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A Critical Review of Soil Models and Factors Affecting Earth Retaining Structures Design

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ABSTRACT

Earth retaining structures, ERSs, are used in many engineering fields. Special considerations and technical knowledge in geotechnical engineering should be adopted in the modeling, analysis, and design of ERSs. Some of these considerations are related to the soil models, and (quality, settlement, and inclination) of the backfill and foundation soils of the retaining wall, RW, as presented in this critical review. This review shows that the analysis and design of ERSs are highly affected by model adopted for soil behavior, and the quality and characteristics of backfill materials. The backfill materials affect the selection of material type and performance of the ERSs, furthermore, they impact the soil interaction with RW. In selecting an appropriate model, it is important to consider the effect of soil history and stress changes that may the soil experience in the future. The design of some types of ERSs, backfilled with a material predominantly finer than the coarse sand grain size, should be conducted with precautions due to the possibility of lateral earth pressure (LEP) changing from active state to at-rest state after construction. It is worthwhile to consider both short-term and long-term settlements in the analysis and design of ERSs as there are specific types of ERSs that can tolerate large short-term settlements but cannot tolerate large long-term settlements. Finally, under both static and dynamic loadings, the angle of inclination of the backfill soil greatly affects the distribution of LEP and the value of the resultant force behind the RW.

Keywords: Retaining walls design; soil modeling; plastic equilibrium; rigid-plastic; settlement toleration

INTRODUCTION

One of the important design elements in geotechnical engineering practices is the "lateral earth pressure, LEP". Estimation of the LEP is essential for stability analysis or design of a number of engineering facilities. Engineering members that are constructed to support the ground, earth banks, natural soil, fill materials, water, ore piles, coal, grains, or any other materials are called earth retaining structures, ERSs. ERSs are used in many engineering fields including architectural, coastal, bridge, and road engineering (Zhano et al. 1998; NCMA 2009; Tobar & Meguid 2010; Gao et al. 2017; Hazirbaba et al. 2019; Javadi et al. 2021).

Technical knowledge of the designers in structural and soil mechanics is essential for modeling and the typical design of ERSs. Special considerations should be adopted to ensure ERSs' unique design. Some of these considerations are related to the adopted soil model in the analysis, the backfill soil quality and inclination, and the foundation soil below the retaining wall, RW. Information about the description of soil models, retained soils and infill materials should be considered in the analysis and design of ERSs. Generally, a cost-effective, safe, and efficient ERS design can be developed if sufficiently accurate information about the soil models, quality, inclination, and settlement of backfill soil and foundation below the RW is adopted (Brooks 2010; O'Neal & Hagerty 2011; Clayton et al. 2013). Such information has been reviewed and discussed in this paper.

SOIL MODELS AND ANALYSIS OF ERSS

Modeling has become an integral part of the analysis and design of many engineering systems. It's not just limited to traditional structures but also extends to geotechnical systems like retaining walls and different soil-structure applications. When it comes to soil models, numerous scholars have developed different constitutive models to simulate the behavior of soil under varying loading conditions. Various researchers have also examined the practicality of these soil models for geotechnical applications, (Wani & Showkat 2018; Ghazvinian et al. 2020; Dias & Jenck 2022; Çetindemir 2023). In modeling an ERS, the main aim is to gain insight into how the system will react under different scenarios or conditions. This understanding is crucial in creating a logical and effective design that takes into account all possible outcomes. To better comprehend the behavior of the ERS under various circumstances, it is necessary to create a model of the actual system and analyze its response through simulation. Actually, the development of a numerical model to simulate an ERS necessities an idealization of the problem under consideration, numerical model formulation and defining the boundary conditions (by involving the constraints and utilizing the relevant constitutive law), and assessing the response of the numerical system and visualizing of its results (Rahman & Ulker 2018).

