idea of VOF method is to define a function F to represent the fractional volume water fluid

$$\frac{\partial F}{\partial t} + \frac{\partial(u_i F)}{\partial x_i} = 0 \quad (8)$$

where $F = 0$ indicates that the cell is full of air, and $F = 1$ indicates that the cell is full of water. The piecewise linear interface calculation (PLIC) method (Rider and Kothe, 1998) is applied to reconstruct the air-water interface.

2.2 Beach module

The mechanical behaviors of sandy seabed are dependent not only on the shape of particles, angular or round, but also on its density, its experienced strain history, and even on the degree of structure formed in its deposition. Sand may behave totally differently under different loading and drained condition. For instance, when subjected to undrained cyclic loading, loose sand will liquefy without transition from contractive to dilative state; for medium dense sand, liquefaction with cyclic mobility occurs while for dense sand, the liquefaction will never occur. Loose sand subjected to cyclic loading will liquefy under undrained condition but may be compacted to a denser state under drained condition. Based on the concepts of subloading (Hashiguchi 1977) and superloading (Asaoka et al. 1998), Zhang et al. (2007, 2011) and Ye et al. (2011) proposed a new constitutive model. The newly proposed model is

1 able to describe not only the mechanical behavior of soils under monotonic loading,
 2 but also the behavior of soils under dynamic loading with different drained condition.
 3 A brief description of the yield surfaces is presented in Fig. 1a.

4 Two state variable, that is, R^* , the similarity ratio of the superloading yield surface
 5 to normal yield surface, and R , the similarity ratio of the superloading yield surface to
 6 subloading yield surface, are defined in the same way as those in (Asaoka et al. 1998),
 7 namely,

$$8 \quad R^* = \frac{\hat{p}}{\bar{p}} = \frac{\hat{q}}{\bar{q}}, \quad 0 < R^* \leq 1 \quad \text{and} \quad \frac{\hat{a}}{\hat{p}} = \frac{a}{p} \quad (9)$$

$$9 \quad R = \frac{p}{\bar{p}} = \frac{q}{\bar{q}}, \quad 0 < R \leq 1, \quad \text{and} \quad \frac{\bar{q}}{\bar{p}} = \frac{\hat{a}}{\hat{p}} = \frac{a}{p} \quad (10)$$

10 where p is mean stress, q is deviatoric stress. (p, q) , $(\hat{p}, \hat{q})$ and $(\bar{p}, \bar{q})$ represent the
 11 present stress state, the corresponding normally consolidated stress state and the
 12 structured stress state on the p - q plane, respectively. The present stress state is always
 13 situated on the subloading surface, which is given in the following form:

$$14 \quad f = \ln \frac{\sigma_m}{\sigma_{m0}} + \ln \frac{M^2 - \zeta^2 + \eta^{*2}}{M^2 - \zeta^2} + \ln R^* - \ln R - \frac{\varepsilon_v^p}{C_p} = 0 \quad (11)$$

15 In Eq. (11), σ_m is the mean effective stress, and $\sigma_{m0} = 98\text{kPa}$ is a reference stress.

16 $\zeta = \sqrt{\frac{3}{2} \beta_{ij} \beta_{ij}}$ is an anisotropic state variable, with β_{ij} as the anisotropic stress tensor.

17 M is the critical state ratio. $\eta^* = \sqrt{\frac{3}{2} \hat{\eta}_{ij} \hat{\eta}_{ij}}$ represents the difference between the
 18 stress ratio tensor η_{ij} and the anisotropic stress tensor β_{ij} , in which

$$19 \quad \hat{\eta}_{ij} = \eta_{ij} - \beta_{ij}, \quad \eta_{ij} = \frac{S_{ij}}{\sigma_m}, \quad S_{ij} = \sigma_{ij} - p\delta_{ij} \quad (12)$$

where S_{ij} is the deviatoric stress tensor, σ_{ij} is the effective stress tensor and δ_{ij} is Kronecker delta tensor.

In Eq. (11), C_p is expressed as:

$$C_p = \frac{\lambda - \kappa^{si}}{1 + e_0} \quad (13)$$

where λ and κ^{si} are the compression and swelling index, and e_0 is a reference void ratio at pressure $p_0 = 98$ kPa.

In this model, the gradient of the Critical State Line (C.S.L.) is assumed to be constant. Fig.1b shows the yielding surfaces and its change in the flat ratio of the elliptical yield surface due to the anisotropy. The third state variable introduced in the model, ζ , the stress-induced anisotropy, is assumed that the larger the ζ is, the larger the eccentric ratio of the ellipse will be.

