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Introduction of a quasi-coupled hyperbolic stress-strain constitutive model
Introducción de un modelo constitutivo hiperbólico cuasiacoplado de esfuerzo-deformación

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Abstract

Simplified nonlinear effective stress constitutive models are commonly used in one-dimensional (1D) geotechnical site response analysis for assessment of porewater pressure generation and liquefaction potential in soft soil deposits. This study presents the performance of a 1D quasi-coupled constitutive model termed MRDF + u (modulus reduction and damping curve fit using a reduction factor and including porewater pressure generation, u) at a particular history case (i.e., Port Island, Japan), where liquefaction effects have been reported and the site could be potentially liquefied again. The study included evaluation of the performance of two porewater pressure generation models (Dobry-PWP and GMP-PWP models) coupled into the MRDF + u constitutive model using the Port Island history case. The new coupled model reasonably captures the soil cyclic behavior observed in the history case and may be used to perform effective stress-based 1D site response analysis in engineering practice.
Keywords: Constitutive model, liquefaction, modulus degradation, site response analysis.

Resumen

Los modelos constitutivos simplificados no lineales de esfuerzo-deformación se utilizan comúnmente en análisis geotécnico unidimensional (1D) de "respuesta de sitio" para evaluar el potencial de licuación y la generación de presiones de poros en depósitos de suelos blandos. Este documento presenta los resultados del modelo constitutivo 1D cuasiacoplado denominado MRDF + u (ajuste de las curvas de reducción de módulo y de amortiguamiento empleando un factor de reducción e incluyendo la generación de presiones de poro, *u*) en un caso histórico particular (Port Island, Japón), donde se han reportado efectos de licuación y donde potencialmente esta se podría presentar de nuevo. El estudio incluyó la evaluación del desempeño de dos modelos de generación de presión de poros (modelos Dobry-PWP y GMP-PWP) acoplados dentro del modelo constitutivo MRDF + u empleando el caso histórico de Port Island. El nuevo modelo acoplado representa razonablemente el comportamiento cíclico de los suelos observado en el caso histórico y se puede emplear de manera eficaz para realizar análisis 1D de "respuesta de sitio" basados en la aproximación de esfuerzos efectivos en la práctica regular de ingeniería.
Descriptores: Modelo constitutivo, licuación, degradación de módulo, análisis de respuesta de sitio.

Introduction

The impacts of porewater pressure (PWP) buildup, soil softening, and potential liquefaction on ground motions and the resulting response spectra are not yet well understood, and the actual ability to predict and/or compute these effects is not particularly precise. In sites susceptible to large PWP increase, most practitioners address those questions performing site response analysis as a two-step process:

1) Assessing liquefaction potential.
2) Performing a total-stress site response analysis.

The most widely used approach for estimating liquefaction potential at a site is applying the *cyclic stress method* proposed by Seed and his co-workers (e.g., Seed and Idriss (1971); Seed *et al*. (1985)) and more recently updated by Youd *et al*. (2001); Cetin *et al*. (2004); and Idriss and Boulanger (2008). The cyclic stress method addresses the triggering of liquefaction but does not provide an estimate of the corresponding surface accelerations if PWP increases and liquefaction is triggered. Site-specific response analysis using one-dimensional (1D) wave propagation is the most common approach for evaluating ground surface shaking due to wave pro-

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pagation in a soil column. The equivalent linear (EL) total stress analysis is the first approach to analyze problems that involves site response analysis using 1D wave propagation, but this approach (Schnabel *et al*., 1972) is not suitable for conditions where the soil response is highly nonlinear and the level of shaking is strong-conditions commonly associated with PWP buildup and liquefaction. However, nonlinear (NL) analysis codes are available to better represent soil nonlinear response. These codes often employ a total stress approach, ignoring PWP generation due to cyclic loading of the soil.

The effects of PWP generation in site response analysis can be accounted performing a NL effective stress analysis using a new quasi-coupled constitutive model proposed inhere. The quasi-coupled constitutive model is introduced in a simplified hyperbolic constitutive model termed MRDF+u (modulus reduction and damping curve fit using a reduction factor and including PWP generation, u) and implemented in the software DEEPSOIL (Hashash, 2011). The model is evaluated by studying the behavior of an actual soil site (i.e., case history), which was subjected to an earthquake. For analytical purposes, the input motion recorded at the site is propagated through the interpreted profiles using EL-, NL- total stress, and NL effective stress analyses as coded in DEEPSOIL (Hashash, 2011). The predicted response is compared to measured response (i.e., history case) to evaluate the effectiveness of the various site response procedures (i.e., EL-, NL-total stress, and NL effective stress) and the quasi-coupled constitutive stress-strain model.

