

Seismic analysis of cantilever earth retaining walls embedded in dry sand by simplified approaches and finite element method

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Abstract: In engineering practice simplified methods are essential to the seismic design of embedded earth retaining walls, as fully-dynamic numerical analyses are costly, time-consuming and require specific expertise. Recently developed pseudostatic methods provide earth stresses and internal forces, even in those cases in which the strength of the soil surrounding the structure is not entirely mobilised. Semiempirical correlations or Newmark sliding block method provide an estimate of earthquake-induced permanent displacements. However, the use of these methods is hindered by uncertainties in the evaluation of a few input parameters, affecting the reliability of the methods. This study uses 1D site response analyses and 2D fully-dynamic finite element analyses to show that simplified methods can provide a reasonable estimate of the maximum bending moment and permanent displacements for stiff cantilever walls embedded in uniform sand, providing that a few input parameters are evaluated through semiempirical correlations and a simple 1D site response analysis.

Keywords: embedded retaining wall; seismic design; finite elements; pseudostatic methods; Newmark; permanent displacement

1 Introduction

The seismic design of embedded cantilever earth retaining walls requires the evaluation of internal forces and permanent displacements. This is often achieved through simplified approaches, e.g. pseudostatic methods and Newmark sliding block method, as more complex fully-dynamic numerical analyses are time-consuming and require specific expertise and costly software.

In pseudostatic methods earthquake inertial force is replaced with a force of constant amplitude and direction, defined by the seismic coefficient k_h . As with static design, limit equilibrium is imposed to compute the required wall embedment depth and the internal forces. Several solutions for the seismic active and passive earth stresses have been proposed. The most traditional is the formulation by Mononobe and Okabe, an extension of Müller-Breslau solution^[1–2], generally adopted to calculate the active earth stresses and for the passive stresses only if soil/wall friction is null or significantly low, as it assumes that failure occurs along a planar surface. Other solutions that consider soil-wall friction are available in the literature, e.g. the solution based on the lower-bound theorem proposed by Lancellotta^[3–4] and the method developed by Mylonakis et al.^[5]. Typically, in seismic

design with pseudostatic methods, it is assumed that the geotechnical system reaches critical conditions, i.e. a soil failure mechanism is mobilised during the design earthquake (e.g. EC8^[6]). Provided that the distribution of the horizontal earth stresses can be evaluated realistically, this approach allows for a sensible design of the wall if the maximum acceleration in the soil behind the wall ($a_{\max}$) exceeds the critical acceleration (a_c) of the geotechnical system ($a_c/a_{\max} < 1$). In contrast, it cannot estimate the stresses in the wall for earthquakes that are not sufficiently strong to mobilise a geotechnical failure mechanism ($a_c/a_{\max} > 1$). This may be the case of walls subjected to weak earthquakes (small $a_{\max}$) or requiring a large embedment depth (large a_c) to prevent failure by piping or uplift.

Embedded cantilever retaining structures accumulate permanent displacements during earthquakes, which must be compatible with strength/functionality of the retaining structure itself and of the near structures. Newmark sliding block method^[7] is widely adopted to evaluate such displacements. It assumes that a geotechnical system behaves like a rigid block on shaking ground. Once ground acceleration exceeds a threshold value (a_c), the frictional force between the block and the ground is fully mobilised and the block slides relatively to the

ground. In theory Newmark method can be applied to any geotechnical systems, providing that a_c and the input accelerogram can be defined, but it is most reliable when the system exhibits an almost rigid behaviour. Limit equilibrium pseudostatic methods can be used to compute a_c while the accelerogram must be representative of soil acceleration near the structure^[7–9]. For design purpose, semi-empirical correlations between the permanent displacement and the ratio a_c/a_{max} have been proposed based on the integration by Newmark method of many recorded accelerograms^[10–14]. However, in the case of embedded retaining structures Newmark method should be used with caution, as significant permanent displacements may develop even for soil accelerations well below the critical value; moreover, the definition of the input accelerogram is not straightforward, as the acceleration in the soil surrounding the structure is hardly uniform^[15–16].

A more rigorous assessment of the seismic performance of embedded retaining structures can be obtained by fully-dynamic numerical analyses. Previous studies have shown that finite difference and finite element analyses, in which soil non-linear dynamic behaviour is considered, can reproduce realistically soil dynamic response and the seismic behaviour of embedded cantilever walls in dry sand^[17–21]. However, compared with simplified approaches, they require the definition of a greater number of parameters, and are computationally onerous and time-consuming. Fully-dynamic analyses may take hours to complete and need to be run multiple times, as standards require the use of 3 to 7 accelerograms^[22].

To overcome the above limitations, modifications of traditional simplified approaches have been recently proposed. Modified pseudostatic methods were introduced to allow a more realistic evaluation of the internal forces by Conti et al.^[15] and Conte et al.^[23]. Moreover, semi-empirical correlations, based on fully-dynamic finite difference analyses, have been proposed for the direct evaluation of the permanent displacements or the estimate of a reduced critical acceleration for Newmark analyses^[24–25]. The use of these methods, however, is hindered by uncertainties in the definition of the parameters required for the analyses. The definition of the seismic coefficient k_h for the pseudostatic methods is not obvious, as the acceleration in the soil surrounding the structure is not uniform. The semiempirical correlations for the evaluation of the displacements are expressed as a function

of the maximum acceleration in the soil behind the structure, which is not known a priori.

Considering the above, this study investigates the ability and practicality of the above-mentioned recent simplified approaches to provide a realistic estimate of the internal forces and permanent displacements for stiff, elastic, infinitely strong, embedded cantilever walls in uniform sand. This is achieved through the analysis of the results from fully-dynamic 2D finite element simulations carried out with the code PLAXIS 2D (v. 2019). To consider stiffness non-linearity and hysteretic damping, the soil was modelled using the hardening soil model with small strain stiffness (HS small), validated against empirical damping/stiffness curves and 1D site response analyses (SRA).

The numerical results are used to derive semiempirical relationships for estimating k_h , needed to evaluate soil/structure contact stresses and the permanent displacements, as a function of soil acceleration at ground surface from simple equivalent linear 1D site response analyses, which can be easily carried out by civil engineers using freely available codes. The accuracy of the estimates is assessed against the results from the finite element simulations.

2 Simplified approaches

2.1 Pseudostatic methods

This study considers the simplified pseudostatic method suggested by Conti et al.^[15], which will be termed PSM1. This method assumes, based on numerical and experimental evidence, that embedded cantilever walls rotate approximately rigidly around a pivot point close to the toe^[15, 21, 23–26]. Above this point passive earth stresses (in front of the wall) and active earth stresses (behind the wall) are fully mobilised from ground surface to a certain depth (X) below dredge level. Below X , for seismic coefficients (k_h) less than the critical seismic coefficient (k_c), passive conditions are not fully mobilised in front of the wall and stresses are assumed to reduce linearly with increasing depth until active conditions are reached at the depth of the pivot point. Behind the wall active stresses are fully mobilised between X and the pivot point. Below the pivot, the distributions are inverted, with passive stresses behind the wall and active stresses in front of it. As a result, a discontinuity of the stress distribution is introduced at the depth of the pivot. In critical conditions ($k_h = k_c$), the passive earth stresses

are fully mobilised down to the pivot point.

Examples of the earth stress and bending moment distribution by PSM1 are shown in Fig.1, where the excavation height is $H = 4$ m, the embedment depth is $d = 4$ m, the soil is a dry cohesionless sand with friction angle $\phi = 35^\circ$ and unit weight $\gamma = 18$ kN/m³, and soil/wall friction angle is $\delta = 20^\circ$. The active and passive

earth stress coefficients are computed using the solution by Lancellotta^[3–4], with seismic coefficients (k_h) between 0 (static conditions) and k_c (critical conditions). The passive earth stresses are computed assuming $k_h = 0$, as below the excavation depth soil accelerations are relatively low and passive stresses have been found to follow approximately the static distribution^[15].

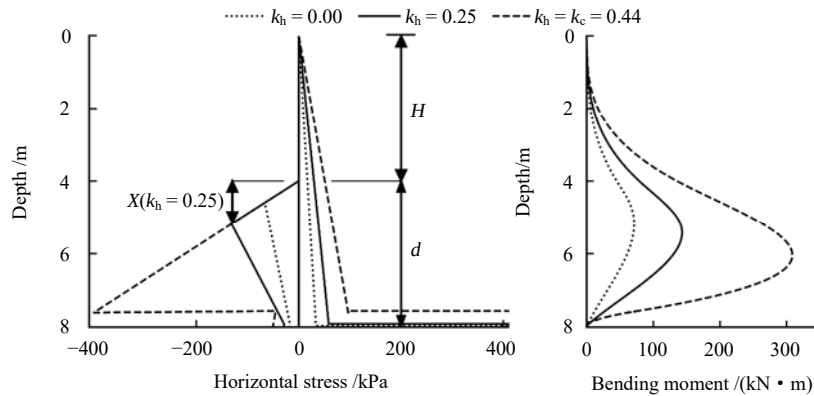


Fig.1 Distributions of the earth stresses behind and in front of the wall and bending moment from pseudostatic method PSM1^[15]

The main difficulty in the use of this method is estimating the input parameter k_h . For example, it could be estimated as $k_h = a_{\max 2D}/g$, where $a_{\max 2D}$ is the maximum acceleration behind the wall^[15]. However, this would require a fully-dynamic soil-structure interaction analyses to determine the (non-uniform) distribution of soil accelerations behind the wall and hence $a_{\max 2D}$. To overcome this limitation, this paper will illustrate a pseudo-empirical approach for estimating k_h without complex numerical analyses. The same approach can be potentially applied to other methods, such as the pseudostatic method by Conte et al.^[23], hereafter termed PSM2. The latter, provides simpler formulations, which are easier to implement, and its results are in close agreement with PSM1, providing that the same formulations are used in PSM1 and PSM2 to compute the active/passive earth stress coefficients (e.g. for the wall of Fig.1 it gave maximum bending moment and seismic coefficient in critical conditions of $M_{\max} = 292$ kN·m and $k_c = 0.43$ respectively). However, being expressed in terms of net earth stresses, it cannot clearly distinguish between the stresses in front and behind the wall.