An associated flow rule is employed in the model:

$$d\varepsilon_{ij}^p = \Lambda \frac{\partial f}{\partial \sigma_{ij}} \quad (14)$$

where, ε_{ii}^p is the plastic strain and Λ is the plastic multiplier.

The consistency equation for the subloading yield surface can then be given as:

$$df = 0 \text{ } \mathbb{P} \frac{\mathbb{P}f}{\mathbb{P}s_{ij}} ds_{ij} + \frac{\mathbb{P}f}{\mathbb{P}b_{ij}} db_{ij} + \frac{1}{R^*} dR^* - \frac{1}{R} dR - \frac{1}{C_p} de_v^p = 0 \quad (15)$$

The evolution rule for the degree of structure, R^* , is defined as:

$$dR^* = U^* d\varepsilon_a^p, \quad U^* = \frac{aM}{C_p} R^* (1 - R^*) \quad (0 < R^* \leq 1) \quad (16)$$

in which a is a parameter that controls the rate of collapse of the structure during shearing.

The changing rate of overconsolidation is assumed to be controlled by two factors, namely, the plastic component of strain and the incremental anisotropy:

$$dR = U \|d\varepsilon_{ij}^p\| + R \frac{\eta}{M} \frac{\partial f}{\partial \beta_{ij}} d\beta_{ij}, \quad (17)$$

where

$$\|d\varepsilon_{ij}^p\| = \sqrt{d\varepsilon_{ij}^p d\varepsilon_{ij}^p} = \Lambda \sqrt{\frac{\partial f}{\partial \sigma_{ij}} \frac{\partial f}{\partial \sigma_{ij}}} = \Lambda \frac{\sqrt{6\eta^{*2} + \frac{1}{3}(M^2 - \eta^2)^2}}{(M^2 - \zeta^2 + \eta^{*2})\sigma_m} \quad (18)$$

$$U = -\frac{mM}{C_p} \left(\frac{(\sigma_m / \sigma_{m0})^2}{(\sigma_m / \sigma_{m0})^2 + 1} \right) \ln R \quad (\sigma_{m0} : \text{reference stress, } = 98.0 \text{ kPa}) \quad (19)$$

Here, m is a parameter that controls the losing rate for over consolidation.

The evolution rule for the anisotropic stress tensor is defined as:

$$d\beta_{ij} = \frac{M}{C_p} b_r (M - \zeta) d\varepsilon_d^p \frac{\hat{\eta}_{ij}}{\|\hat{\eta}_{ij}\|} = \sqrt{\frac{3}{2}} \frac{M}{C_p} b_r (M - \zeta) d\varepsilon_d^p \frac{\hat{\eta}_{ij}}{\eta^*}, \quad (20)$$

where b_r is a parameter that controls the developing rate of anisotropy, and $d\varepsilon_d^p$ is the plastic deviatoric strain.

If the incremental strain tensor is divided into elastic and plastic components, the elastic component is as follows:

$$d\sigma_{ij} = E_{ijkl} d\varepsilon_{kl}^e = E_{ijkl} (d\varepsilon_{kl} - d\varepsilon_{kl}^p) = E_{ijkl} d\varepsilon_{kl} - \Lambda E_{ijkl} \frac{\partial f}{\partial \sigma_{kl}} \quad (21)$$

where ε_{kl} is the strain tensor, ε_{ii}^e is the elastic strain tensor and E_{ijkl} is the elastic module tensor.

Therefore, by substituting Eqs. (16), (17), (18), (20) and (21) into Eq. (15), the positive variable Λ can be rewritten as :

$$\Lambda = \frac{\frac{\partial f}{\partial \sigma_{ij}} E_{ijkl} d\varepsilon_{kl}}{h_p + \frac{\partial f}{\partial \sigma_{ij}} E_{ijkl} \frac{\partial f}{\partial \sigma_{kl}}}, \quad (22)$$

where:

$$h_p = \frac{1}{C_p(M^2 - \zeta^2 + \eta^{*2})\sigma_m} [M_s^2 - \eta^2] \quad (23)$$

$$M_s^2 = M^2 - \frac{mM \ln R}{R} \left[\frac{(\sigma_m / \sigma_{m0})^2}{(\sigma_m / \sigma_{m0})^2 + 1} \right] \sqrt{6\eta^{*2} + \frac{1}{3}(M^2 - \eta^2)^2} - 2aM(1 - R^*)\eta^* + (1 - \frac{\eta}{M}) \frac{\sqrt{6Mb_r(M - \zeta)\eta^{*2}(2M^2 - 3\eta_{ij}\beta_{ij})}}{(M^2 - \zeta^2 + \eta^{*2})(M^2 - \zeta^2)} \quad (24)$$

The loading criteria are given below:

$$\begin{cases} \Lambda > 0 & \text{loading} \\ \Lambda = 0 & \text{neutral} \\ \Lambda < 0 & \text{unloading} \end{cases} \quad (25)$$

Eight parameters are involved in the proposed model, among which five parameters, M , e_0 , λ , κ^{si} and ν are the same as in the Cam-clay model. The other three parameters are listed below.

m : parameter which controls the losing rate of over consolidation.

a : parameter which controls the decay rate of structure.

b_r : parameter which controls the developing rate of stress-induced anisotropy.