The considered case history corresponds to Port Island and represents a specific problem with respect to the intensity of ground shaking, configuration of the liquefiable sand layers within the profile, dynamic properties of the different soil layers, observed surface manifestations, and availability of acceleration and PWP records during the seismic event. It is important to remark that the MRDF+u effective stress constitutive model can be used in any soil profile susceptible to liquefaction, and the Port Island case history was selected because the site is one of the most representative examples where liquefaction causes great damage.

Simplified quasi-coupled hyperbolic stress-strain constitutive model: MRDF+u

Based on the work by Hardin and Drnevich (1972); Matasovic (1993) proposed two degradation indices, which introduce excess PWP-induced softening into a simplified hyperbolic soil constitutive model: the modulus degradation index (δ_G) and stress degradation index (δ_τ). These indices reduce the shear stress mobilized during the loading-unloading process as a result of PWP increase (Matasovic, 1993), and are defined as

$$\delta_G = \sqrt{1 - r_u} \tag{1}$$

$$\delta_\tau = 1 - (r_u)^{\vartheta} \tag{2}$$

where r_u = excess PWP/σ'_{vo} or $\Delta u/\sigma'_{vo}$; and ϑ = dimensionless exponent generally equal to 3.5 (Matasovic, 1993) obtained of matching the stress-strain hysteresis loops over a wide range of r_u-values for Santa Monica Beach sand, Wildlife Site sands A and B, Heber Road point bar (PB) and channel fill (CF) sands. The advantage of the degradation indices is that they can use r_u values defined by any PWP generation model.

The modified hyperbolic model MRDF+D (modulus reduction and damping curve fit using a reduction factor) which simultaneously match modulus reduction and damping soil curves for nonlinear site response analysis was introduced by Phillips and Hashash (2009) and it was modified to incorporate the degradation indices defined above. Then, Moreno *et al.* (2010) proposed the following equations to compute shear stress values (τ) during loading and unloading - reloading, respectively, corresponding to a given strain.

Loading

$$\tau = \frac{G_0 \cdot \delta_G \cdot \gamma_c}{1 + \beta' \cdot \left(\frac{\delta_G}{\delta_\tau}\right)^t \cdot \left(\frac{\gamma_c}{\gamma_r}\right)^t} \tag{3}$$

Unloading – Reloading

$$\tau = F(\gamma_m) \cdot \left[\frac{G_0 \cdot \delta_G \cdot 2 \cdot \left(\frac{\gamma_c - \gamma_{rev}}{2}\right)}{1 + \beta' \cdot \left(\frac{\delta_G}{\delta_\tau}\right)^t \cdot \left(\frac{\gamma_c - \gamma_{rev}}{2 \cdot \gamma_r}\right)^t} - \frac{G_0 \cdot \delta_G (\gamma_c - \gamma_{rev})}{1 + \beta' \cdot \left(\frac{\delta_G}{\delta_\tau}\right)^t \cdot \left(\frac{\gamma_{max}}{\gamma_r}\right)^t} \right] + \frac{G_0 \cdot \delta_G (\gamma_c - \gamma_{rev})}{1 + \beta' \cdot \left(\frac{\delta_G}{\delta_\tau}\right)^t \cdot \left(\frac{\gamma_{max}}{\gamma_r}\right)^t} + \tau_{rev} \tag{4}$$

where

G_0 = initial shear modulus
δ_G = modulus degradation index
Υ_c = given shear strain
β' = dimensionless factor
$\delta\tau$ = stress degradation index

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Υ_r = reference shear strain
τ = dimensionless exponent
$F(\Upsilon_m)$= reduction factor
Υ_{rev} = reversal shear strain
Υ_{max} = maximum shear strain
τ_{rev} = reversal shear stress

Moreno (2012) evaluated four available PWP generation models and concluded that those proposed by Dobry *et al*. (1982, 1985) (termed the Dobry model), by Green *et al*. (2000); Green (2001) and Polito *et al*. (2008) (termed the GMP model) best predicted PWP generation for a large database of cyclic triaxial and cyclic simple shear tests. In addition, those tests were used to verify the validity of the quasi-coupled constitutive model with the evaluation of the stress-strain behavior incorporating the Dobry and GMP models. In general, the two models reasonably predict stress-strain and r_u-strain behavior in loose to dense specimens for shear strains (Υ_c) less than 5%. As expected, the models more poorly predict stress-strain and r_u-strain behavior when dilation becomes more pronounced (Υ_c > 5% and r_u > 0.65) and significant modulus degradation occurs.

These PWP generation models, described by Moreno (2012), were implemented in DEEPSOIL (Hashash, 2011) for use NL effective stress site response analysis quasi-coupled with the MRDF+u constitutive model. For the Dobry model, Moreno (2012) proposed the correlations presented in Table 1 to estimate the required model parameters, *p*, *F*, and *s* for stress- and strain-controlled loading.