2.2 Semi-empirical approaches for permanent displacements

Semi-empirical correlations can be used to estimate earthquake-induced permanent displacements exhibited by cantilever embedded walls in sand. For example, Callisto et al.^[24] and Conti et al.^[25] used the following

expressions to fit the results from their 2D numerical analyses and provide a direct estimate of wall permanent displacement:

$$u = C_1 \exp\left(-C_2 \frac{a_c}{a_{\max 2D}}\right) \quad (1)$$

$$\frac{u_t}{L} = C_3 \left(\frac{a_c}{a_{\max 2D}}\right)^{-C_4} \quad (2)$$

where u is the relative top-toe displacement; u_t is the displacement at the top of the wall; L is the total length of the wall; a_c is the critical acceleration by pseudostatic method; $a_{\max 2D}$ is the maximum soil acceleration behind the wall in proximity of ground surface by 2D numerical analysis, and C_1, C_2, C_3, C_4 are fitting parameters. To provide a tool that can be used for design without performing 2D numerical analyses, the above formulations can be expressed as a function of $a_c/a_{\max 1D}$, where $a_{\max 1D}$ is the maximum surface acceleration from a 1D SRA. However, this may increase significantly the scatter of the data, reducing the accuracy^[25].

Another possible approach involves using Newmark sliding block method even though, in its traditional form, the method strongly underestimates the permanent displacements of embedded walls, as sand exhibits significant permanent deformation even for accelerations well below a_c . To obtain larger, more realistic displacements, it may be used a substantially reduced critical acceleration

(a_{cN}). Previous research has shown that the ratio a_{cN}/a_c is smaller than 0.5 and may be expressed as a function of $a_{\max 2D}/a_c$ ^[25]. However, it is not clear the level of accuracy of this approach and whether a_{cN}/a_c can be expressed as a function of $a_{\max 1D}$, so that it can be evaluated through a simple 1D SRA.

3 Numerical model

3.1 Seismic input

Three accelerograms (TLM, ASS, AQV) were

considered for the fully-dynamic finite element (FE) analyses, which are publicly available from the engineering strong motion (ESM) database^[27]. These were “corrected” to eliminate the effects of the noise that is present inevitably in the recordings. Even with modern digital transducers, the velocity and displacement time series obtained through the integration of “uncorrected” accelerograms may show significant drifts that make them unphysical^[28–29]. The key features of the “corrected” accelerograms are summarised in Table 1.

Table 1 Main features of the “corrected” accelerograms

ID	Event name (ID) Magnitude M_W	Station code, Location Date, Time	PGA /g	PGV /(m • s ⁻¹)	PGD /m	I_A /(m • s ⁻¹)	T_{5-95} /s
TLM	Friuli (IT-1976-0002) 6.4	TLM1, Tolmezzo (Italy) 06/05/1976, 20:00:12	3.33–0.34	0.22	0.036	0.78	4.0
ASS	Umbria/Marche (IT-1997-0006) 6.0	ASS, Assisi (Italy) 26/09/1997, 09:40:24	1.57–0.16	0.09	0.011	0.27	4.3
AQV	L'Aquila (IT-2009-0009) 6.1	AQV, L'Aquila (Italy) 06/04/2009, 01:32:40	5.96–0.61	0.40	0.067	2.52	7.4

Note: PGA, PGV, PGD are the peak ground acceleration, velocity and displacement respectively; I_A is the Arias intensity^[30], $I_A = \frac{\pi}{2g} \int [a(t)]^2 dt$, where a is the acceleration and t is time; T_{5-95} is the time interval during which $I_A(t)$ increases from 5% to 95% of the final value.

The accelerograms were “corrected” following the scheme used for the ESM database^[31–33]. This consists of the following steps: (1) constant detrending of the accelerogram; (2) application of a cosine taper; (3) application of an acausal 2nd order Butterworth filter; (4) double integration of the acceleration to displacement; (5) linear detrending of the displacement; (6) double differentiation to obtain the “corrected” acceleration. The duration of the accelerograms was carefully reduced, after constant detrending, to minimise the computational cost of the FE simulations with minimal influence on the main features of the motions. The high-pass and low-pass frequencies for the Butterworth filter were set to 0.15 Hz and 15 Hz respectively. The length of the cosine taper was set to 5% of the record length. The “correction” procedure produced compatible accelerograms, i.e. the velocity and displacement time series obtained by double integration of the “corrected” accelerograms are physical with zero initial and final conditions. As an example, the “uncorrected” and “corrected” acceleration time series and correspondent Fourier amplitude spectra, and the velocity/displacement time series from integration of the “corrected” acceleration are shown in Fig.2 for TLM earthquake. For all motions, the “corrected” accelerograms and Fourier spectra are shown in Fig.3.

3.2 Model description

The FE numerical model is 2D with plane strain conditions and its geometry is represented in Fig.4. The excavation has a depth $H = 4$ m and width $B = 16$ m. Three values of embedment depth d are used (3 m, 4 m, 5 m). The soil profile consists of a layer of dry sand of thickness $L_y = 30$ m underlain by bedrock. The horizontal distance between each wall and the closest side boundary is $L_x = 90$ m.

Preliminary analyses indicated that the effect of L_x on the permanent relative top-toe displacement induced by the earthquake (u) and on the maximum bending moment ($M_{\max}$) in the wall is negligible for $L_x \geq 3L_y = 90$ m (Fig.5). This is consistent with a previous study, which shows that the total width of a 2D FE model should be 2 to 4 times greater than its depth to reproduce accurately the results of a 1D SRA^[34].

Initial stresses are generated via k_0 -procedure, with the earth stress coefficient at rest (k_0) from the widely-known Jacky formula. In the static phases (activation of walls and excavation) the side boundaries are horizontally fixed and the bottom boundary is fully fixed. In the dynamic calculation phase the “compliant base” and the “free-field” boundary conditions are assigned to bottom and side boundaries respectively. The “compliant

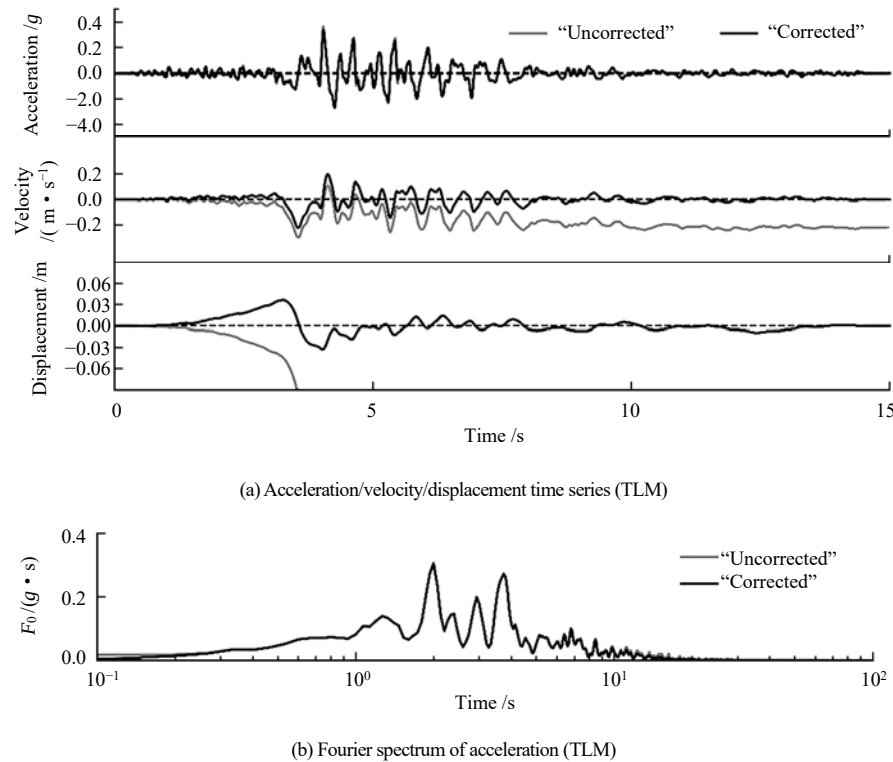


Fig. 2 “Uncorrected” and “corrected” accelerograms, velocity/displacement time series from integration of “corrected” accelerogram, and “uncorrected” and “corrected” Fourier amplitude spectra of acceleration (smoothened through a Savitzky-Golay filter) for TLM earthquake