These three parameters have clear physical meanings and can be easily determined through laboratory tests (Zhang et al. 2007, 2011; Ye et al. 2007, 2011, 2012). The model has been successfully applied to numerical analyses on various dynamic responses phenomena (Bao et al. 2012; Bao et al. 2012, 2014; Xia et al. 2010; Ye et al. 2007, 2011, 2012; Zhang et al. 2011).

2.3 Soil-water fully coupled FEM in beach

In the dynamic finite element analysis of sandy slope, a fully coupled two-phase field theory with u - p formulation (Zienkiewicz and Shiomi 1984; Oka et al. 1994) is adopted, where the displacement increment of soil skeleton and pore water pressure are the unknown variables. The governing equations are given out as follows:

1 (1) Equilibrium equation:

$$2 \quad \rho_s \ddot{u}_i^s = \frac{\partial \sigma_{ij}}{\partial x_j} + \rho_s b_i \quad (26)$$

3 where ρ_s and u^s denote the soil density and displacement, respectively, b_i is the body
4 force.

5 (2) Continuity equation:

$$6 \quad \rho_f \ddot{\varepsilon}_{ii}^s - \frac{\partial^2 p_d}{\partial x_i \partial x_i} - \frac{\gamma_w}{k} \left(\frac{\dot{\varepsilon}_{ii}^s}{n} - \frac{1}{K^f} \dot{p}_d \right) = 0 \quad (27)$$

7 where ε_{ii}^s is the volumetric strain of soil skeleton, ρ_f is the fluid density, p_d is the pore
8 pressure, γ_w is the unit weight of the pore fluid, k_s is the coefficient of permeability, K^f
9 and n denote the volumetric compressibility of the fluid and the soil porosity,
10 respectively.

11 A finite element and finite difference (FE–FD) hybrid scheme is adopted for spatial
12 discretization of the governing equations (26) and (27). In the method, the
13 displacements in the equilibrium equation are discretized in the space domain by the
14 finite element method, whereas the terms of excess pore pressures in the continuity
15 equation are spatially discretized by the finite difference method. A brief procedure is
16 given in the following. Detailed formulation of the numerical method and its
17 evaluation of accuracy are available in Jeng and Ou (2010).

18 The equilibrium equation (26) is discretized in the space domain by the finite
19 element method:

$$20 \quad [M] \ddot{\bar{u}}_N + \mathbf{K}_v p_{dE} = \bar{F}_d - \bar{K}_d|_t \quad (28)$$

21 where $[M]$, $[C]$, $[K]$ are the mass matrix, damping matrix and stiffness matrix,
22 respectively. K_v is the fluid conservation vector, $p_{dE} = p_E - p_{E|t=0}$ is the excessive

1 pore water pressure, F_d is the incremental load vector, and the residual force vector
 2 $R_{d|t}$ is defined as follows:

$$3 \quad \bar{R}_{d|t} = \int_V [B]^T (\bar{\sigma}'_t - \bar{\sigma}'_{t=0}) dV \quad (29)$$

4 Rayleigh damping is incorporated into the dynamic equations, using a linear
 5 combination of the mass matrix $[M]$ and the stiffness matrix $[K]$ for the construction
 6 of a damping matrix $[C]$:

$$7 \quad [C] = \alpha_0 [M] + \alpha_1 [K] \quad (30)$$

8 The proportionality factors α_0 and α_1 can be evaluated from damping ratios and
 9 specific frequencies of the system.

