Comparison of different approaches to seismic design of geosynthetic-reinforced retaining structures

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ABSTRACT: Earthquake geotechnical engineering has developed enormously in the last decades. This article focuses on the seismic design of geosynthetic-reinforced soil structures. It presents the available calculation methods. Pseudo-static and pseudo-dynamic approaches based on the limit state analysis are compared. Their advantages and disadvantages are discussed. This article also presents the results of external stability analysis for a waterfront reinforced soil retaining structure subjected to seismic forces. The results were obtained by a selected pseudo-static method of calculations. Parametric studies were carried out to quantify the effect of different factors, such as the angle of internal friction and the magnitude of seismic accelerations, on the minimum length of the geosynthetic reinforcement required to ensure the seismic stability of reinforced soil structures. Main practical conclusions are described.

Keywords: reinforced soil, seismic design, geosynthetics

1 INTRODUCTION

The technique of ground improvement by geosynthetic reinforcement has developed over the past several decades. Owing to technical and economic advantages of retaining structures with reinforced backfill (low material cost and short construction period), they have been constructed far more frequently than their conventional counterparts. In earthquake-prone areas, it is very important to understand the behavior of reinforced soil structures under seismic loading conditions. The seismic design of reinforced soil structures has gained worldwide attention only in recent decades. The dynamic response of even the simplest type of retaining wall is quite complex and needs both experimental and theoretical investigations.
The purpose of this study is to review the present state of the art on the seismic design of geosynthetic-reinforced soil structures.
Information obtained from post-earthquake investigations is invaluable for the verification and improvement of the seismic design procedure. Recent earthquake events have provided excellent opportunities to document the seismic performance of geosynthetic-reinforced structures. Therefore, the current paper begins with a summary of main conclusions from post-earthquake observations.
The second part of the present contribution is devoted to reviewing the main methods used in the seismic design of geosynthetic-reinforced soil structures. The pioneering work on earthquake-induced lateral earth pressure under active and passive conditions acting on retaining walls was done by Okabe (1926) and by Mononobe and Matsuo (1929). Currently

several different methods are used to analyse the behaviour of reinforced soil structures under seismic loading. They can be classified into several groups. The most commonly used methods for the seismic design of reinforced structures are presented in this paper. Their advantages and disadvantages are discussed.
In the last part of this paper, the results of an analysis of the external stability of a waterfront reinforced soil retaining structure under seismic conditions are presented. Although much progress has been made in the seismic design of retaining structures, only very limited research has been done to investigate the effect of water in backfill. Most of the research has been devoted to structures with dry backfill. In the present study, the seismic stability of a geosynthetic-reinforced soil wall was investigated by a selected pseudo-static approach. Special attention was focused on factors strongly affecting the required geosynthetic length, such as the soil friction angle and the values of horizontal and vertical seismic accelerations.

2 MAIN CONCLUSIONS FROM POST-EARTHQUAKE INVESTIGATIONS

2.1 *General information*

Information obtained from post-earthquake investigations is invaluable for the verification and improvement of the seismic design procedure. Recent earthquake events have provided excellent opportunities to document the seismic performance of geosynthetic-reinforced structures. Case histories and extensive post-earthquake investigations have shown that geosynthetic-reinforced soil walls with granular backfill soils perform well under earthquake loading. At sites where reinforced and unreinforced soil retaining structures have been built, a better performance has been achieved with the reinforced soil structures. Due to the above advantages, the application of geosynthetic reinforcement has been extending to wider areas. Geosynthetic-reinforced earth structures have been selected for the construction of important new permanent structures, as well as for the repair and reconstruction of existing ones (Koseki, 2012).
Their satisfactory seismic performance can be attributed to:

- The technical advantages of geosynthetic-reinforced soil structures. They have a better seismic stability due to their greater dynamic flexibility and high ductility.
- The conservatism of the static design procedures. Very often, large factors of safety are used in calculation procedures.

The conservatism and the large factors of safety have decreased substantially in recent years. As a result, a more accurate seismic design may be required for a satisfactory seismic performance of such structures in the future.