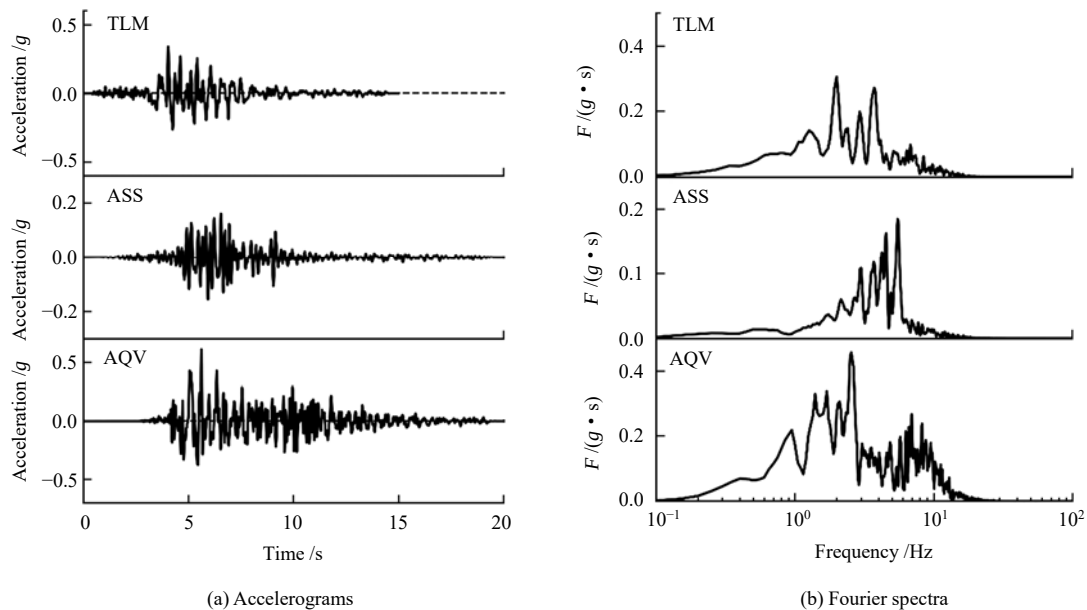


Fig. 3 “Corrected” acceleration time-series and Fourier spectra (smoothened through a Savitzky-Golay filter)

base” allows imposing the upward propagating motion (i.e. half of the outcrop motion) while absorbing the downward waves reflected by the above soil layers and structures; the “free field” boundaries impose free-field motion and absorb the waves reflected by the structures^[35–37]. Safety phases, in which the factor of safety (FoS) is calculated through the strength reduction method, are

added before and after the dynamic phase to evaluate the margin of safety against static geotechnical failure.

The sand was modelled using the hardening soil model with small strain stiffness (HS small) to take into account soil non-linear stiffness at small strain^[38]. The soil properties used, listed in Table 2, may be representative of a medium density sand^[39].

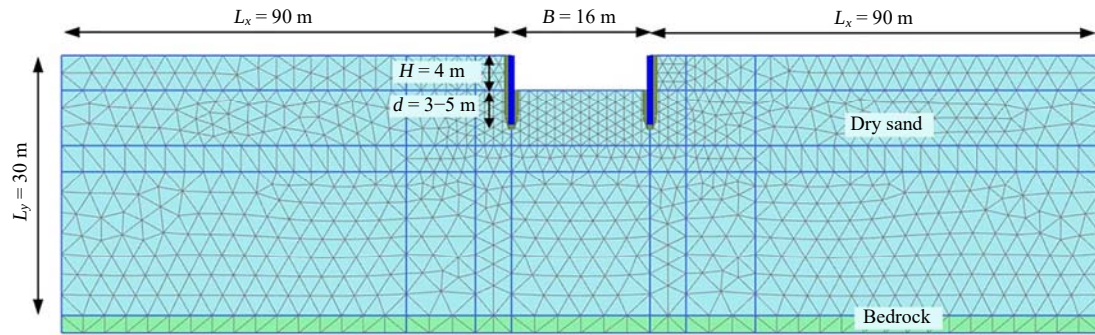
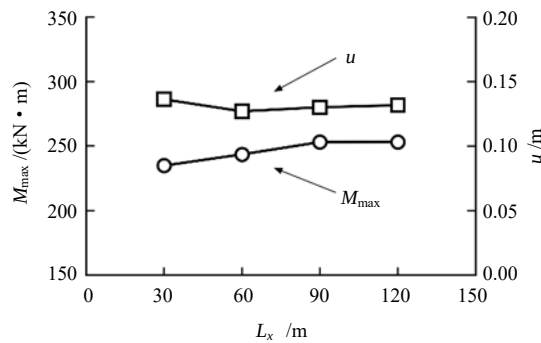


Fig. 4 Geometry of the problem

Fig. 5 Effect of the width of the FE model (the input motion is TLM earthquake with peak acceleration at the rock outcrop (a_g) of 0.34g; the embedment depth is $D = 4$ m)

The reference initial shear modulus G_0^{ref} and the reference shear strain γ_{07} define the modified Hardin and Drnevich relationship between shear modulus (G_s) and shear strain (γ_s), used by the HS small model^[38, 40]:

$$G_s / G_0 = \frac{1}{1 + 0.385 \gamma_s / \gamma_{07}} \quad (3)$$

where the initial shear modulus G_0 increases with the stress level according to the following relationship:

$$G_0 = G_0^{\text{ref}} \left(\frac{c' \cos \phi' - \sigma'_3 \sin \phi'}{c' \cos \phi' + p^{\text{ref}} \sin \phi'} \right)^m \quad (4)$$

in which c' and ϕ' are the effective cohesion and friction angle of the soil; σ'_3 is the minor principal stress; p^{ref} is a reference pressure (by default $p^{\text{ref}} = 100$ kPa); and m is a stress-level dependency parameter.

The tangent shear modulus (G_t) is obtained by derivation of the shear stress vs shear strain relationship ($\tau = G_s \gamma$) and is low-bound by the unloading-reloading shear

stiffness G_{ur} :

$$G_t = \frac{G_0}{(1 + 0.385 \gamma_s / \gamma_{07})^2} \geq G_{ur} \quad (5)$$

The shear strain at which $G_t = G_{ur}$ is termed $\gamma_{\text{cut-off}}$ and is given by the following expression^[41]:

$$\gamma_{\text{cut-off}} = \frac{1}{0.385} (\sqrt{G_0 / G_{ur}} - 1) \gamma_{07} \quad (6)$$

Soil hysteretic behaviour is based on Masing rules. The HS small model provides almost no damping at very small strain. Therefore, small additional viscous Rayleigh damping should be assigned^[18]. Rayleigh damping is frequency dependent and defined by the parameters α_R and β_R . These can be selected so that the damping ratio D equals a target value of damping ratio D^* at two target frequencies f_n and f_m . Between the target frequencies $D < D^*$, otherwise $D > D^*$. Preliminary analyses, not presented in this paper, showed that small variations in material damping (from 0.5% to 1%) have negligible effect on soil 1D response. For all analyses the same Rayleigh parameters were used, $\alpha_R = 6.43 \times 10^{-2}$ and $\beta_R = 3.06 \times 10^{-4}$, which provide a damping between ~0.5% and ~1% in the range of frequencies representative of the input strong motions, i.e. 0.5 Hz to 10 Hz.

The walls are modelled as elastic plate elements, i.e. 5-node line elements based on Mindlin's beam theory. The flexural stiffness of the wall sections (EI) was set to 2.7×10^5 kN · m² / m so that the walls behave almost rigidly during the earthquake^[24]. Flexural stiffnesses greater than 2.7×10^5 kN · m² / m are typical of high modulus steel walls (e.g. king-piles, pipe walls, combi walls),

Table 2 HS small soil parameters

$\gamma / (\text{kN} \cdot \text{m}^{-3})$	$G_{0, \text{ref}} / \text{kPa}$	γ_{07}	m	c' / kPa	$\phi' / (^\circ)$	R_f	α_R	β_R	$p_{\text{ref}} / \text{kPa}$	ν_{ur}	ψ	G_0 / G_{ur}	E_{ur} / E_{50}	E_{50} / E_{cod}
18	1.1×10^5	1.5×10^4	0.5	0	35	0.9	0.064 28	0.000 31	100	0.2	0	2.5	3	1

concrete pile walls and concrete diaphragm walls; smaller stiffnesses may be representative of sheet-pile walls or micro-pile walls^[42].

The behaviour at the interface between the walls and the soil is modelled through elasto-plastic interface elements. The relative wall/soil shear strength is expressed in terms of $R_i = \tan \delta / \tan \phi$, where δ is the relative soil/wall friction angle. In all analyses $R_i = 0.52$ or $\delta / \phi = 0.57$. The normal stiffness k_n and the shear stiffness k_s of the interfaces were set to 1 000 kPa/mm and 30 kPa/mm.

The soil elements consist of 15-node triangular elements, with 4th order shape functions and 12 stress points. The domain is subdivided into regions with different average element sizes, as clear from Fig.4. The average element size is ~ 2.5 m close to the bottom and lateral boundaries, and ~ 1 m close to walls. The distance between the mesh nodes l_n is smaller than ~ 0.6 m and meets the criterium suggested by Kuhlmeier and Lysmer^[43]:

$$l_n \leq \lambda / 8 = V_s / (8 f_s) \quad (7)$$

where λ , V_s and f_s are the shear wave length, velocity and frequency respectively; V_s is greater than ~ 100 m/s (Fig.6); while f_s is limited to ~ 15 Hz, as explained in Section 3.1.

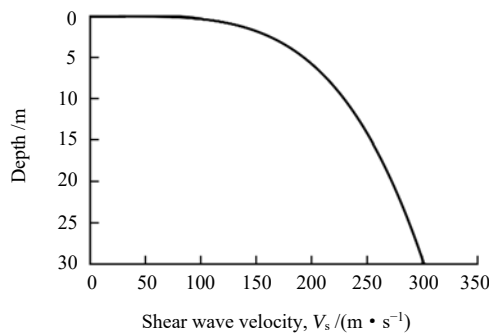


Fig. 6 Shear wave velocity profile - derived from Eq. (7)

The input motions correspond to the accelerograms described in Section 3.1. To investigate the effect of the intensity of the earthquakes, they were scaled without affecting the frequency content to obtain values of a_g (the peak acceleration at the rock outcrop) from $0.05g$ to $1.4g$. Extremely large values of a_g may not be representative of actual very strong earthquakes and were considered solely to fully mobilise the strength in the soil surrounding the wall and estimate the maximum internal forces that the structure could potentially experience. However, such high accelerations determined large wall displacements of over 1 m, which may be incompatible with the small strain assumption on which the FE model is based.