In addition, Moreno (2012) confirmed the model parameter correlations reported by Polito *et al*. (2008) for the GMP model. Equation (5) presents the corresponding correlation.

$$\ln(PEC) = \begin{cases} \exp(0.0139 * D_r) - 1.021 & \text{if } FC < 35\% \\ -0.587 * FC^{0.312} + \exp(0.0139 * D_r) - 1.021 & \text{if } FC \geq 35\% \end{cases} \quad (5)$$

where

PEC = pseudoenergy capacity
D_r = relative density
FC = fine content

Port island case

Site description and soil profile characteristic

The Port Island Site, located close to the city of Kobe (Japan), was constructed in a land reclaimed from the sea by filling parts of Osaka Bay. Two major islands (Port Island and Rokko Island) were constructed by barging granular soil excavated from nearby mountains and dumping the soil into Osaka bay. Only a few localities were well-compacted during post-fill process to make the granular soil denser to prevent liquefaction. As a result, liquefaction was widespread and devastating in much of the filled area during the 1995 Kobe earthquake (Youd and Carter, 2003). The Port Island Site is 25 km northeast of the epicenter of the 1995 Hyogoken-Nanbu earthquake (Surface wave magnitude, M_s, 7.2) (Figure 1 (Iwasaki *et al*., 1996)).

The soil profile and instrumentation installed before 1995 at the Port Island Site are presented in Figure 2. The soil profile was defined on the basis of the standard penetration test (SPT), geophysical measurements, and laboratory tests (Ishihara *at al*., 1996). The analysis of the field data presents a soil profile that includes the uppermost 17.5 m thick layer of liquefiable sand fill, 10.5 m of silty clay and 9 m of layers of gravel and sand. Sand boils consisting of reclaimed fill were observed following the earthquake, indicating that this layer liquefied during shaking. The horizontal components of the acceleration records used as input motion in the present work were recorded at 32 m downhole by the SM3 accelerometer (Figure 3) and the horizontal ground surface component was recorded by the accelerometer SM1. Inagaki (Inagaki *et al*., 1996) constructed a 1/17-scale quay wall model to simulate the conditions at the Port Island Site. This model was tested on a shake table and PWP measurements were recorded. In this paper, the PWP measurements of the model-scale are compared to the PWP predictions from the site response analyses conducted based on new quasi-coupled constitutive model considering Dobry and GMP PWP models.

Table 1. Correlations for Dobry Model (Moreno, 2012)

Loading method	*P*	*F*	*s*
Stress- controlled	1.38 (all D_r)	0.16 (all D_r)	0.35 for D_r < -10%[1] 0.32 – 0.28D_r (-10% < D_r < 100%)[1]
Strain- controlled	1.0 (all D_r)	3.0 (D_r < 18%) 3.75 – 4.4D_r (20% < D_r < 80%) 0.16 (D_r > 80%)	2.0 (Dr < 20%) 2.88 – 4.18D_r (20% < D_r < 45%) 1.0 (D_r > 45%)

[1] For relative density (Dr), D_r <0 % applies to laboratory specimens

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The analytical soil profile used for modeling purposes is shown in Figure 4. The soil is represented by 18 layers with different stiffness and strength characteristics. The shear wave velocity profile is based on interpretation of standard penetration resistance, SPT profiles, and direct measurement of shear wave velocity. As shown in Table 2 (Profile input variables), the upper boundary of layer 3 coincides with the groundwater table (GWT). The bottom boundary condition is represented by a rigid halfspace boundary where the accelerometer SM3 was placed. The modulus reduction and damping curves were obtained using the curves measured by Ishihara *et al*. (1996) for sand fill, alluvial clay, and sand. The measured modulus reduction and damping curves were corrected to account for implied soil strength at large strain and adjusted using the MRDF-UIUC model (Hashash *et al*., 2010) and Table 3 shows the shear stress-shear strain input variables.

Dobry's PWP model parameters (*p*, *F*, and *s*) were assigned to submerged layers 3 to 18 using again the correlations developed by Moreno (2012). For the clayey material, the PWP parameters proposed by Matasovic (1993) were used in the analysis. In addition, practical values of the volumetric threshold shear strain, Υ_{vp}, equal to 0.02% for sand and 0.2% for clay were selected for the layers below the GWT following recommendations from Vucetic (1986) and Matasovic (1993). The Equation (5) was applied to calculate the corresponding PEC value for the GMP-PWP model and Table 3 shows the PWP input variables.