10 The excess pore pressures in the continuity equation (27) are spatially discretized
 11 by the finite difference method:

$$12 \quad \rho^f \bar{K}_v^T \ddot{u}_N - \frac{\gamma_w}{nk} \bar{K}_v^T \ddot{u}_N - \alpha p_{dE} + \sum_{i=1}^m \alpha_i p_{dEi} + A \dot{p}_{dE} = 0 \quad (31)$$

13 where α_i is defined by both the length of the side of the element concerned and the
 14 distance between the gravity center of the neighboring element i and that of the
 15 element concerned, and $\alpha = \sum_{i=1}^4 \alpha_i$. A is defined as follows:

$$16 \quad A = \int_V \frac{\gamma_w}{kK^f} dV \quad (32)$$

17 Finally, Newmark- β method is used for the integration of the field equations (28)
 18 and (31) in the time domain.

19 Based on the FE–FD hybrid scheme, a static/dynamic 2D/3D finite element
 20 analysis program DBLEAVES was developed (Ye et al. 2007). The applicability and
 21 accuracy of the program have been firmly verified by the investigation on group-pile
 22 foundations in full-scale test (Jin et al. 2010), shaking table tests (Bao et al. 2012) and

1 other various liquefaction phenomena (Bao et al. 2014; Xia et al. 2010; Ye et al.
2 2011).

3 **3. Model Verification**

4 *3.1 Wave model validation*

5 In this section, the breaking solitary wave run-up on the beach is simulated.
6 Synolakis (1986) and Zelt (1991) carried out a series of wave flume experiments to
7 investigate the sequence of breaking solitary wave run-up on the beach and the
8 experimental results are used to validate the accuracy of proposed numerical analysis
9 method.

10 The detailed experimental setup for this testing problem is referred to Synolakis
11 (1986) and Zelt (1991). The still water depth varies from 0.21m to 0.29m. The ratio of
12 wave height to water depth H/d is equal to 0.28. The beach slope is 2.88° and is about
13 $1/20$. Wave gauges were used to record the free surface displacement during the
14 run-up and run-down in the flume experiments. In the numerical simulation, the
15 uniform grid system is with $\Delta x=0.02\text{m}$, $\Delta z=0.002\text{m}$. The $k-\varepsilon$ model is employed to
16 simulate the turbulence process during the computations. Since the water depth varies
17 in the experiment, the results can only be compared after normalization. Following
18 Zelt (1991), the length scale is normalized by the water depth and the time is
19 normalized by $\sqrt{g/d}$. Figure 2 shows the comparisons of experimental data and
20 numerical results in terms of the free surface profile. The whole process of wave
21 breaking, run-up, and run-down is provided. The agreement between the simulated
22 wave profiles and experimental data is excellent. On the whole, the comparison with
23 the flume experiments verifies the validation of our numerical analysis method.

3.2 Soil model validation

Although the numerical analysis method presented in Section 2 has been validated for earthquake induced liquefaction, it had not been tested for wave loading. In this session, the experimental results from flume tests of Sumer et al. (2006) are used to validate the accuracy of proposed numerical analysis method.

Sumer et al. (2006) carried out a series of wave flume experiments to investigate the sequence of sediment behavior beneath progressive waves. Fig.3 shows a schematic of the wave flume and the soil box. Among their seven tests with various wave loading (mainly the wave height), the test 4, whose liquefaction procedure was described in detailed in the article, was used to evaluate the ability of the numerical method to predict the progressive buildup of pore pressure under wave loading. The wave parameters for the progressive wave in test 4 were: wave height $H=16\text{cm}$, wave period $T=1.6\text{ sec}$, water depth= 42 cm . Four pore pressure transducers (PPTs) were installed at the depths of $Z=2.5\text{cm}$, 7.5cm , 12.5cm and 17cm .

The silt with a particle size $d_{50}=0.060\text{mm}$ and a relative density $D_r=38\%$ was filled in the soil box before testing. Usually, the material parameters and the initial state variable of the Cyclic Mobility model (CM model) are calibrated by advanced laboratory tests (i.e., cyclic triaxial tests), however, these test data of the silt were not available.

The sediment model is a 2D model with a dimension of 90cm (width) * 17.5cm (height). The left and right boundaries are fixed in horizontal direction and free in vertical direction. The bottom is fixed in both horizontal and vertical directions. Since the wave made by the wave-maker is difficult to simulate, the water surface elevation is simplified to progress wave obtained from the small-amplitude, linear wave theory (Dally et al. 1984). In the wave model, the domain length is 90cm . The wave height is

16cm and water depth is 42 cm. The lower boundary representing the concrete base is considered a zero-flux, zero-displacement boundary. The left and right boundary representing the concrete wall was also considered a zero-flux, zero-displacement boundary. The upper boundary represents the free surface elevation.

Fig.4 shows the simulated and measured time series of excess pore water pressure (EPWP). When the excess pore water pressure is equal to the initial vertical effective stress, the liquefaction will occur. It can be seen that the calculation matched well with experiment, in terms of the build-up of EPWPs at different depths, and the vibrations of EPWPs after liquefaction. Moreover, the larger and larger phase delay in the pressure with increasing depth, which was observed in the experiment, also can be seen in the calculation. The numerical results reach a good agreement with the experimental data as illustrated in Fig.5. Generally, the present model has the ability to simulate wave-induced poro-elastoplastic seabed response.