2.2 *Lessons learned from post-earthquake investigations*

Although, the performance of geosynthetic-reinforced soil structures under earthquake loading has generally been good, a few cases of such structures being destroyed have also been reported. On the basis of post-earthquake inspections, the following causes of the failure of reinforced soil structures have been identified (Koseki, 2012; Ling et al, 2001):

- insufficient compaction of the backfill material;
- inadequate resilience of the foundation material;
- excessive reinforcement spacing (>800mm);
- insufficient reinforcement length (<0.7 of wall height);
- mistakes in general failure analysis of the structure and/or incorrect assumptions concerning potential failure modes. (Cracks have been observed behind reinforced soil wall structures. They did not have adequate global stability.);

- insufficient rigidity of the wall facing, pull-out resistance of the reinforcement, and strength of the connection between the two. (The connection between the facing elements and the reinforcement is vital for a satisfactory performance of reinforced soil structures under seismic loading.);
- insufficient strength and/or stiffness of some elements/materials used for the connection (e.g. pins).

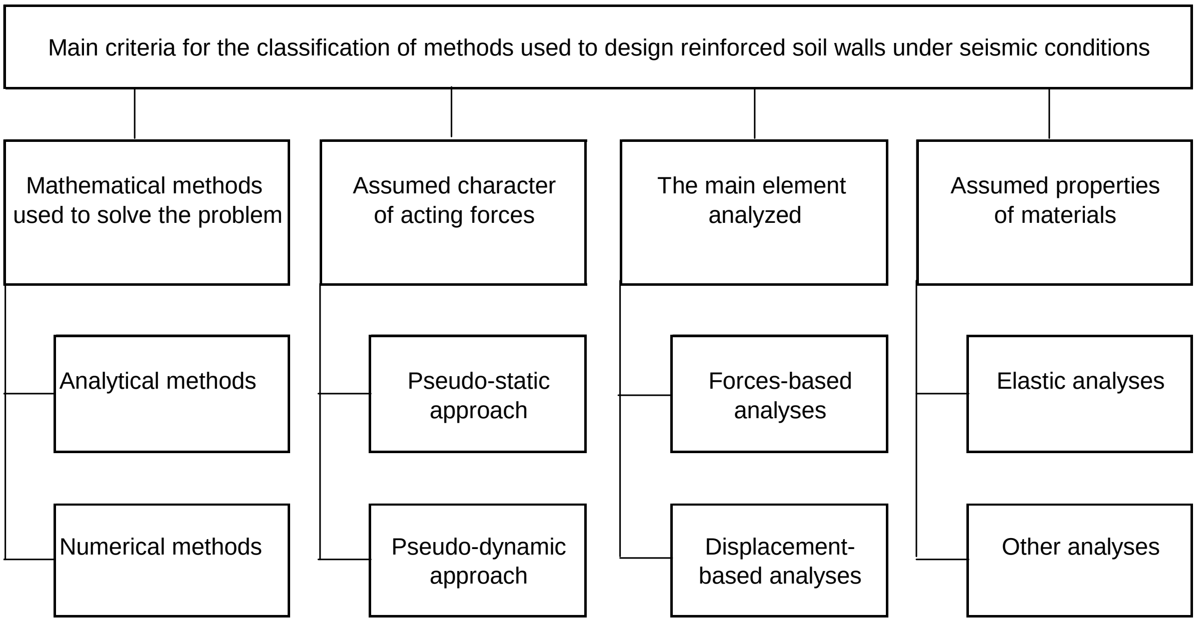
2.3 *Conclusion*

It is well-known that the external stability of geosynthetic-reinforced structures is reduced when the wall is subjected to horizontal loads during an earthquake event. Nonetheless, most of the reported post-earthquake investigatios show that the advantages of geosynthetic-reinforced soil structures over conventional retaining structures include earthquake resistance.

3 METHODS USED FOR SEISMIC DESIGN

3.1 *General characteristics*

Table 1. Methods used to design the seismic stability of geosynthetic-reinforced structures.



The methods of the analysis and design of geosynthetic-reinforced soil structures under seismic conditions can be classified into several groups (Table 1). It is possible to distinguish two general approaches to this problem. In the first, analytical models are used. This approach requires proper and realistic assumptions as to working of reinforced soil structures (e.g. failure mechanisms) and their particular elements. Modelling errors cause significant errors in calculation results.
The second approach to the seismic design of geosynthetic-reinforced walls is based on numerical methods (especially finite element methods). These design methods make it possible to determine material parameters that would be difficult to measure in an experimental study.

Table 2. Comparison of pseudo-static and pseudo-dynamic methods of seismic design.