Analysis and results

Figure 5 shows the spectral acceleration, S_a, for North - South (NS) and East - West (EW) directions on surface. For these directions, the comparison of the actual and calculated spectra for EL, MRDF+D, and MRDF+u indicates that EL and MRDF+D constitutive models are overpredicting the response and MRDF+u (Dobry and GMP) reasonable describes the response at all periods. The period at the maximum spectral acceleration is captured by all constitutive models. MRDF+u constitutive model in comparison with Sivathasan *et al*. (2000) and Foerster *et al*. (2007) presents better prediction of spectral accelerations. Both researches performed effective stress analysis differentiating one to each other in the constitutive model used; Sivathasan *et al.* (2000) considered a hypo-plastic constitutive model and Foerster *et al*. (2007) considered an elasto-plastic constitutive model. The current analysis is closer in prediction, because the constitutive models used by Sivathasan *at al*. (2000) and Foerster *et al*. (2007) need sophisticated procedures to obtain the soil properties that the model uses. This sophistication could introduce some errors in the interpretation of the soil properties that could affect the final result. However, a careful basic selection of the soil parameters could lead to gain in accuracy of the final result of response spectrum using the MRDF+u effective stresses constitutive model. Again, the period at the maximum spectral acceleration, $S_{a\text{-}max}$ is not presenting any shifting between total stress/effective stress analy-

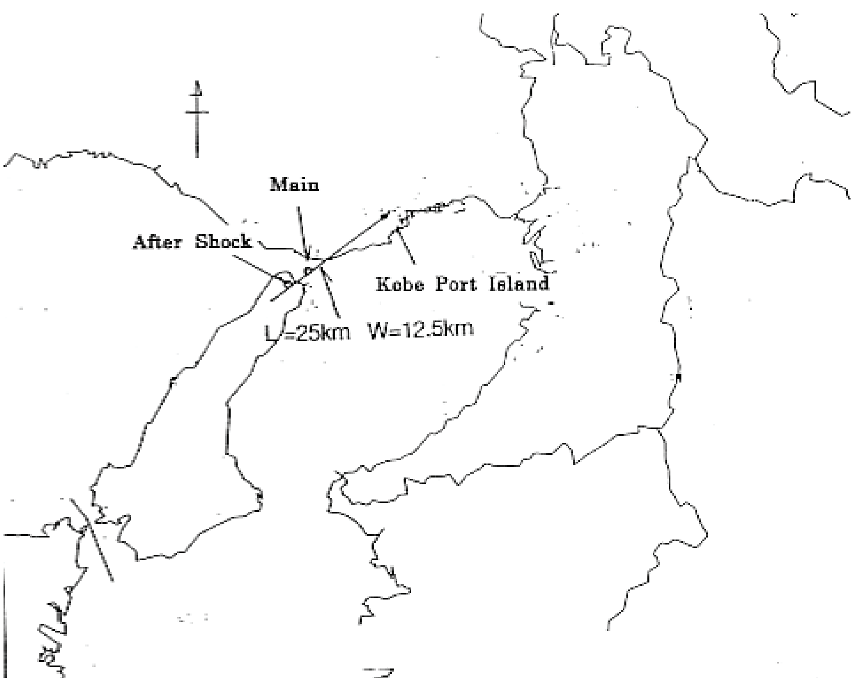

Figure 1. Port Island-Location Map and the epicenters of the major and aftershock earthquakes in the area (Iwasaki and Tai, 1996)