4. Results and Discussion

The sloping beach model shown in Fig.6 is a single layer of sandy ground. It is a 2D model with a thickness of 2 m. The left and right boundaries are free in Z direction and fixed in X direction. The bottom is fixed in both X and Z direction. The toe of beach is located at $x = 0\text{m}$, and wave is started at $x = 0\text{m}$ at $t = 0\text{s}$. The beach slope is selected to 1:20 considering the natural condition. The seabed soils are assumed as the Toyoura sand and the Toyoura sand is Japanese standard sand that widely used in the liquefaction related element and model tests. The parameters for median dense Toyoura sand are shown in Table 1 (Zhang et al., 2011). In the wave model, the wave maker is located at left boundary. For the model presented in Fig. 6, the similar boundary conditions where the lower boundary representing the concrete base is considered a zero-flux, zero-displacement boundary and the right boundary

1 representing the concrete wall is also considered a zero-flux, zero-displacement
2 boundary, are applied as in the case of the model presented in section 3. The upper
3 boundary is broken into two sections, one representing the exposed bed surface, and
4 one representing the free surface elevation.

5 In this section, a 0.2m height solitary wave over various slope angles is simulated
6 to investigate the corresponding coast slope responses, as shown in Fig. 6. These
7 responses consist of EPWPR and effective stress path within the beach. Special
8 attention is given to two different positions $x=7.6$ m and $x=10$ m, which are closely
9 near the shoreline. For engineering practice, the influence of soil permeability
10 coefficient and various coast slopes on the soil response subjected to tsunami-like
11 solitary waves are also discussed.

12 *4.1 Development of EPWPR in beach*

13 The excess pore water pressure ratio ($EPWPR = u / \sigma'_{v0}$), defined as the ratio of the
14 excess pore water pressure to the initial vertical effective stress (the initial vertical
15 effective stress is equal to the effective gravity multiplied by the depth), is widely
16 used as an index to indicate liquefaction. Generally, $EPWPR \geq 0.95$ is adopted as
17 liquefaction criterion to ensure the numerical stability while without loss of
18 generality. Based on this criterion, the analysis on EPWPR is an important part in this
19 study to investigate the response of the beach.

20 The development of EPWPR within a 1:20 sloping beach under solitary wave
21 loadings is shown in Fig.7. Red lines indicate the solitary wave pressure that acts on
22 the slope surface. The results show that the EPWPR increases with the wave pressure
23 gradient (Fig.7a) and there exists a maximum EPWPR area just below the wave crest
24 (Fig.7b). The EPWPR develops at the shallow soil of beach firstly, then propagating
25 downward into the deep soil. The highest EPWPR appears at $x=7.6$ m right before the

1 sloping shoreline (Fig.7b). It is found that the maximum EPWPR area moves
2 progressively with the forward propagation of the wave. However, the solitary wave
3 begins to break in the front of shoreline ($x=10\text{m}$) and the EPWPR decreases because
4 of the wave-breaking effect. (Fig.7c and 7d) After that, the solitary wave runs up
5 along the slope, resulting in the decrease of the wave pressure subsequently (Fig.7e).
6 At the final stage, there is a negative pressure at the slope due to the hydraulic jump
7 formed by wave drawdown off the slope (Fig.7f).

8 Fig.8 shows the time histories of the distribution of EPWPR along the soil depth
9 direction at $x=7.6\text{m}$, where the highest EPWPR appeared. It can be seen that the
10 EPWPR appears maximum at $t=5\text{s}$ and the maximum EPWPR forms at the shallow
11 soil of the beach. Furthermore, the distribution of EPWPR along the depth direction
12 shows a decreasing trend. The largest EPWPR value in these positions is about 0.59 at
13 0.125m, 0.21 at 0.25m, and 0.12 at 0.5m. This is because the shear strength increases
14 in the shallow zone, due to the densification during the wave propagating. After the
15 wave run-up and down the slope, a hydraulic jump formed offshore at $t=9\text{s}$ while the
16 EPWPR increases with the hydraulic jump subsequently.

17 Fig.9 illustrates the time history of the distribution of EPWPR in the depth direction
18 at $x=10\text{m}$ right on the shoreline. It can be observed that the maximum EPWPR also
19 develops at the shallow soil of the beach. The distribution of EPWPR along the depth
20 direction shows a decreasing trend as well as that shown in Fig.8. However, the value
21 of EPWPR is considerably reduced than that of position at $x=7.6\text{m}$. The largest
22 EPWPR value is about 0.29 at 0.125m and 0.11 at 0.25m. This is because the wave
23 began to break, resulting in a considerable reduction of pore water pressure. In the
24 stage of wave draw down from the slope at $t=12\text{s}$, the surface elevation is lower than

the initial water surface. In addition, a negative pressure forms and the negative pressure also shows a decreasing trend along the depth direction.