Moreover, previous research has shown that even non-linear soil models may not be able to reproduce realistically soil dynamic response for strong earthquakes, e.g. with a_g larger than $\sim 0.4g$ ^[44–46]. Therefore, the FE results for particularly strong earthquakes should be treated with caution.

The implicit Newmark's scheme was used for time-integration, with parameters $\alpha = 0.5$ and $\beta = 0.25$ for an unconditionally stable average acceleration scheme^[36]. The time step dt was set to 0.005 s to obtain an accurate solution within reasonable computational time^[34, 47]. This was verified through a few preliminary analyses which showed only a slight increase in wall displacement and maximum bending moment with reducing dt (Fig.7).

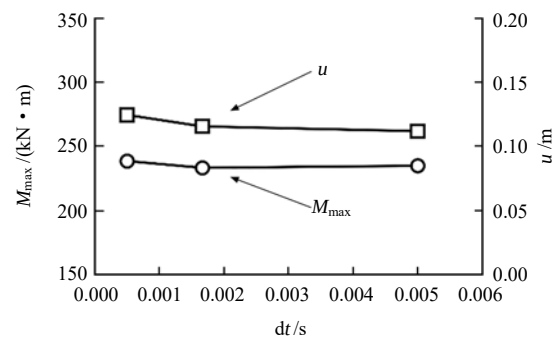


Fig. 7 Effect of the time-step (dt) on maximum bending moment in the wall (M_{max}) and final top-toe displacement (u) induced by TLM earthquake with $a_g = 0.34g$, $d = 4$ m, and a larger excavation width of $B = 32$ m

3.3 Damping and modulus curves

Cyclic simple shear tests under different vertical stresses (σ_v of 100 kPa, 200 kPa, 300 kPa) and shear strain amplitudes (γ_c from $10^{-3}\%$ to 1%) were simulated using PLAXIS Soil Test option to obtain the modulus reduction and damping curves for the HS small soil model used in the FE analyses. These were compared with the traditional, experimentally validated curves proposed by Idriss^[48].

The comparison between the different curves is shown in Fig.8(a), where D is the damping ratio, G_s is the secant shear modulus, G_0 is the shear modulus at small strain and γ_c is the shear strain amplitude. Below $\gamma_c \approx 0.03\%$ (corresponding to $\gamma_{cut-off}$ of Eq. (6)), the HS small soil shows increasing D with increasing γ_c . Between $\gamma_c \approx 0.03\%$ and $\gamma_c \approx 0.06\%$ – 0.1% , the hysteresis loops become narrower, owing to Eq. (5), and D reduces. For greater γ_c , D increases dramatically due to yielding. Once yielding

occurs, the behaviour of the soil becomes dependent on σ'_v . The ratio G_s/G_0 reduces with increasing γ_c . The negative gradient of the curve increases at $\gamma_c = \gamma_{\text{cut-off}}$ and reduces beyond $\gamma_c \approx 0.06\%$ – 0.1% due to yielding. The normalised hysteresis loops for the 3 key ranges of shear strain identified above (i.e. 0% to 0.03% , 0.03% to 0.06% – 0.1% , $>0.06\%$ – 0.1%) are shown in Fig.8(b), where τ_c is the maximum shear stress in a cycle.

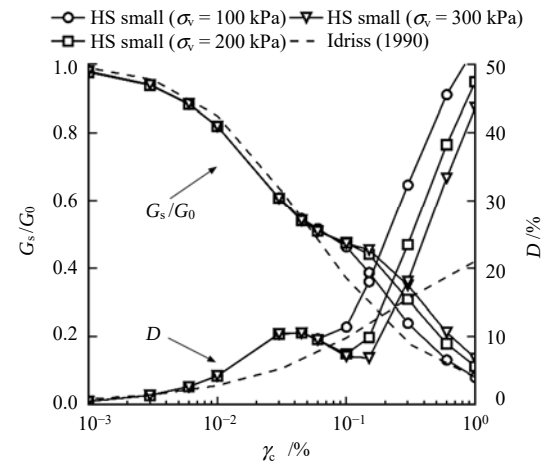
These results are consistent with those obtained by others by 3D FE analysis^[18] and suggest that the modulus reduction and damping offered by the HS small model is in good agreement with the empirical modulus/damping curves by Idriss for strains up to 0.06% – 0.2% . At larger strains, the HS small may show significantly larger damping, owing to yielding.

3.4 Free-field site response

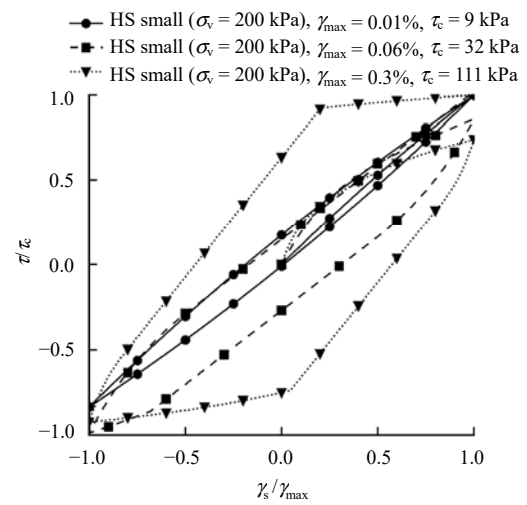
Site response in free-field conditions was investigated through non-linear (NL) fully-dynamic FE SRAs carried out in PLAXIS and simpler, but experimentally validated, equivalent-linear (EL) analyses in the frequency domain conducted using the code STRATA^[49].

The FE model consisted of a 1 m wide column of soil with laterally tied degrees of freedom to allow only vertical and shear strains^[35], a compliant base as bottom boundary condition, and soil layers and properties as described in Section 3.2. These analyses will be referred to as 1D NL, although obtained with a 2D model.

The 1D EL SRAs in STRATA used two sets of modulus/damping curves, those by Idriss^[48] (the dashed lines in Fig.8(a)) and those corresponding to the HS small soil (the solid lines with markers in Fig.8(a)). Previous research^[44–46] has found that 1D EL SRAs with modulus/damping curves by Darendeli^[50] (similar to those by Idriss used in this study) can reproduce soil dynamic response with acceptable accuracy for shear strains up to $\sim 0.05\%$ – 0.1% or a_g up to $\sim 0.2g$ – $0.4g$; for stronger earthquakes both EL and more complex NL models may be inaccurate. Therefore, the 1D EL SRAs in STRATA with Idriss soil model can be used as a reference to assess the ability of the HS small model to reproduce realistically soil response within a sufficiently wide range of ground accelerations. Also, as it will be shown, the 1D EL SRAs were used to define correlations with the results from the 2D FE analyses and allow the seismic analysis of the embedded cantilever wall through simplified methods.



(a) G_s/G_0 – γ_c curves



(b) τ/τ_c – $\gamma_s/\gamma_{\text{max}}$ curves

Fig. 8 Comparison between traditional damping / stiffness curves for EL analyses^[48] and the curves corresponding to the HS small soil model for different vertical stresses σ_v (a); normalised shear stress-shear strain loop for the HS small model (b)

The 1D NL analyses and 1D EL analyses compared well in terms of acceleration Fourier spectra and acceleration time series at ground surface for a_g up to $0.4g$, as shown in Fig.9 and Fig.10 for TLM earthquake.

A linear correlation between the maximum accelerations at surface from the EL analyses and the NL analyses was found, as shown in Fig.11, where $a_{\text{max}1\text{D}}^{\text{EL}}$ is obtained using Idriss curves, $a_{\text{max}1\text{D}}^{\text{EL*}}$ using the HS small curves, and $a_{\text{max}1\text{D}}^{\text{NL}}$ from the NL analyses in PLAXIS. As expected, the correlation of Fig.11(b) between $a_{\text{max}1\text{D}}^{\text{EL*}}$ and $a_{\text{max}1\text{D}}^{\text{NL}}$ is characterised by smaller scatter. In general, increasing scatter can be observed for $a_g > 0.6g$ (the grey markers in the figure), owing to significant yielding in the NL model. As also observed in a previous study^[51], the EL analyses tend to overestimate the peak acceleration at

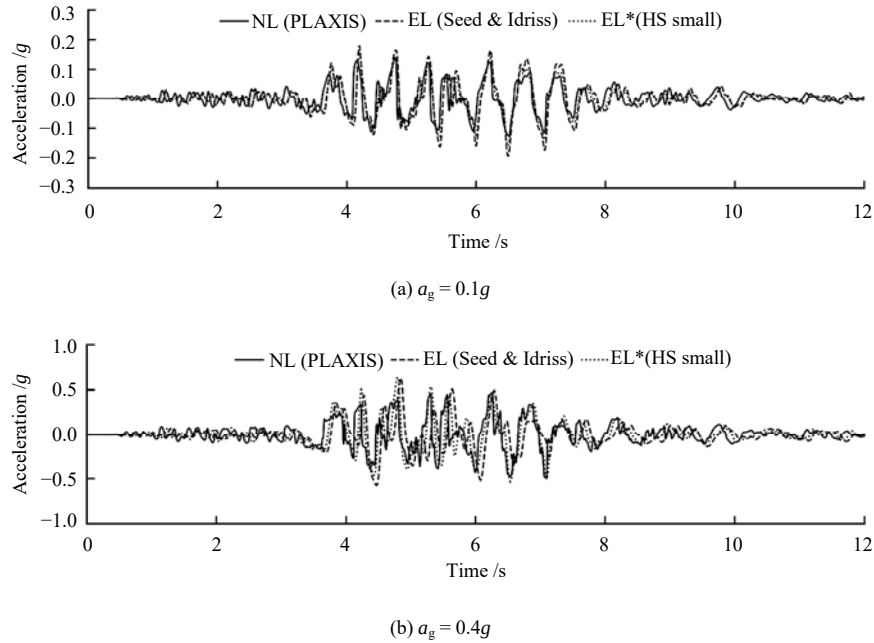


Fig. 9 Acceleration time series at surface from 1D SRAs; input motion is TLM earthquake; NL analyses carried out in Plaxis; equivalent linear analyses EL and EL* in STRATA with modulus/damping curves from Fig.8