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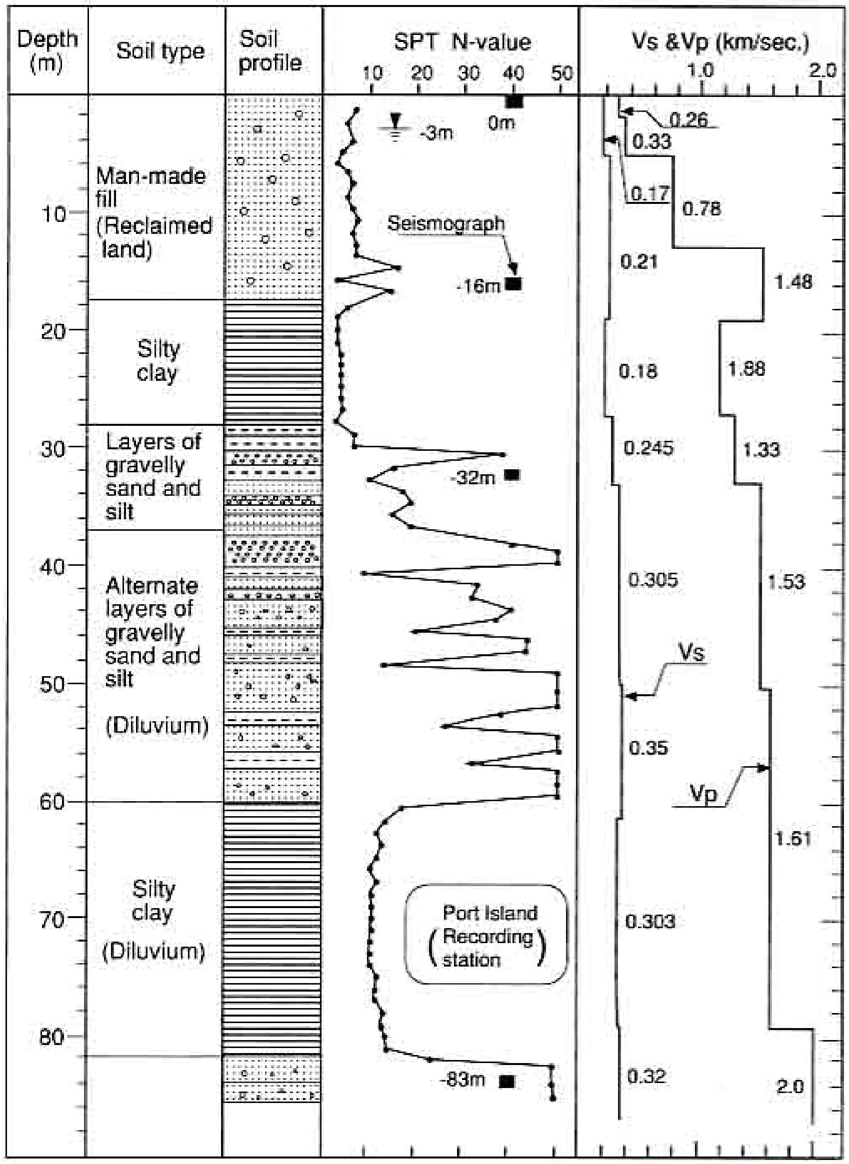

Figure 2. Port Island-Soil profile and location of the instrumentation (Ishihara *et al.,* 1996)

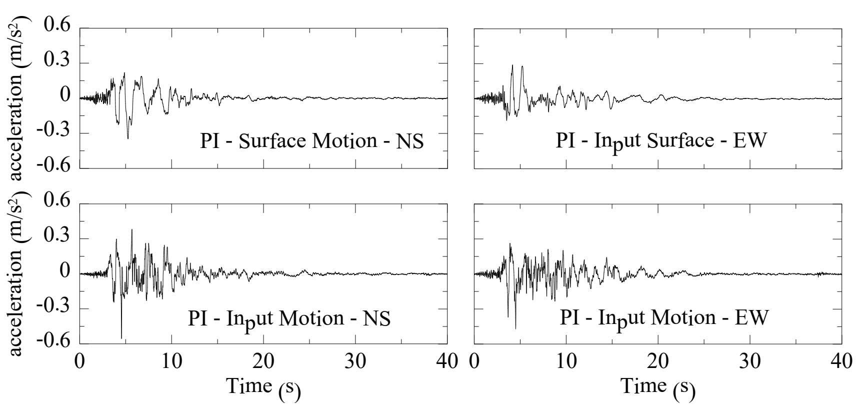

Figure 3. Input and Surface acceleration time history records during the 1995 Kobe Earthquake (Ishihara *et al.*, 1996)

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Table 2. Port Island Site-Analytical Soil Profile

Material type	Layer number	Thickness (m)	Unit weight (kN/m^3)	V_s (m/s)	Soil parameters
	1	1.5	17.7	170	
	2	1.5	17.7	170	GWT = 3 m
	3	2.0	17.7	170	
	4	2.0	17.7	210	
Sand fill	5	2.0	17.7	210	PI = 0%, ϕ' = 31°, K_o =0.485
	6	1.7	17.7	210	
	7	1.7	17.7	210	
	8	1.7	17.7	210	
	9	1.7	17.7	210	
	10	1.7	17.7	210	
	11	1.5	14.8	210	
	12	2.6	14.8	180	
Alluvial Clay	13	2.8	14.8	180	S_u (kPa) = 56-95, PI = 60%, ϕ' = 25, OCR = 1.7, K_o = 0.624
	14	2.6	14.8	180	
	15	1.0	14.8	245	
	16	1.5	18.2	245	
Sand	17	1.0	18.2	245	PI = 0%, ϕ' = 33°, K_o =0.455
	18	1.5	18.2	245	
			Rigid Base		