4.2 Effective stress path

The effective stress path reveals the soil stress state and stress history. The soil stress state represents the shear stress and the mean effective stress of the soil, which indicates the strength and deformation of soil. On the other hand, various stress history reflects different loading progress, in which the soil shows various strength properties. Hence, the effective stress path analysis is necessary in the soil response research.

Fig.10 presents the effective stress path of the element at $x=7.6\text{m}$ on the slope. The stress path shows that shear stress σ_{xy} oscillated with the wave loads. The mean effective stress σ_m decreases slightly during $t=0\sim 3.6\text{s}$. In this initial stage, the wave has not yet arrived at this position and the shear stress keeps constant. When the wave arrives at the position at 3.6s , the stress path begins to oscillate obviously. The shear stress increases to the maximum value during a short time. Subsequently, the shear stress reduces and the mean effective stress increases when solitary wave begins to draw down. After that, the wave draws down off the slope and the hydraulic jump occurs at $t=6.5\text{s}$, leading to decrement of the mean effective stress.

Fig.11 shows the effective stress path of the element at $x=10\text{m}$ right on the shoreline. The shear stress does not change at $t=0\sim 5\text{s}$, before the wave propagates at the position. The mean effective stress increases slightly because of the position is not under the water surface elevation. When the wave propagates at the position at 5s , the stress path oscillates. The shear stress oscillates constantly as the wave run-up the slope, while the mean effective stress increases. Both the shear stress and the mean effective stress reduces when the wave draw down the slope.

Fig.12 illustrates the stress path of the element at $x=7.6\text{m}$ on the slope, describing in the coordinate of $\tau_{xy} \sim (\sigma_y - \sigma_x)/2$. It is well known that the principal stress angle in plane strain condition is defined as $\tan \theta = 2\tau_{xy} / (\sigma_y - \sigma_x)$, so this kind of coordinate is used to represent the rotation of principal axes. As shown in Fig.12, when $t=0 \sim 3.6\text{s}$, the stress path starts from an initial K_0 stress state then runs leftward. Subsequently, the stress path runs around the origin anticlockwise. The round stress path is close to a circle. Ishihara and Towhata (1983) pointed out that in the case of wave motion by linear wave theory; the stress path ran around a circle with the diameter equal to stress difference $(\sigma_y - \sigma_x)$. The stress path under the solitary wave in the current study and the circle-shape one in the study of Ishihara and Towhata (1983) indicates that different kinds of wave motion produce different stress paths. Fig.13 shows the stress path of the element at $x=10\text{m}$ on the shoreline, describing in the coordinate of $\tau_{xy} \sim (\sigma_y - \sigma_x)/2$. It can be observed that in the first $0 \sim 5\text{s}$, the stress path starts from an initial K_0 stress state, then runs leftward (the same as Fig.12). During the $t=5 \sim 12\text{s}$, the stress path runs anticlockwise for more than one circle. That is because of the wave run-up and drawn down the shoreline. The round stress path also proved that the principal axes rotate under the wave load. Laboratory tests (Ishihara and Towhata 1983; Nakata et al. 1998) indicate that the pore pressures and strain are accumulated under rotation of principal stresses despite the deviatoric stress remaining constant and consequently the strength of sand is reduced. The influence of principal axes rotation to the strength of sand under solitary wave should be taken into consideration in the future.

4.3 Influence of the permeability coefficient on the EPWPR

Permeability coefficient is one of the most important seabed parameters in soil dynamic response analysis. It has an obvious influence on the excess pore water

1 pressure development, which plays an important role in seabed liquefaction. To
2 investigate the seabed liquefaction and seabed deformation, permeability coefficient is
3 a key factor to be considered.