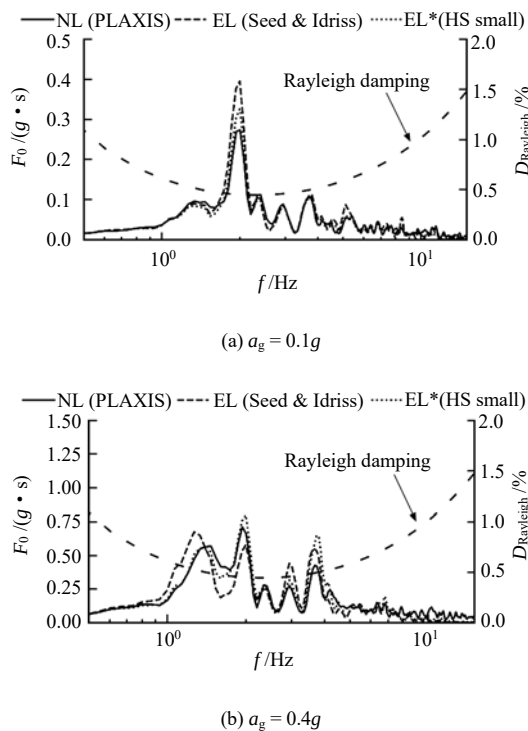


Fig. 10 Fourier amplitude spectra at surface from 1D SRAs; input motion is TLM earthquake; the EL and EL* were carried out in STRATA, with modulus/damping curves from Fig.8; the NL analyses were carried out in PLAXIS

surface, in this case by ~15%–30%, even for modulus/damping curves artificially selected to match the behaviour of HS small model used in the NL analyses. Overall, assuming that the EL analyses with Idriss curves can reproduce realistically site response for a_g up to ~0.4g,

the HS small model should also be able to simulate it with good accuracy. For stronger earthquakes, also considered in this study, results should be considered carefully.

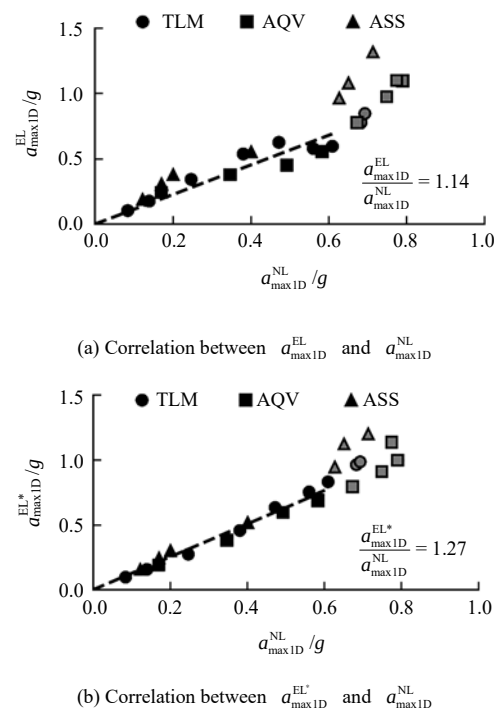


Fig. 11 Maximum soil acceleration at surface from 1D EL SRA and 1D NL SRA with $0.05 \leq a_g \leq 1.4$

3.5 Effect of excavation on site response

The effect of the excavation on site response was

investigated by comparing the results from the 1D NL model (described in Section 3.4) and the 2D FE model (described in Section 3.2) in terms of maximum acceleration at ground surface, $a_{\max 1D}^{NL}$ and $a_{\max 2D}$, with the latter evaluated at surface, about 1 m behind the wall.

As shown in Fig.12, the excavation amplified soil response by a factor of ~ 1.3 for a_g up to $\sim 0.6g$ (black markers). For stronger earthquakes (grey markers) the excavation seems to cause significantly larger amplification of soil accelerations. These results are in line with previous studies, which found that in proximity of similar excavations soil response may be amplified by up to ~ 2 times^[25, 52]. As already observed in the previous section, for strong earthquakes the FE model may be less reliable and the small strain assumption may not be accurate. A more in-depth study is underway to investigate the applicability of the FEM with HS small model for high values of a_g .

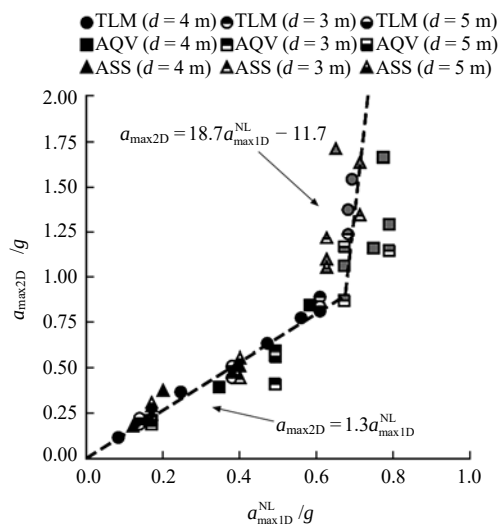


Fig. 12 Maximum acceleration at surface from 1D ($a_{\max 1D}^{NL}$) and 2D ($a_{\max 2D}$) finite element analyses; the 2D model includes excavation and structure; grey markers are associated with $a_g > 0.6g$

4 Results

Hereafter the results from the FE analyses of the embedded wall are presented and the potential of the simplified approaches described in Section 2.1 to predict the seismic behaviour of the structure is discussed. Static conditions (before the earthquake) are also briefly discussed for completeness. With regards to the evaluation of the earth stresses and internal forces in the wall, only pseudostatic method PSM1 was considered, as it provides the earth stresses both behind and in front of the wall.

As for earthquake-induced wall permanent displacements, the potential of semiempirical correlations and Newmark sliding block method to predict them was assessed.

4.1 Static conditions

The static distributions of the horizontal earth stresses and bending moment by finite element method (FEM) and pseudostatic method PSM1 are shown in Fig.13 for walls with different embedment depths (d). Wall displacements from the FE analyses are also shown. The stresses from PSM1 compare well with those from FEM. Above the excavation depth, the soil adjacent to the wall is close to active conditions, although they are not fully reached. At greater depth, the stresses from PSM1 are less accurate but reasonably close to those from the numerical analyses. In this region passive and active conditions are not reached, owing to the relatively small displacements, and PSM1 underestimates the stresses behind the wall.

PSM1 underestimates the maximum bending moment ($M_{\max}$) by $\sim 10\%$ for $d = 3$ m, and by $\sim 16\%$ for $d = 4$ m and $d = 5$ m. This is explained by the active earth stresses, assumed in PSM1, underestimating slightly the earth stresses behind the wall, even above the excavation depth. A few preliminary analyses, not reported in this paper, showed that for very flexible walls ($EI = 2.3 \times 10^4$ kN·m²) the correspondence between PSM1 and FEM improved significantly, as greater displacements mobilised almost entirely the active stresses above the excavation depth.

Wall displacements, represented on the right-hand side of Fig.13, reduced with d . The shortest wall ($d = 3$ m) experienced an almost rigid rotation about a point close to its toe. For longer walls with $d = 4$ m and $d = 5$ m the contribution of wall flexibility to the overall displacement became appreciable.

Deeper walls exhibited smaller displacement but greater bending moment. This can be only partly reproduced by PSM1, which accounts for the greater length of the wall but does not consider that the longer wall, exhibit smaller displacements and hence are subject to larger earth stresses.

The FoS before the earthquake for the walls with $d = 3$ m, $d = 4$ m and $d = 5$ m were 1.24, 1.54 and 1.82 respectively. Therefore, the shortest wall is representative of a structure close to the minimum safety margin against geotechnical failure requested by EC8, i.e. FoS = 1.25.