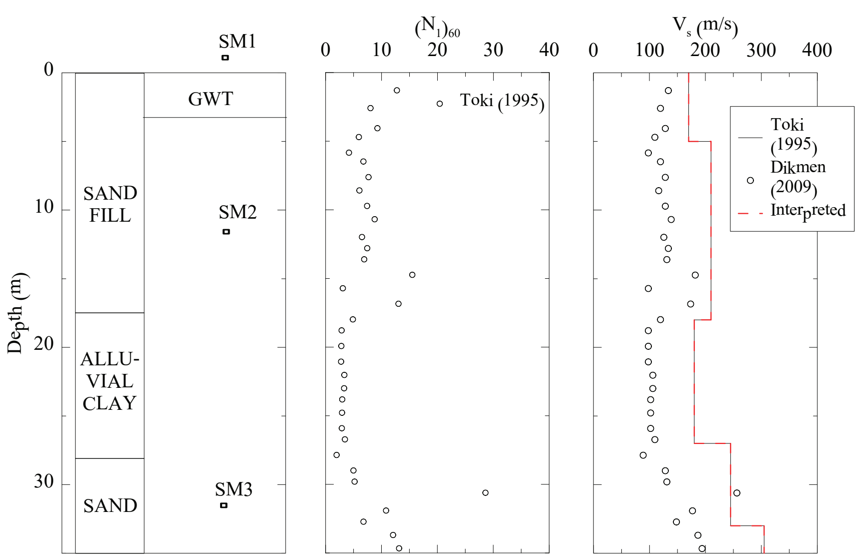

Figure 4. Soil, SPT and vs profiles at Port Island

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Table 3. Port Island Site – MRDF + u and PWP Model Parameters

		MRDF+u MODEL								DOBRY MODEL										GMP MODEL	
Material type	Layer No.	Thick. (m)	Damping Ratio (%)	Reference Strain (%)	Reference Stress (MPa)	β	t	f	p	F	s	ϒtvp (%)	υ	OCR	r	A	B	C	D	Dr (%)	FC (%)
Sand fill	1	1.5	0.086	0.034	0.18	1.29	1.08	1.2	1.05	2.8	1.8	0.025	3.8	1	-	-	-	-	-	36	20
	2	1.5	0.082	0.056	0.18	1.245	0.96	1.2	1.05	2.8	1.8	0.025	3.8	1	-	-	-	-	-	36	20
	3	2.0	0.073	0.074	0.18	1.485	0.915	1.2	1.05	2.8	1.8	0.025	3.8	1	-	-	-	-	-	36	20
	4	2.0	0.074	0.046	0.18	1.02	0.945	1.2	1.05	2.8	1.8	0.025	3.8	1	-	-	-	-	-	36	20
	5	2.0	0.073	0.033	0.18	0.735	0.915	1.2	1.05	2.8	1.8	0.025	3.8	1	-	-	-	-	-	36	20
	6	1.7	0.069	0.078	0.18	1.455	0.885	1.2	1.05	2.8	1.8	0.025	3.8	1	-	-	-	-	-	36	20
	7	1.7	0.069	0.077	0.18	1.365	0.885	1.2	1.05	2.8	1.8	0.025	3.8	1	-	-	-	-	-	36	20
	8	1.7	0.059	0.06	0.18	0.915	0.825	1.2	1.05	2.8	1.8	0.025	3.8	1	-	-	-	-	-	36	20
	9	1.7	0.06	0.12	0.18	1.455	0.825	1.2	1.05	2.8	1.8	0.025	3.8	1	-	-	-	-	-	36	20
	10	1.7	0.045	0.13	0.18	1.38	0.78	1.2	1.05	2.8	1.8	0.025	3.8	1	-	-	-	-	-	36	20
Alluvial clay	11	1.5	2.455	0.224	0.18	1.545	0.96	1.2	-	-	0.05	0.1	-	1.7	0.48	12.9	-26.3	15.3	-1.99	-	-
	12	2.6	2.455	0.224	0.18	1.545	0.96	1.2	-	-	0.05	0.1	-	1.7	0.48	12.9	-26.3	15.3	-1.99	-	-
	13	2.8	2.455	0.206	0.18	1.38	0.96	1.2	-	-	0.05	0.1	-	1.7	0.48	12.9	-26.3	15.3	-1.99	-	-
	14	2.6	2.456	0.243	0.18	1.545	0.96	1.2	-	-	0.05	0.1	-	1.7	0.48	12.9	-26.3	15.3	-1.99	-	-
	15	1.0	2.456	0.231	0.18	1.38	0.96	1.2	-	-	0.05	0.1	-	1.7	0.48	12.9	-26.3	15.3	-1.99	-	-
Sand	16	1.5	2.122	0.051	0.18	0.615	0.885	1.2	1.05	2.8	1.8	0.025	3.8	1	-	-	-	-	-	50	0
	17	1.0	2.122	0.051	0.18	0.615	0.885	1.2	1.05	2.8	1.8	0.025	3.8	1	-	-	-	-	-	50	0
	18	1.5	2.119	0.051	0.18	0.615	0.87	1.2	1.05	2.8	1.8	0.025	3.8	1	-	-	-	-	-	50	0
						Rigid base															

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sis and actual values of S_a, the only visible change is in the magnitude of the S_a. The decent response of the model could be attributed to a good interpretation of the soil properties.