4 To study the influence of permeability coefficient on the dynamic response of
5 coastal slope, different permeability coefficients ($k_s=5\times10^{-2}\text{m/s}^2$, $1\times10^{-3}\text{m/s}^2$,
6 $5\times10^{-3}\text{m/s}^2$, $1\times10^{-4}\text{m/s}^2$, $5\times10^{-4}\text{m/s}^2$, $1\times10^{-5}\text{m/s}^2$) are analyzed. Fig.14 shows the
7 impact of permeability coefficient on the effective stress path and maximum EPWPR
8 value of the position at $x=7.6\text{m}$ on the slope. As can be seen from Fig.14a, when the
9 permeability coefficient k_s is greater than $1\times10^{-4}\text{m/s}^2$, the shear stress and the mean
10 effective stress oscillate and return to the initial state eventually. When the
11 permeability coefficient k_s is less than $1\times10^{-4}\text{m/s}^2$, the effective stress paths present
12 similar development pattern. We find that the maximum EPWPR increases with the
13 soil permeability coefficient, shown in Fig.14b. Furthermore, the increasing trend is
14 gradually weakened, especially when the permeability coefficient k_s is less than
15 $1\times10^{-4}\text{m/s}^2$. When the permeability coefficient k_s is smaller than $1\times10^{-4}\text{m/s}^2$, the
16 variations of effective stress path become neglectful. Fig.15 shows the impact of
17 permeability coefficient on the effective stress path and maximum EPWPR value of
18 the position at $x=10\text{m}$ on the shoreline. It can be observed from Fig.15a that the
19 effective stress path shows a similar trend as that in Fig.14a. When the permeability
20 coefficient k_s is greater than $1\times10^{-4}\text{m/s}^2$, the shear stress and the mean effective stress
21 oscillate obviously. The maximum EPWPR also increases with increasing
22 permeability coefficient as shown in Fig.15b.

23 *4.4 Influence of the various slopes on the EPWPR*

24 The solitary wave will break when it propagates at the coast slope. That makes the
25 wave pressure distribution change. In general, the coast slope varies from place to

place. Different coast slopes lead to different wave breaking and different wave pressure distribution. Therefore, it is necessary to investigate the influence of various slopes on slope response.

The coast slope is 1:20 in the research of Synolakis (1987) and Zelt (1991). Various slopes (1:15, 1:10, and 1:5) are selected to analyze the extreme cases. Figure 16 shows the impact of slope on the effective stress path and maximum EPWPR of the position on the shoreline. It can be seen from Fig.16a that the effective stress path shifts leftward with the increase of the slope. It reveals that the shear stress increases and the mean effective stress declines. The pore water pressure rises with the reduction of the mean effective stress and liquefaction will occur eventually, when the EPWPR reaches 1.0. The mean effective stress is obviously small and excess pore pressure is considerably large in the case of slope 1:10, which means liquefaction is about to happen. Fig.16b shows the relationship between the maximum EPWPR and the slope at the position on the shoreline. Gravity is applied to the seabed model to calculate the initial vertical effective stress of each element for the sloping seabed. The initial vertical effective stress decreases with the increased slope steepness due to the loss of horizontal stress, which causes high EPWPR. However, the degree of wave breaking rises with the increasing of the slope. Eventually, the maximum EPWPR almost increases linearly with the increase of the slope before the EPWPR reached 1.0 as shown in Fig.16b. The EPWPR reaches 1.0 and the liquefaction occurs in the case of 1:5. It can be predicted that the liquefaction will occur when the coastal slope between 1:10 and 1:5. For the case of slope larger than 1:10, preventive measures are needed.

5. Conclusions

1 In this study, a coupled approach for the investigation of tsunami-like solitary
2 wave-induced slope response is developed. The volume-averaged Reynolds-averaged
3 Navier–Stokes (VARANS) equations are employed to govern the ocean wave motion,
4 while the poro-elastoplastic equations are adopted to describe the mechanical
5 behavior of the beach under wave loading. The proposed model is validated against
6 experimental test available in the literature. An overall agreement between the present
7 model and the experimental measurements demonstrates the capacity of the present
8 model to predict the behavior of the solitary wave-slope interaction.

9 Based on the results obtained, the following conclusions can be drawn:

10 (1) The EPWPR in the shallow soil is affected significantly by the solitary waves.
11 The EPWPR develops at the shallow soil of beach firstly, then propagating downward
12 into the deep soil. The distribution of EPWPR along the depth direction shows a
13 decreasing trend. It is found that the maximum EPWPR area moves progressively
14 with the forward propagation of the wave. At the final stage, there is a negative
15 pressure at the slope due to the hydraulic jump formed by wave drawdown off the
16 slope.

17 (2) The stress path of soil element will oscillate obviously when the wave
18 propagates close to the shoreline. After that, mean effective stress increases as the
19 wave run-up the slope and declines when wave draw down the slope. Meanwhile,
20 shear stress history is various in the offshore and shoreline due to the influence of
21 slope and hydraulic jump. Throughout the wave loading process, the principal axes
22 rotation is proved and the influence of principal axes rotation to the strength of sand
23 under solitary wave should be taken into consideration in the future.