<table>
<tr><th>Main aspects</th><th>Pseudo-static methods</th><th>Pseudo-dynamic methods</th></tr>
<tr><td>Advantages/ disadvantages</td><td>- Simplicity of calculation methods.
- These methods do not consider the effects of time and body waves travelling through the soil during the earthquake.
- Seismic design of reinforced soil structures is typically done by this approach.</td><td>- The dynamic nature of earthquake loading is taken into account. It is considered in an approximate and simple manner.
- Calculations are more complicated compared with those made according to the pseudo-static approach.</td></tr>
<tr><td>Basic input parameters</td><td colspan="2">- Structure parameters: height (H); slope angle (β).
- Soil parameters: cohesion (c), soil friction angle (ϕ).
- Reinforcement parameters: length (Lgeo); strength (R).
- Seismic forces parameters: seismic acceleration coefficients.
Boundary conditions: e.g. water level, internal forces acting on the structure.</td></tr>
<tr><td>Main goals of the design procedure</td><td colspan="2">- To find the minimum required geosynthetic reinforcement parameters (length, stiffness and number of layers) needed for seismic design.
- To check the stability of the structure under the forces acting on it.</td></tr>
<tr><td>Calculation methodology</td><td colspan="2">- Potential failure modes are identified.
- The stability analysis of the structure is performed.
- The minimum parameters of geosynthetic reinforcements are calculated.</td></tr>
<tr><td>The total inertia forces (in horizontal and vertical directions)</td><td>- The magnitudes of the pseudo-static forces (in horizontal and vertical directions) are:
$F_h = W \cdot \frac{a_h}{g} = k_h \cdot W$
$F_V = W \cdot \frac{a_V}{g} = k_V \cdot W$
a_h , a_v– horizontal and vertical pseudo-static accelerations
W – weight of the failure wedge</td><td>- The total inertia forces (in horizontal and vertical direction):
$F_h = \int_0^H m(z) \cdot a_h(z,t)$
$F_V = \int_0^H m(z) \cdot a_V(z,t)$
- The mass of a thin element slice with thickness dz at depth z is given by:
$m_z = \frac{\gamma}{g} \cdot \left(\frac{H-z}{\tan\alpha}\right) dz$</td></tr>
<tr><td>Seismic acceleration coefficients (in horizontal and vertical directions)</td><td>- Constant values of both horizontal (k_h) and vertical (k_v) coefficients of pseudo-static accelerations are assumed.
-The seismic coefficients are used to express the earthquake inertia force as a percentage of the deadweight of the potential failure soil mass.</td><td>- The base of the structure is subjected to harmonic horizontal and vertical seismic accelerations of amplitudes a_h and a_v.
- Accelerations at depth z below the top of the wall are expressed as:
$a_h(z,t) = a_h \cdot \sin\omega\left(t - \frac{H-z}{V_s}\right)$
$a_v(z,t) = a_v \cdot \sin\omega\left(t - \frac{H-z}{V_p}\right)$</td></tr>
<tr><td>Other seismic wave parameters used for calculations</td><td>They are not taken into account.</td><td>- The shear wave velocity:
$V_s = (G/\rho)^{1/2}$
- The primary wave velocity:
$V_p = [G(2-2\nu)/\rho(1-2\nu)]^{1/2}$;
$V_p / V_s = 1.87$ (Kramer, 1996)
- The period of lateral shaking:
$T = 2\pi/\omega = 4H/V_s$</td></tr>
</table>

However, the development of typical numerical calculation procedures led to some important idealizations of the problem. The main ones deal with the geometry of the structure, load conditions, material behaviour, constitutive models of materials, and the selection of numerical techniques. The main problem of this approach is that particular models are not verified against extensive experimental data.

3.2 *Analytical methods*

Analytical approaches for the seismic analysis of retaining walls can be divided into three main categories: pseudo-static methods, pseudo-dynamic methods and displacement methods. The first two categories are based on force equilibrium analysis. The last one includes the displacement-based sliding block method. The pseudo-static approach, in which the effects of earthquake forces are expressed by constant horizontal and vertical accelerations attached to the mass, is well established in geotechnical practice. The first seismic design method for reinforced soil structures was proposed by Richardson and Lee (1975). It was based on the assumptions of the Mononobe-Okabe method (Mononobe et al., 1929; Okabe, 1926) and can also be used for the seismic analysis of reinforced soil walls. Currently, numerous methods are available for the seismic design of reinforced soil structures based on the pseudo-static method of analysis. Most of these methods are used to determine the required length and strength of the reinforcement in soil walls by assuming different failure modes. The pseudo-static limit equilibrium method does not consider the effect of time or of body waves travelling through the soil during the earthquake. As a result, pseudo-static analyses are relatively simple, but they constitute a conservative approach to the design of reinforced soil structures. In the pseudo-dynamic methods, an advancement over the previous approach is that the dynamic nature of earthquake loading is considered in an approximate and simple manner. The effects of time and body waves are included in calculation procedures. In addition, both the shear and primary waves propagating through the soil with variation in time are usually taken into account by assuming harmonic horizontal and vertical seismic accelerations. The main aspects of pseudo-static and pseudo-dynamic approaches are summarized in Table 2, including their advantages and disadvantages, as well as the main assumptions used in calculations.