As shown in Figure 6, the profiles of maximum shear strain (Υ_{max}), peak ground acceleration (PGA), shear stress ratio ($\tau/\sigma'_{\varpi o}$) and excess of PWP ratio (r_u) present that the major increment on PWP occurs at 10 m below ground level. In addition, the calculations capture the PWP buildup in most of the piezometers reported in Inagaki's scale model (1996), but liquefaction was not reached, even when field evidence indicate that liquefaction occurred at the site. The maximum shear strains calculated by all constitutive models are in the range of 3.5% to 6%. PGA profiles start in agreement with the actual (measured in field) input motion PGA and then all the models (total and effective stress analysis) decrease in a narrow band to the ground level capturing the PGA actual value. However, at 16 m below the ground surface, the PGA is not captured. The calculated PGA data show slight amplification of the motion between 16 m and 32 m that is expected in a relatively competent clay layer, but the predicted results indicate that this clay layer is relatively soft, with shear strains of about 3%, relatively high PWP and significant deamplification of the motion. Therefore, the predicted results are in agreement with the measured shear wave velocity which is V_s ~150 m/s and the corrected (i.e., by overburden pressure) standard penetration resistance is $(N_1)_{60}$ ~ 2 blows/feet suggesting a soft clay layer. The observed high PWP values in the clay layer are attributed to the lack of knowledge of the clay parameter for the PWP model available. The parameter value used was suggested by Matasovic (1993), who obtained the parameter using a clay at different preconsolidation pressure.

Figure 7 presents the complete set of time histories of input motion, surface motion, excess of PWP ratio (r_u), shear strain (Υ_c), time windows response spectra ratio (S_{aact}/S_{ainp}), and the response spectra (S_a) at the surface and input. The time windows analysis shows that there is some deamplification of the motion at short periods during the generation of PWP, while at the same time, there is moderate amplification of the motion at longer periods. Once the soil liquefies, there is little amplification or deamplification at any period, because the soil cannot transmit energy to the surface, which is in concordance with the observations presented by Moreno (2012). The important amplifications occurred later during shaking- after significant modulus reduction had already occurred- and amplification or deamplification occurred after $r_u > 0.9$.