24 (3) Various permeability coefficients result in diverse effective stress path patterns.
25 When the permeability coefficient k_s is greater than $1 \times 10^{-4} \text{ m/s}^2$, the shear stress and

1 the mean effective stress oscillates obviously. On the other hand, the effective stress
2 paths tend steady and present similar development pattern when the permeability
3 coefficient k_s is no more than $1 \times 10^{-4} \text{m/s}^2$. In addition, larger shear stress in oscillating
4 effective stress path leads to significant EPWPR response. However, the increasing
5 trend is gradually weakened, especially when the permeability coefficient k_s is less
6 than $1 \times 10^{-4} \text{m/s}^2$.

7 (4) Greater slope leads to larger shear stress and lower mean effective stress.
8 Hence, the effective stress path shifts leftward with the increasing of the slope.
9 Meanwhile, the decline of the mean effective stress leads to the raise of the EPWPR
10 and the maximum EPWPR reaches 1.0 eventually, where the liquefaction occurs in
11 the case of slope 1:5. In addition, the maximum EPWPR almost increases linearly
12 with the growth of the slope before the case 1:10. It can be deduced that the
13 liquefaction had occurred between the slope 1:10 and 1:5.

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10

Table 1 Input parameters used for numerical validation (in section 3)

Parameter	Value	Unit
Seabed Parameters		
Relative density D_r	38%	-
Permeability coefficient k	10^{-5}	m/s ²
Compression index λ	0.06	-
Swelling index k	0.008	-
Critical state parameter M	1.2	-
Void ratio N ($p'=98\text{kPa}$ on N.C.L)	0.8	-
Poisson's ratio ν	0.30	-
Degradation parameter of overconsolidation state m_R	0.1	-
Degradation of structure m_R^*	2.2	-
Evolution parameter of anisotropy b_r	1.5	-
Initial degree of structure R_0^*	0.7	-
Initial consolidation ratio I/R_0	25	-
Initial anisotropy ζ_0	0.0	-
Wave Parameters		
Wave height H	0.16	m
Water depth d	0.42	m
Water period T	1.6	s

Table 2 Input parameters used for numerical simulation (in section 4)

Parameter	Value	Unit
Seabed Parameters		
Relative density D_r	62%	-
Permeability coefficient k	10^{-4}	m/s ²
Compression index λ	0.05	-
Swelling index k	0.0064	-
Critical state parameter M	1.2	-
Void ratio N (p'=98kPa on N.C.L)	0.74	-
Poisson's ratio ν	0.30	-
Degradation parameter of overconsolidation state m_R	0.1	-
Degradation of structure m_{R^*}	2.2	-
Evolution parameter of anisotropy b_r	1.5	-
Initial degree of structure R_0^*	0.8	-
Initial consolidation ratio I/R_0	25	-
Initial anisotropy ζ_0	0.0	-
Wave Parameters		
Wave height H	0.2	m
Water depth d	0.5	m

FIG.1 Yield surfaces in p - q plane

FIG.2 Profile of breaking solitary wave run-up on beach in validation

FIG.3 Illustration of wave flume and soil box in the experiments of Sumer et al. (2006)

FIG.4 Comparison of FEM calculated results and experimental data, in terms of build-up of EPWP

FIG.5 Comparison of FEM calculated results and experimental data, in terms of time development of liquefaction depth

FIG.6 Coastal slope model and the position mainly analyzed

FIG.7 Time histories of EPWPR development within a 1:20 slope

FIG.8 Time histories of EPWPR at $x=7.6\text{m}$ in different depth

FIG.9 Time histories of EPWPR at $x=10\text{m}$ in different depth

FIG.10 Effective stress path of element at $x=7.6\text{m}$ (depth= 0.125m) on the slope

FIG.11 Effective stress path of element at x=10m (depth=0.125m) on the shoreline

FIG.12 Effective stress path of element at $x=7.6\text{m}$ (depth= 0.125m) on the slope, in the coordinate of $\sigma_{xy} \sim (\sigma_y - \sigma_x)$

FIG.13 Effective stress path of element at x=10m (depth=0.125m) on the shoreline, in the coordinate of $\sigma_{xy} \sim (\sigma_y - \sigma_x)$

(a) Effective stress path

(b) The maximum EPWPR

FIG.14 Effective stress path and maximum EPWPR of element at $x=7.6\text{m}$ (depth=0.125m) on the slope in calculation with different permeability coefficient

(a) Effective stress path

(b) The maximum EPWPR

FIG.15 Effective stress path and maximum EPWPR of element at $x=10$ m (depth=0.125m) on the shoreline in calculation with different permeability coefficient

(a) Effective stress path

(b) The maximum EPWPR

FIG.16 Effective stress path and maximum EPWPR of element at $x=10\text{m}$ (depth= 0.125m) on the shoreline in calculation with different slope steepness