3.3 *Numerical methods*

Numerical methods of calculations (especially those based on finite element methods) are becoming very popular which is due to the increasing availability of professional software, as well as to the growing knowledge of their advantages. It is not possible to predict the load-deformation response or deformation of reinforced soil walls by the limit equilibrium methods. Numerical methods do not have this restriction. During numerical calculations, the soil and reinforcement can be considered as a composite, homogeneous reinforced soil structure or modelled as different materials. Numerical investigations of the seismic behaviour of reinforced soil structures are also more economical than the physical model tests (because they are cheaper and less time consuming). Carefully planned and executed numerical experiments improve the knowledge about the influence of dynamic loading on reinforced soil structures. They can be used to investigate the influence of geosynthetic reinforcement properties (e.g. reinforcement length, strength and stiffness), wall geometry, facing type, and base condition on the response of the system to seismic loading. A summary of typical input parameters used in numerical simulations performed for the seismic design of geosynthetic-reinforced soil structures is presented in Table 3.

Table 3. Basic information about selected numerical methods used to investigate the seismic performance of geosynthetic-reinforced soil structures.

Reference	Software	Facing model	Reinforcement model	Backfill model	Interface model	Input motion
Burke (2004)	Diana-Swandyne	Linear elastic	1-D bounding surface (3-node bar element)	Pastor-Zienkiewicz III (8-6-node element)	Slip element	Kobe record
El-Emama et al. (2001)	Flac	Not given	Elastic-plastic (2-node cable element)	Mohr-Coulomb elastic-plastic	Not given	Sinusoidal record
Fujii et al. (2001)	Flip	Elastic	Elastic (linear beam element)	Multi-spring	Joint element	Kobe record
Helwany et al. (2001)	Dyna3D	Not given	Linear elastic (shell element)	Perfectly elastic	Penalty-based interface	Sinusoidal record
Lee et al. (2010)	Ls-Dyna	Linear elastic	Plastic-kinematic	Cap model	Linear elastic	Kobe record
Ling et al. (2004)	Diana-Swandyne	Linear elastic	Bounding surface	Generalized plasticity soil	Elastic perfectly plastic	Sinusoidal record
Liu et al. (2011)	Abaqus 6.4	Linear elastic	1-D bounding surface (3-node bar elements)	Elastoplastic-viscoplastic	Thin layer elements Mohr-Coulomb	Kobe record
Ye et al. (2012)	Flip	Linear elastic	Elastic material	Elasto-plastic	Goodman's elements	Sinusoidal record

4 METHODOLOGY USED IN CALCULATIONS

4.1 *Formulation of the problem*

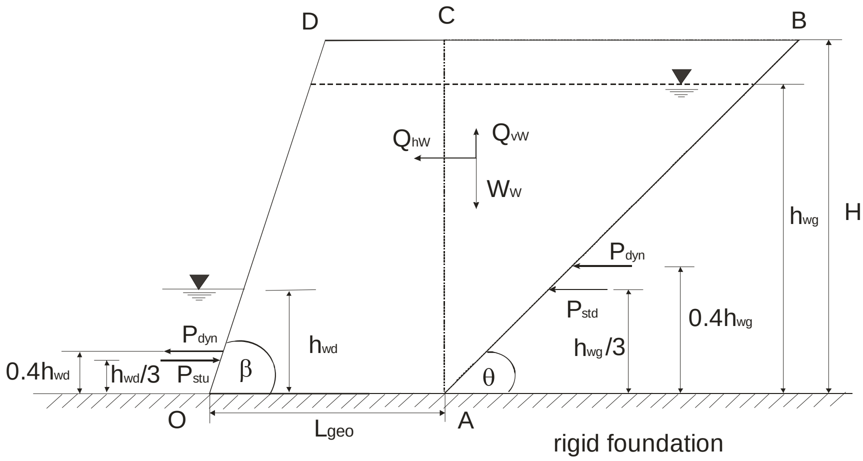
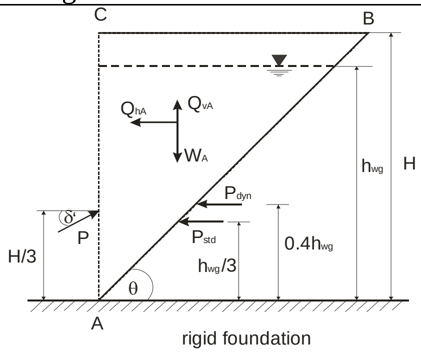
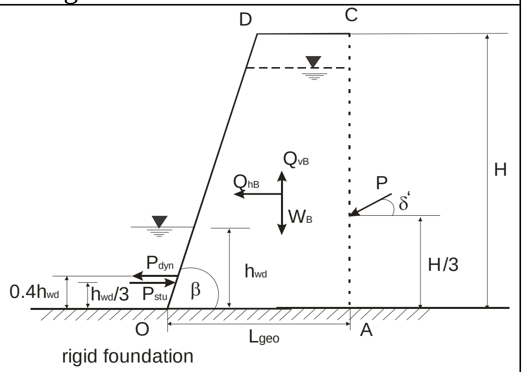