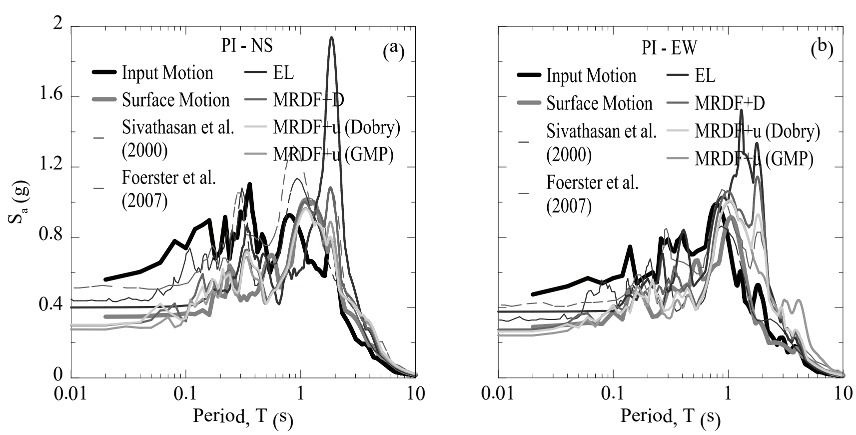

Figure 5. Response spectra for: a) NS motion and b) EW motion at Port Island

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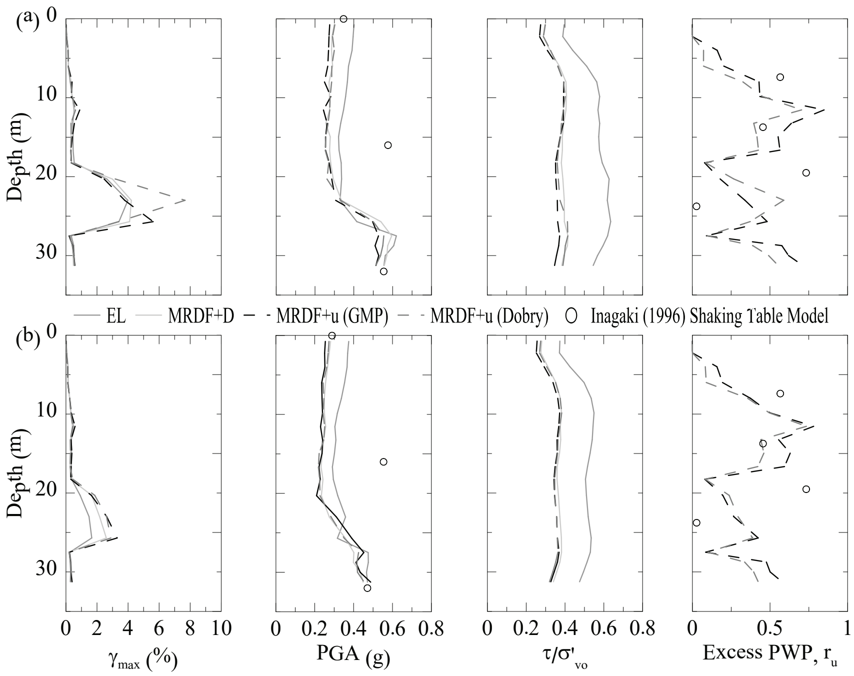

Figure 6. Υ_{max}, PGA, τ/σ'_{vo} and r_u profiles for: a) north-south motion and b) east-west motion at Port Island

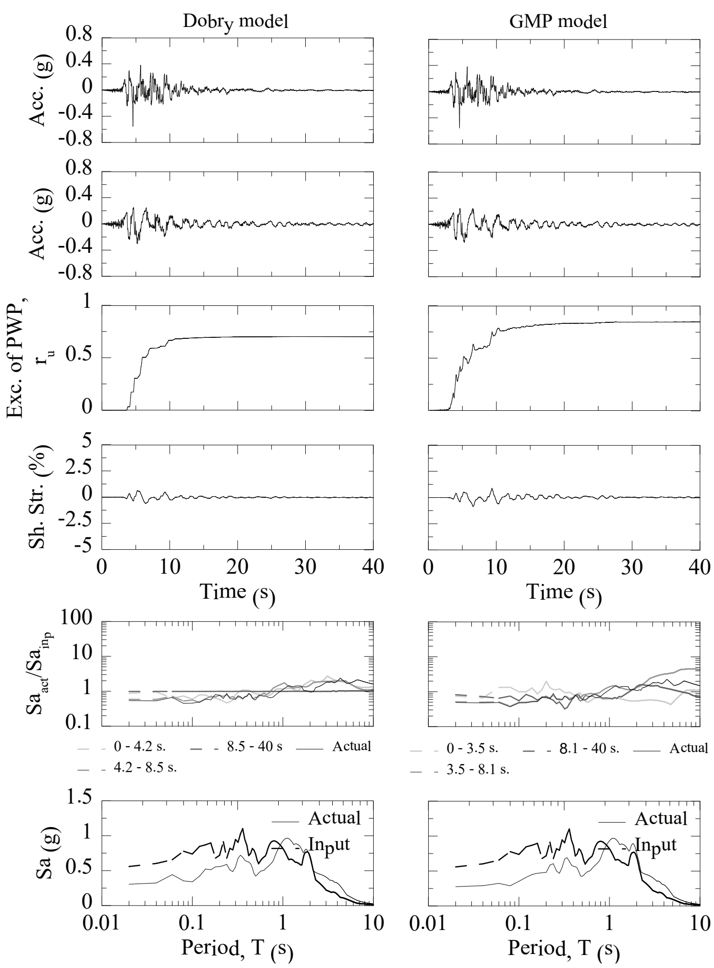

Figure 7. Time history of input motion, Time history of surface motion, Time history of PWP (Depth 11.6), Time history of shear strain (Depth 11.6), Time windows response spectra ratio, and response spectra at the input level and at the surface level for a) Dobry model and b) GMP model for PI – NS

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Conclusions

In this paper, a new quasi-coupled constitutive model, termed MRDF+u, is introduced. This constitutive model was used to conduct site response analysis considering a specific history case (i.e., Port Island, Japan) to evaluate its performance. This evaluation showed an appropriate description of the spectral acceleration, S_a, and that the whole spectrum of frequency content can be capture when the soil properties are fully measured.

The quasi-coupled MRDF+u constitutive model can capture the time history PWP behavior at different levels, which is a major success of this constitutive model compared with other similar constitutive models.

Given the ability of the modified hyperbolic constitutive MRDF+u model (i.e., Dobry- and GMP-PWP generation) to reasonably predict the response spectrum and the PWP generation in the history case analyzed, the MRDF+u model is recommended for application in engineering practice of site response analysis where the soil profile is susceptible to liquefaction (loose sands). Further research should focus on improving the model to account for softening the soil faster to get the liquefaction stage in.

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