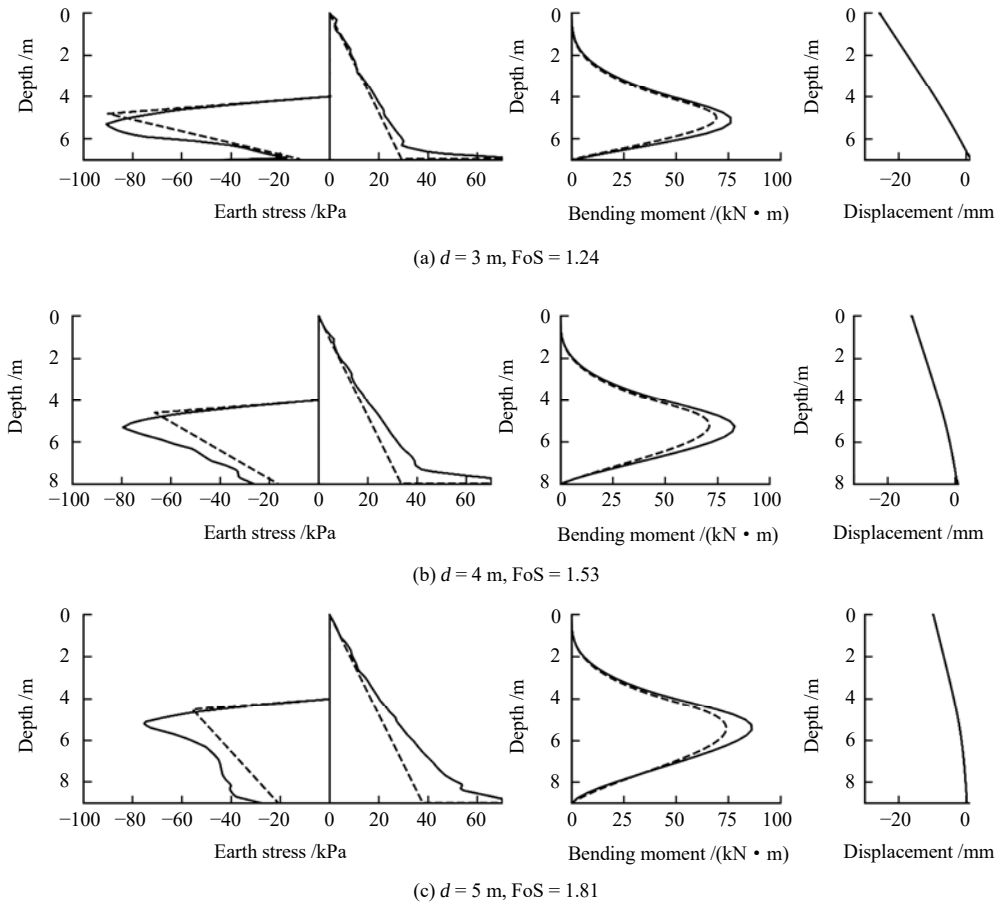


Fig. 13 Static distributions of horizontal earth stresses, bending moment, horizontal displacements; FEM (solid line) vs. PSM1 (dashed line) for walls with different embedment depths d

4.2 Earth stresses and bending moments during the earthquake

This section compares the behaviour of the embedded retaining walls by FEM and PSM1 to assess the potential of the latter to predict the maximum bending moment in the wall ($M_{\max}$). All results are referred to the time instant at which the bending moment in the wall is maximum.

Fig.14 shows earth stresses and bending moment by FEM for AQV earthquake with two different intensities ($a_g = 0.1g$ and $a_g = 0.6g$), and provides a comparison with the lower-bound and upper-bound distributions by PSM1, i.e. those with $k_h = 0$ and $k_h = k_c$ respectively, while passive earth stresses are always calculated assuming $k_h = 0$. In front of the wall the earth stresses follow approximately the static passive distribution down to a certain depth, below which they gradually reduce. The stronger the earthquake, the greater is the proportion of the passive force that is mobilised in front of the wall and hence, the peak stress. This distribution is qualitatively consistent with that assumed by PSM1, where X and

hence the peak stresses in front of the wall increase with increasing k_h (Fig.1). The maximum active earth stress behind the wall increases with increasing a_g but remains abundantly smaller than the maximum passive earth stress which, as already explained, also increases with a_g . Behind the wall the stresses are distributed approximately linearly only for relatively weak earthquakes ($a_g = 0.1g$). Strong earthquakes ($a_g = 0.6g$) are characterised by a strongly non-linear stress distribution on the retained side of the wall, with significantly larger stresses above the excavation depth. Therefore, the assumption of a linear distribution of the stresses behind the wall, on which the pseudostatic methods are based, does not agree well with the FE analyses for strong earthquakes. For a given earthquake, $M_{\max}$ increases strongly with increasing d and, if the earthquake is strong enough, may exceed abundantly the critical bending moment M_c provided by PSM1. This is the case of $d = 3$ m or $d = 4$ m and $a_g = 0.6g$ (Fig.14(a), (b)). For $d = 5$ m and $a_g = 0.6g$, $M_{\max}$ is larger but does not exceed M_c , which also increases with d (Fig.14(c)).

The smaller earth stresses below dredge level may

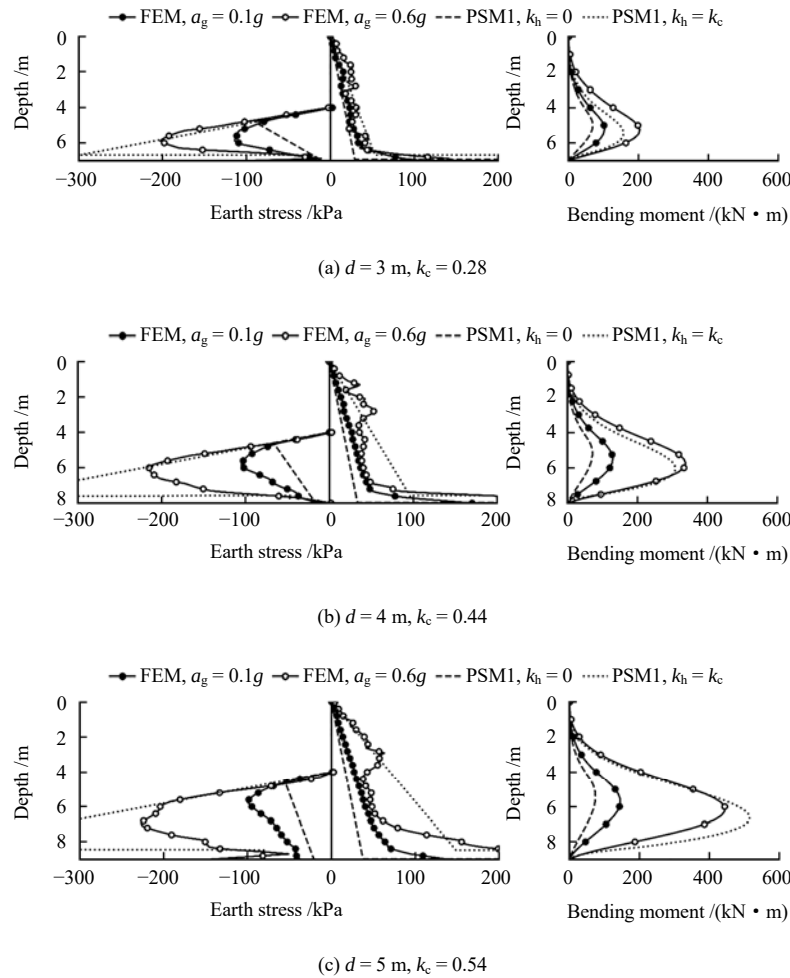


Fig. 14 Distribution of horizontal earth stresses and bending moment at the time instant of maximum bending moment

be explained by the smaller accelerations in this region, as shown in Fig.15. Therefore, a possible improvement of PSM1 may be obtained by adopting, below the excavation depth, a smaller seismic coefficient for the active stresses (and not only for the passive stresses, which are already calculated assuming $k_h = 0$).

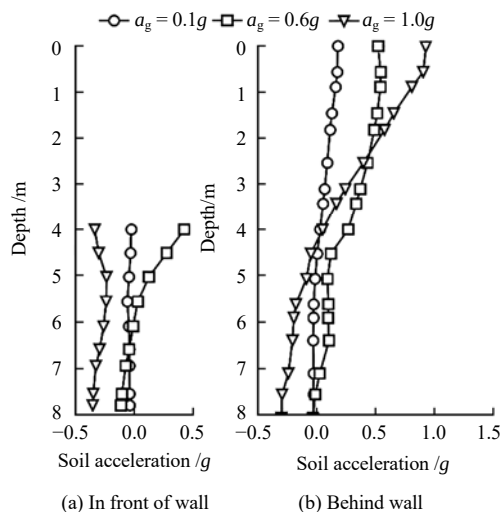


Fig. 15 Soil acceleration close the wall (in front and behind) for AQV earthquake with $0.1 \leq a_g/g \leq 1.0$ and $d = 4 \text{ m}$

Nevertheless, PSM1 can still provide a qualitative description of the seismic behaviour of the structure and an estimate of the critical acceleration (a_c) representative of the seismic strength of the geotechnical system.

As shown in Fig.16, independently of the seismic input (TLM, AQV, ASS), for strong earthquakes $M_{\max}$ by FEM tended to ~ 1.6 times M_c , the theoretical upper-bound value of bending moment by PSM1.