Figure 1: Overview of the waterfront reinforced soil retaining structure

To date, several investigations for the analysis and design of earthquake-resistant reinforced soil walls have been carried out by the pseudo-static or pseudo-dynamic methods. Most of the research has been conducted for dry soil conditions. Although much progress has been made in the seismic design of reinforced soil structures, only very limited attention has been devoted to the presence of water in the backfill (Choudhury and Ahmad, 2009; Ahmad and Choudhury, 2012).

Table 4. Preliminary calculations – short description

Main points	Wedge A	Wedge B
Forces acting on the selected wedges (A, B)	 θ - its value is variable; to obtain the worst-case combination of the forces, a special calculation procedure (presented below) was used	 L_{geo} – minimum required length of the reinforcement needed to resist failure modes
Weights of wedges A (W_A) and B (W_B)	$W_A = \dfrac{\gamma' H^2}{2\tan\theta}$;	$W_B = \dfrac{1}{2}\gamma' H\left(2L_{geo} - K\right)$; $K = H/\tan\beta$
	$\gamma' = \left(\dfrac{h_{wg}}{H}\right)^2 \gamma_{sat} + \left[1 - \left(\dfrac{h_{wg}}{H}\right)^2\right]\gamma_d$ γ' - equivalent unit weight of soil, modified due to the submergence of the backfill; γ_d, γ_{sat} - dry and saturated unit weights of soil	
Angle (θ) that the failure rupture surface (AB) makes with the horizontal	$\theta = \phi - \psi + \tan^{-1}\left[\dfrac{-\tan(\phi-\psi) + C_{1E}}{C_{2E}}\right]$ $C_{1E} = \sqrt{\tan(\phi-\psi)\left[\tan(\phi-\psi)+\cot(\phi-\psi)\right]\left[1+\tan(\delta+\psi)+\cot(\phi-\psi)\right]}$ $C_{2E} = 1 + \tan(\delta+\psi)\left[\tan(\phi-\psi)+\cot(\phi-\psi)\right]$; $\psi = \tan^{-1}\left[\dfrac{\gamma_{sat} k_h}{\gamma'(1-k_v)}\right]$	
Seismic inertia forces acting on wedges A and B in horizontal (Q_h) and vertical (Q_v) directions	$Q_{hA} = k_h W_A$ $Q_{vA} = k_v W_A$	$Q_{hB} = k_h W_B$ $Q_{vB} = k_v W_B$
	k_h, k_v – seismic acceleration coefficients in horizontal and vertical directions	
Hydrostatic forces acting on downstream (P_{std}) and upstream (P_{stu}) sides of the structure	$P_{std} = \dfrac{1}{2}\gamma_{we}\left(h_{wg}\right)^2$ γ_{we} - unit weight of water, modified due to the submergence of the backfill $\gamma_{we} = \gamma_w + (\gamma' - \gamma_w) r_u$ γ_w - unit weight of water r_u - pore pressure ratio	$P_{stu} = \dfrac{1}{2}\gamma_w\left(h_{wd}\right)^2$ γ_w - unit weight of water
Hydrodynamic force (P_{dyn})	P_{dyn} is calculated by the Westergaard (1933) approach as: $P_{dyn} = \dfrac{7}{12} k_h \gamma_w \left(h_{wg}\right)^2$	
Interwedge force (P)	$P = \dfrac{Q_{hA} - W_A\tan(\phi-\theta) + Q_{vA}\tan(\phi-\theta) + P_{dyn} + P_{std}}{\cos\delta' - \sin\delta'\tan(\phi-\theta)}$ $\delta' = \varphi$ - inclination of interwedge force P to the horizontal	

Its effect on the behaviour of the structure under seismic loading conditions has not been sufficiently discussed. The aim of this study was to improve the knowledge about the stability of a waterfront reinforced soil retaining structure subjected to seismic loading. Special attention was focused on factors strongly affecting the required geosynthetic length. A typical model of the retaining wall with cohesionless backfill was considered in the analysis. The cross-section of a waterfront reinforced soil retaining structure is shown in Figure 1 (Choudhury and Ahmad, 2009).

The failure zone is defined by the planar rupture surface AB, inclined at the angle θ to the horizontal. Two parts of the reinforced zone were selected: the triangular portion ABC, (denoted as wedge A in Figure 1) and the quadrilateral portion OACD (denoted as wedge B in Figure 1). The equilibrium of these two parts of the structure under loading was analysed separately. Two failure mechanisms of the reinforced wall (sliding instability and overturning instability) were examined. The minimum length of the geosynthetic reinforcement (L_{geo}) required to resist both selected failure modes was calculated by a selected limit equilibrium method and a pseudo-static approach (Choudhury and Ahmad, 2009). Series of calculations were carried out to better understand the mechanism of the working of geosynthetic-reinforced structures and to investigate the influence of seismic loading on their stability.

The analyses were conducted:

- to assess the impact of the magnitude of seismic loading on the external stability of a waterfront reinforced soil retaining structure (different values of the horizontal and vertical seismic acceleration coefficients were assumed during calculations);
- to investigate the relative effect of selected variables known to affect the performance of geosynthetic-reinforced soil walls, such as the height of the structure and the value of its slope angle;
- to examine backfill properties by testing different types of subgrade materials (the influence of changing soil friction angles and pore pressure ratio values was investigated).

4.2 *Forces acting on the structure*

Details of the calculating procedure are presented in Table 4 (description of preliminary calculations) and in Table 5 (stability formulae). They were assumed after Choudhury and Ahmad (2009). Forces acting of the two selected parts of the structure (wedge A and wedge B) were calculated separately. They are shown in Figure 1. The calculation methodology is described in Table 4.

4.3 *Stability analysis*

The reinforcement in geosynthetic wall structures must be designed with an adequate strength and length to resist various possible modes of failure. The external seismic stability of the waterfront reinforced soil wall against base sliding and overturning modes of failure was checked. For a direct sliding failure of the structure, the sum of all driving forces and pressures (F_D) acting on the wedge B in the horizontal direction was compared with the sum of all resisting forces and pressures (F_R). The minimum required length of the reinforcement needed to resist a direct sliding failure was calculated for a critical limitation case, in which F_R=F_D. The seismic stability against an overturning failure was checked by comparing the total driving moment (M_D) and the total resisting moment (M_R) acting on the selected part (wedge B) of the reinforced structure. The minimum length of the geosynthetic reinforcement needed to resist an overturning failure of the structure was calculated for a critical condition in which M_R=M_D.

Table 5. Stability conditions – short description.

Main points	Stability against direct sliding	Stability against overturning
Stability conditions	$F_R \geq F_D$	$M_R \geq M_D$
Notations	$F_R; F_D$ - total resisting and driving forces $F_R = (P\sin\delta' + W_B - Q_{vB})C_{ds}\tan\phi + P_{stu}$ $F_D = P\cos\delta' + Q_{hB} + P_{dyn}$	$M_R; M_D$ $M_R = P\sin\delta' l_{ot} + (W_B - Q_{vB})x_c + P_{stu}\dfrac{h_{wd}}{3}$ $M_D = P\cos\delta'\dfrac{H}{3} + Q_{hB}y_c + P_{dyn}(0{,}4h_{wd})$
Minimum required length of the reinforcement	$l_{ds} = \dfrac{P\sin\delta' C_{ds}\tan\phi - P\cos\delta' + P_{stu} - P_{dyn}}{\gamma' H\left[k_h - (1-k_v)C_{ds}\tan\phi\right]} + \dfrac{H}{2\tan\beta}$	$l_{ot} = \dfrac{-b \pm \sqrt{b^2 - 4ac}}{2a}$
Other notations	C_{ds} – the coefficient of direct sliding $C_{ds} = 0.75$ (Ling and Leshinschky, 1998)	$a = 0{,}5\gamma' H(k_V - 1)$; $b = 0{,}5\gamma' H^2 k_h - P\sin\delta'$; $c = \dfrac{1}{6}\gamma' H\left[k_h K^2 - 3k_h KH + K^2(1-k_v)\right] -$ $- P_{stu}\dfrac{h_{wd}}{3} - P\cos\delta'\dfrac{H}{3} - P_{dyn}(0{,}4h_{wd})$ $K = H/\tan\beta$;