Although PSM1 underestimates $M_{\max}$ for very intense earthquakes, it can still be used for weak to intense earthquakes, providing that the seismic coefficient can be defined. For this purpose, the equivalent seismic coefficients (k_h^{eq}) to be used with PSM1 to match $M_{\max}$ from the FE analyses were back-calculated. The results are shown in Fig.17(a), where k_h^{eq} was set equal to k_c for the analyses in which $M_{\max} \geq M_c$. It was found that the ratio k_h^{eq}/k_c can be expressed as a function of $a_{\max 2D}/a_c$, where $a_{\max 2D}$ is the maximum soil acceleration at surface behind the wall, irrespective of the embedment depth d and the seismic input. To allow estimating k_h^{eq} without the need for a complex fully-dynamic analysis of soil/structure

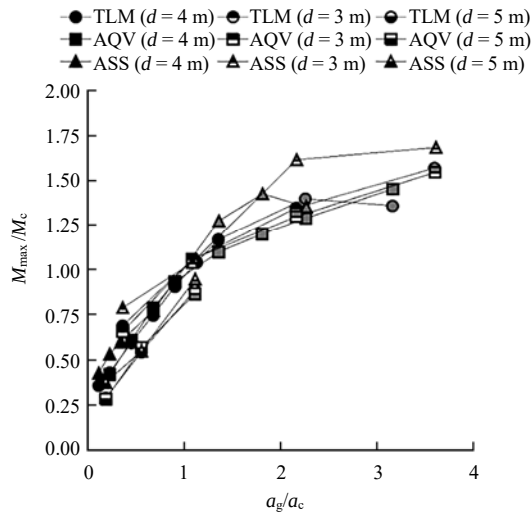
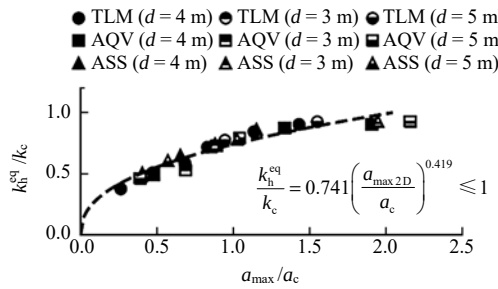
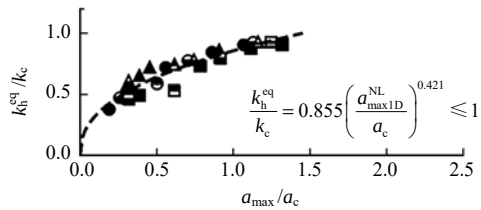


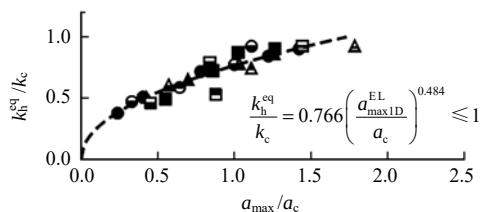
Fig. 16 Increase in bending moment with increasing earthquake intensity; $M_{\max}$ is the maximum bending moment in the wall by FEM; a_g is ground acceleration at the rock outcrop; a_c is the critical acceleration by PSM1; M_c is the corresponding bending moment; grey markers indicate $a_g > 0.6g$



(a) $a_{\max} = a_{\max2D}$



(b) $a_{\max} = a_{\max1D}^{NL}$

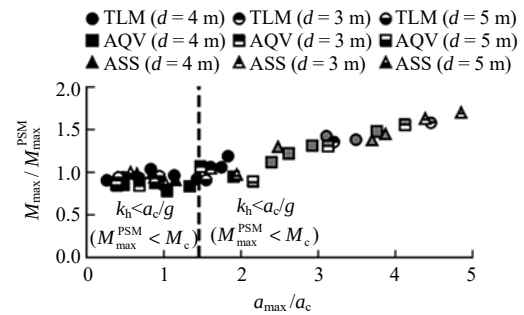


(c) $a_{\max} = a_{\max1D}^{EL}$

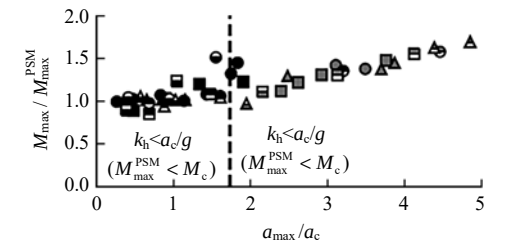
Fig. 17 Equivalent seismic coefficient (k_h^{eq}) to be used with PSM1 to match the maximum bending moment $M_{\max}$ by FEM; k_c and a_c are the critical seismic coefficient and acceleration by PSM1; $a_{\max}$ is the maximum soil acceleration at surface; grey markers for $a_g > 0.6g$

interaction, $a_{\max2D}$ was replaced with $a_{\max1D}^{NL}$ and $a_{\max1D}^{EL}$, i.e. the maximum acceleration at surface from 1D non-linear (NL) and equivalent-linear (EL) site response analyses carried out as described in Section 3.4. The results, shown in Fig.17(b) and Fig.17(c), show a clear correlation between k_h^{eq}/k_c and $a_{\max1D}/a_c$.

For all analyses, the correlations from Fig.17(b) and Fig.17(c) were used to evaluate k_h^{eq} and hence PSM1 was used to compute the maximum bending moment in the wall ($M_{\max}^{PSM}$). If $M_{\max}^{PSM}$ is computed using k_h^{eq} from Fig.17(b) (i.e. from a 1D NL analysis), it deviates from $M_{\max}$ by less than $\pm 20\%$, providing that $a_{\max1D}^{NL}$ does not exceed $\sim 2a_c$ (Fig.18(a)). Beyond $a_{\max1D}^{NL} \approx 2a_c$, $M_{\max}$ exceeds M_c and hence PSM1 increasingly underestimates $M_{\max}$. If $M_{\max}^{PSM}$ is computed using k_h^{eq} from Fig.17(c) (i.e. from a 1D EL SRA), it deviates from $M_{\max}$ by less than $\pm 20\%$ only for $a_{\max1D}^{EL}$ smaller than $\sim 1.5a_c$ (Fig.18(b)).



(a) $a_{\max} = a_{\max1D}^{NL}$ and k_h is given by the equation of Fig.17(b)



(b) $a_{\max} = a_{\max1D}^{EL}$ and k_h is given by the equation of Fig.17(c)

Fig. 18 Ability of PSM1 to predict the maximum bending moment in the wall; $M_{\max}^{PSM}$ and $M_{\max}$ are the maximum bending moment by PSM1 and FEM respectively; grey markers are associated with $a_g > 0.6g$

4.3 Wall displacements

The FE analyses showed that embedded cantilever retaining walls tend to rotate about a point close to their toe. As an example, Fig.19 shows the distribution of the horizontal displacements at two different time instants for AQV earthquake with $a_g = 0.6g$ and different embedment depths d . The walls show an approximately rigid

rotation both when the bending moment is maximum (Fig.19(a)) and at the end of the earthquake (Fig.19(b)).

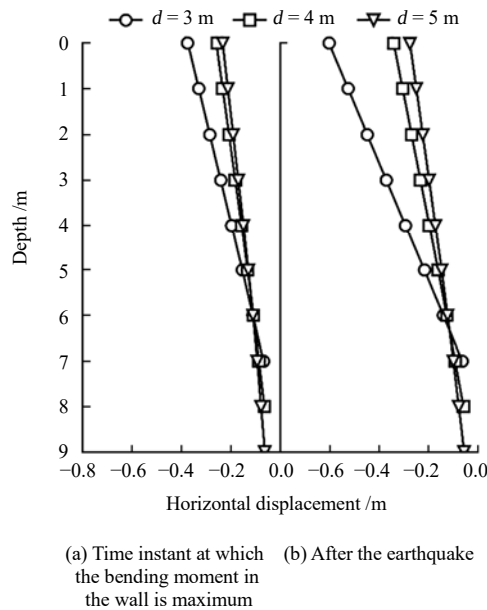


Fig. 19 Distribution of the horizontal displacements for AQV earthquake with $a_g = 0.6g$ and different embedment depths d

Following the approach described in Section 2.2, it was sought a correlation between the normalised displacement (u/L) and the ratio $a_c/a_{\max 2D}$, where u is the permanent relative top-toe displacement induced by the earthquake and $L = H + d$ is the total length of the wall. Only values of $u/L < 5\%$ were considered. It was found that u/L and $a_c/a_{\max 2D}$ can be related through a power equation, as shown in Fig.20(a). Similar trends can be obtained if the maximum soil acceleration at surface is computed by simpler 1D NL or EL site response analyses, as shown in Fig.20(b) and Fig.20(c).

For each analysis, wall top-toe displacement u^* was estimated through the fitting equations of Fig.20(b) and Fig.20(c). The ratio between the displacement from the 2D FE analyses and the estimated displacement (u/u^*) is shown in Fig.21 as a function of the ratio u/L . For u/L up to 4%, the fitting equations of Fig.20(b) and Fig.20(c) provide a realistic estimate of u , using only two input parameters, i.e. a_c (by PSM1) and $a_{\max 1D}^{NL}$ or $a_{\max 1D}^{EL}$ (by 1D SRA). For damage limitation, a tolerable value of u/L may be approximately 3%^[16]. Therefore, the limit of $u/L = 4\%$ should be sufficiently large for serviceability limit states (SLS).

The potential of Newmark sliding block method with reduced critical acceleration (a_{cN}) to estimate u was also investigated. For all FE analyses, a_{cN} was back-calculated

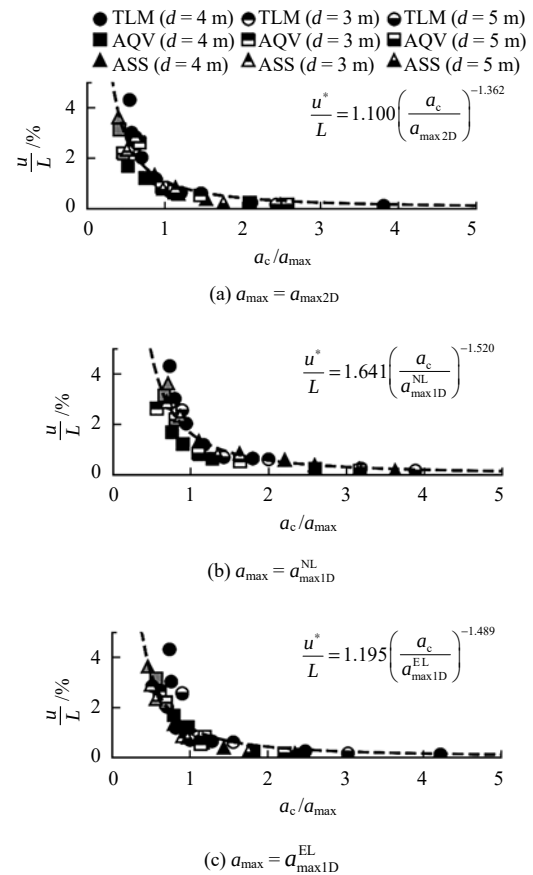


Fig. 20 Correlations between the permanent top-toe displacement induced by the earthquake (u) and maximum acceleration at ground surface ($a_{\max}$); u is normalised to the total length of the wall (L); a_c is the critical acceleration by PSM1; grey markers indicate $a_g > 0.6g$