5 RESULTS OF CALCULATIONS

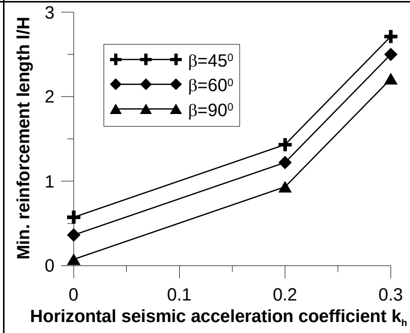
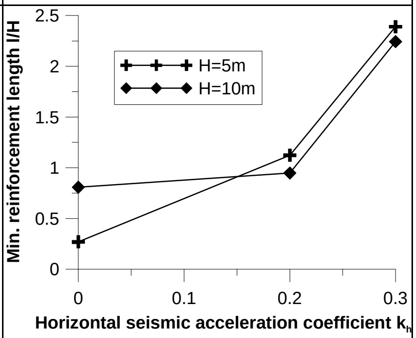
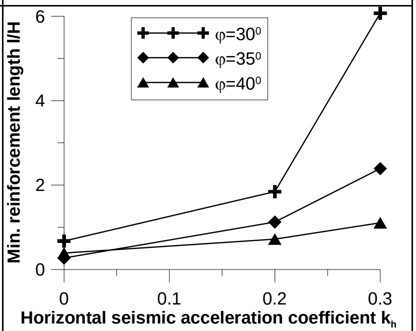
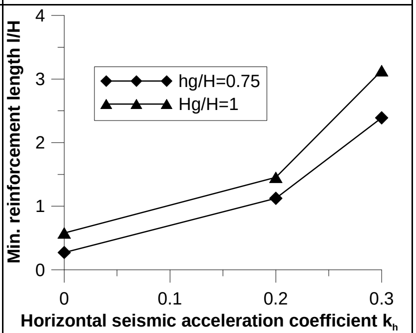
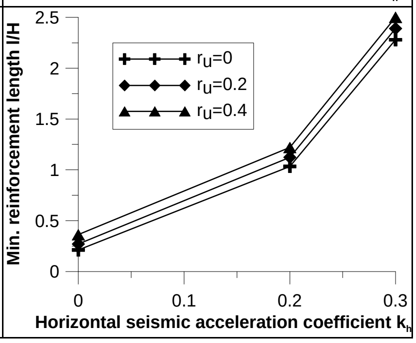
5.1 *Input parameters*

The values of the main initial parameters used in calculations are summarized in Table 6. The external seismic stability was analysed for the waterfront reinforced model wall characterized by typical parameters used in field structures. To broaden the knowledge about the behavior of this kind of structures under seismic loading, several calculations were made to investigate the relative effect of selected variables known to affect the required length of the waterfront reinforcement such as the magnitude of seismic loading, the backfill friction angle and the pore pressure ratio.

Table 6. Basic input data.

Basic Parameters	Ranges of input values
Structure	height of the structure (m): 5; 10; slope angle (degrees): 45^0, 60^0, 90^0
Soil parameters	γ_{sat} = 19 kN/m^3 γ_d = 16 kN/m^3
Water level	h_{wg}/H =0,75 h_{wg}/H =1
Seismic coefficients	$k_h = 0$; $k_h = 0.2$; $k_h = 0.3$ $k_v = k_h/2$
Other parameters	γ_w=10 kN/m^3, $\delta' = \phi$; C_{ds}=0.75

Table 7. The required length of the geosynthetic reinforcement in the direct sliding mode.

<table>
<tr><th colspan="2">Input parameters</th><th rowspan="2">Min. required length of the reinforcement</th></tr>
<tr><th>Constant values</th><th>Changed values</th></tr>
<tr><td>H=5m
$\varphi=35^0$
$r_u=0.2$
$h_{wg}/H=0.75$
$h_{wd}/h_{wg}=0.5$
$k_h=0;\ 0.2;\ 0.3$
$k_v=k_h/2$</td><td>Slope angles:
$\beta_1=45^0$;
$\beta_2=60^0$;
$\beta_3=90^0$</td><td></td></tr>
<tr><td>$\beta=60^0$
$\varphi=35^0$
$r_u=0.2$
$h_{wg}/H=0.75$
$h_{wd}/h_{wg}=0.5$
$k_h=0;\ 0.2;\ 0.3$
$k_v=k_h/2$</td><td>Height of the structure:
H_1= 5m
H_2=10m</td><td></td></tr>
<tr><td>$\beta=60^0$
H=5m
$r_u=0.2$
$h_{wg}/H=0.75$
$h_{wd}/h_{wg}=0.5$
$k_h=0;\ 0.2;\ 0.3$
$k_v=k_h/2$</td><td>Soil friction angle:
$\varphi_1=30^0$;
$\varphi_2=35^0$;
$\varphi_3=40^0$</td><td></td></tr>
<tr><td>$\beta=60^0$
H=5m
$r_u=0.2$
$h_{wd}/h_{wg}=0.5$
$\varphi=35^0$
$k_h=0;\ 0.2;\ 0.3$
$k_v=k_h/2$</td><td>Water level:
$h_{wg}/H=0.75$;
$h_{wg}/H=1$</td><td></td></tr>
<tr><td>$\beta=60^0$
H=5m
$h_{wg}/H=0.75$
$h_{wd}/h_{wg}=0.5$
$\varphi=35^0$
$k_h=0;\ 0.2;\ 0.3$
$k_v=k_h/2$</td><td>Pore presssure ratio:
$r_{u1}=0$
$r_{u2}=0.2$
$r_{u3}=0.3$</td><td></td></tr>
</table>

5.2 *Results and discussion*

The main objective of the internal design of geosynthetic-reinforced soil retaining walls and slopes is to define the required parameters of the reinforcement. The aim of the analysis presented here was to find the relationship between the minimum required length of the geosynthetic reinforcement and the main parameters of the structure and materials under seismic loading. The pull-out of the reinforcement was taken into account in the stability analysis. Typical results of calculations are presented in the graphical form in Table 7. The length of the geosynthetic reinforcement (expressed in the non-dimensional parameter l/H, where H – is the height of structure) required to maintain the stability of the reinforced soil wall under seismic conditions was examined. Calculation results show that direct sliding is the critical failure mode. Therefore the graphs are plotted for this case. Parametric studies illustrate the effects of seismic acceleration on the design of reinforced soil structures having different slope angles, height and soil properties.

6 CONCLUSIONS

The main purpose of this paper was to better understanding the behaviour of geosynthetic-reinforced structures subjected to seismic loading. Special attention was paid to the external stability of the waterfront reinforced soil retaining structure under seismic conditions. A simple theoretical calculation procedure (pseudo-static approach) was applied to investigate the behaviour (Choudhury and Ahmad, 2009) of such structures. External stability analyses were conducted to determine the required length of the geosynthetic reinforcement, considering two selected modes of failure (direct sliding and overturning). The results show that, out of these two modes, direct sliding is the critical one, and thus needs to be given due consideration. The required reinforcement length was calculated for different design parameters of the reinforced soil wall. Typical relationships between the main values of structure and material parameters and the required length of the geosynthetic reinforcement are presented in Table 8.

Table 8. Relationships between the values of selected parameters and the required length of the geosynthetic reinforcement.

Parameter	Influence on the required length of the reinforcement
Slope angle	Assuming no change in the other construction parameters, a longer geosynthetic reinforcement is required for a sloping wall than for a vertical wall.
Soil friction angle	As the value of the soil friction angle increases, the required length of the geosynthetic reinforcement decreases. It means that because of the increasing internal angle of soil friction, the stability of the retaining wall increases and the total geosynthetic mobilized force decreases.
Pore pressure ratio	The required geosynthetic reinforcement length increases when the pore pressure ratio increases. The same relationship is observed for both direct sliding and overturning modes.
Water level	As the water level increases, the required length of the geosynthetic reinforcement increases. The presence of water has a highly destabilizing effect on the reinforced soil structure.
Seismic coefficients: - horizontal - vertical	With an increase in the horizontal and vertical seismic accelerations, the geosynthetic length required for external seismic stability increases.

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