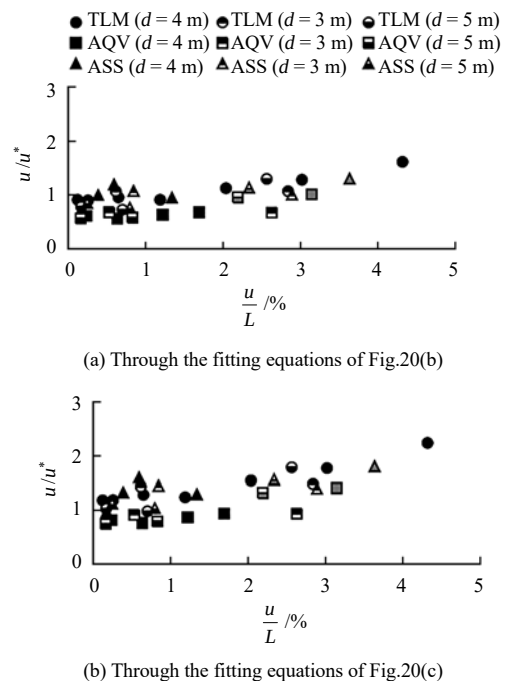
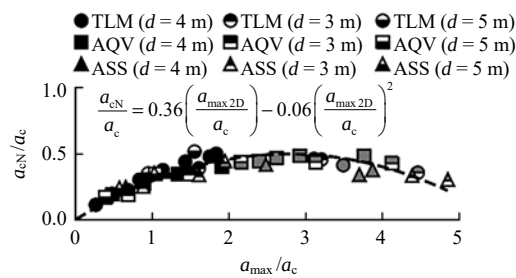
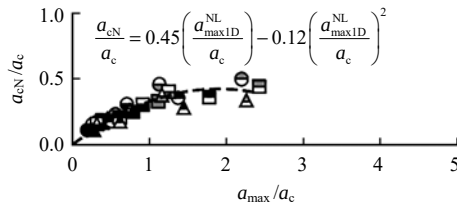


Fig. 21 Accuracy of the semi-empirical correlations of Fig.20(b) and Fig.20(c) to predict wall permanent top-toe displacement induced by the earthquake; u/u^* is the ratio of the displacement by FEM to estimated displacement; L is the total length of the wall; grey markers for $a_g > 0.6g$

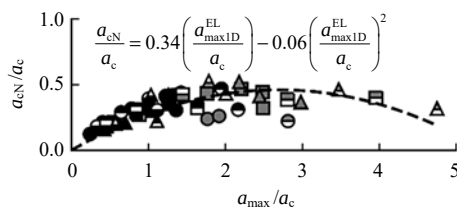
to seek a correlation between a_{cN}/a_c and a_{max2D}/a_c , where the acceleration time-histories to compute a_{cN} are obtained from the 2D FE analyses at surface, behind the wall. As shown in Fig.22(a), a_{cN}/a_c increases from 0.1 to 0.5 with a_{max2D}/a_c increasing from 0 to ~2.5, beyond which it reduces. This trend is consistent with the results shown by other researchers^[25]. The values of a_{cN} were also back-calculated using the surface acceleration time-histories from 1D NL and EL site response analyses. Results, represented in Fig.22(b) and Fig.22(c) respectively, are characterised by greater scatter. The correlation between a_{cN}/a_c and a_{max}/a_c (where $a_{max} = a_{max1D}^{NL}$ in Fig.22(b) and $a_{max} = a_{max1D}^{EL}$ in Fig.22(c)) remains evident.



(a) Acceleration time-histories to compute a_{cN} are obtained from the 2D FE analyses at surface, behind the wall



(b) Obtained from the 1D NL site response analyses, at surface

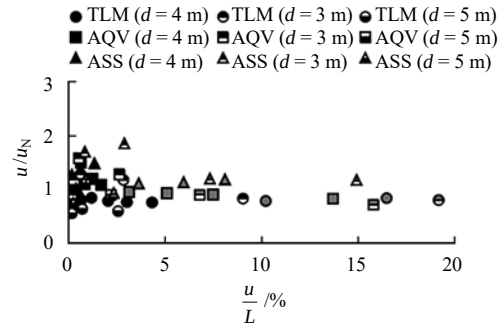


(c) Obtained from the EL site response analyses, at surface

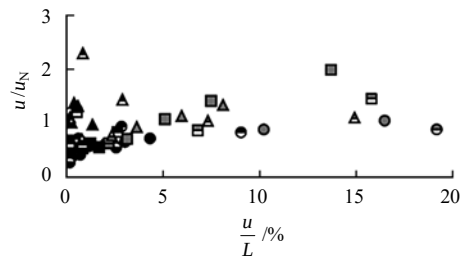
Fig. 22 Reduced critical acceleration for Newmark sliding block method (a_{cN}) back-calculated to match the permanent seismic top-toe displacements by FEM; grey markers indicate $a_g > 0.6g$

As shown in Fig.23, for u/L greater than ~5% a realistic estimate of the permanent top-toe displacements induced by the earthquakes may be obtained by Newmark method, although the input accelerograms are obtained from 1D site response analyses. The estimate is more accurate if the input motion is obtained from 1D NL analyses, as

they use the same soil model (HS small) in the 2D FE analyses. For u/L smaller than ~3%, Newmark method may strongly underestimate or overestimate the displacement. In this case it might be preferable (and easier) adopting the correlations of Fig.20(b) and Fig.20(c).



(a) The accelerograms are obtained from 1D NL site response analyses and a_{cN} from Fig.22(b)



(b) Obtained from 1D NL site response analyses and a_{cN} from Fig.22(c)

Fig. 23 Accuracy of the top-toe displacement predicted by Newmark sliding block method with reduced acceleration

5 Conclusions

Fully-dynamic finite element (FE) analyses were carried out in PLAXIS 2D to simulate the seismic behaviour of cantilever earth retaining walls embedded in a dry uniform sand and assess the ability of simplified approaches to predict the numerical results in terms of earth stresses, wall internal forces and earthquake-induced permanent displacements.

The HS small soil model was used in PLAXIS to reproduce realistically soil non-linear behaviour at small strain. The wall was elastic, with infinite strength. Three different recorded accelerograms, each one scaled to different peak ground accelerations (a_g), were used in the analyses. The embedded depth of the wall was varied between 3 m and 5 m. In this study, the soil properties, excavation depth and bedrock depth were not changed throughout the analyses. The stiffness of the wall was representative of high modulus steel walls, concrete pile

walls and concrete diaphragms. Site response analyses (SRA) were carried out both in PLAXIS (time-domain analyses with non-linear soil model) and STRATA (1D equivalent-linear analyses with Idriss^[48] damping/modulus curves), so that correlations between the results from simpler 1D analyses and those from the 2D FE simulations could be sought. The simplified approaches considered in this study are the pseudostatic method by Conti et al.^[15] or PSM1, with regard to the estimate of the earth stresses and internal forces, and semiempirical correlations and Newmark sliding block method, with regard to the evaluation of wall displacements.

The maximum accelerations at ground surface from the FE 2D models were approximately linearly proportional to those obtained from SRAs, with the excavation amplifying site response by ~30% for $a_g \leq 0.6g$.

The upper-bound value of maximum bending moment from the FE analyses was about 1.5–1.7 times larger than the critical bending moment by PSM1 (M_c); this is explained by pseudostatic methods assuming a linear distribution of the earth stresses behind the wall, while the FE analyses showed significantly higher stresses above the dredge level, except for static conditions or weak earthquakes.

Given an earthquake with a specific value of a_g , the seismic coefficient k_h^{eq} to be used with PSM1 may be estimated as a function of a_{max1D}/a_c , where a_{max1D} is soil maximum acceleration at surface from 1D equivalent-linear SRA with damping/modulus curves by Idriss and a_c is the critical acceleration by PSM1, provided that $a_{max1D}/a_c \leq 1.5g$.

Permanent top-toe wall displacements (u) smaller than ~4% of total wall length (L) may be also estimated as a function of a_{max1D}/a_c ; larger displacements may be estimated more accurately by Newmark sliding block method with reduced critical acceleration a_{cN} valued as a function of a_{max1D}/a_c .

Within the limited range of conditions investigated, the following steps may be followed to assess M_{max} and u for cantilever earth retaining walls in dry sand without the need of complex FE analyses:

- 1) 1D equivalent-linear SRA (with modulus/damping curves by Idriss) to determine a_{max1D} .
- 2) Pseudostatic analysis (with PSM1) to compute a_c .
- 3) Correlation of Fig.17(c) to estimate k_h^{eq} .
- 4) PSM1 to determine M_{max} with $k_h = k_h^{eq}$.
- 5) Correlation of Fig.20(c) to estimate u for

$u/L < 4\%$.

- 6) Correlation of Fig.22(c) to estimate a_{cN} and hence u by Newmark sliding block method for $u/L > 4\%$.

The results from this study are limited to the relatively narrow range of conditions that have been investigated. Further work is underway to extend the validity of this study to a wider range of conditions, i.e. different excavation depths, depths of the bedrock and soil properties, larger set of accelerograms, flexible walls. Moreover, the effect of the formation of plastic hinges in the wall, and that of pore water pressure should also be investigated.

Declaration of competing interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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