The selection of the model(s) of the behavior of the soil and its related parameters for the proposed ERS are needed to conduct geotechnical analysis. In general, it's best to use a straightforward soil model that can provide accurate predictions for any given analysis. The level of complexity needed or possible will depend on various factors. These factors include the design and analysis requirements, the spatial variability and the complexity of the soil profile, and the availability of capabilities necessary to identify the parameters (Clayton et al. 2013).

When modeling the behavior of soils, it's important to take into account features such as the nonlinear behavior of the stress and strain under different loading conditions, the effect of confinement stresses, the dependent nature of the stress path, the impact of the rate of strain, the memory of soils, the hardening, and the dilatancy. Furthermore, the multi–phase system nature of the soils (discrete particulate nature), Figure 1, should be treated when modeling soils' behavior. By factoring in these elements, we can gain a better understanding of how soils will behave in different situations and make more informed decisions about how to approach them (Rahman & Ulker 2018). According to Wood (2004), the crucial factor in effective constitutive modeling is to recognize the significant soil characteristics conducted for a specific purpose. Essential behavior characteristics may be missed if a very simple model is adopted. While many parameters (defined from many field or laboratory tests) are required if a too complex model is adopted.

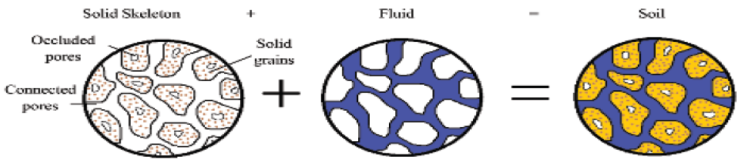


FIGURE 1. Multi-phase system of soil
Source: Rahman & Ulker (2018)

On the other hand, before beginning the development of numerical modeling for ERS, it is important to have a clear understanding of its possible failure modes. A behavioral model that lacks the ability to describe such modes is considered weak. When it comes to understanding failure modes, it's important to gather information from a variety of sources. Case histories that describe real failures are a particularly valuable source of information. Additionally, laboratory tests can offer insight into the behavior of ERS in different scenarios. By studying these observations and applying mechanical concepts, the model can be developed to evaluate different failure modes' probability (Bakker 2000).

In reality, the real behavior of geomaterials is complex, so idealization or approximation must be used so as to formulate the constitutive relations of mathematical expressions. In some cases, geomaterials are assumed to exhibit linear elastic behavior, but in reality, they do not follow either of these properties. In other cases, the behavior of these materials may be assumed as "linear visco–elastic", "nonlinear elastic", or "elastoplastic" (Holtz et al. 2023). To analyze deformations of an ERS, it's important to establish a connection between stress and strain by using a constitutive model, Wood 2004. In such analysis, the commonly used stress and strain relations for some types of soils are shown in Figure 2.

The behaviors shown in these relations are "Rigid Plastic or perfectly plastic", "Linear Elastic Plastic (elastic perfectly plastic)", "Nonlinear Elastic Plastic", and "Strain Softening". There is a group of soil models commonly used in the analysis of ERSs. These models are summarized by Clayton et al. (2013) and include "rigid plastic", "Winkler spring", "elastic (inhomogeneous, cross–anisotropic, linear, and nonlinear)", and "elasto–plastic" models. The main features of each of these models will be reviewed in this section.

The analysis of ERSs according to classical theories does not take the magnitude of RW movement into consideration. According to these theories, the failure condition (active or passive) is produced once the RW movement starts and the full mobilization of the strength is reached to produce the failure condition (Chang, 1997; Clayton et al. 2013; Wang et al. 2018). In such a condition, the behavior of soil (behind the wall) can be considered a rigid plastic, and its stress and strain relation are similar to that shown in Figure 2 a. Materials with perfect plasticity can deform irreversibly without experiencing an increase in loads or stresses. When stress is applied to soil, soil particles initially have a tendency to not exhibit any strain. The strain, however, becomes limitless as soon as the stress reaches the maximal stress (ultimate stress) value (Rahman & Ulker 2018).

The modeling of the geomaterial as a rigid plastic means that no displacement (or strain) is required to take place before reaching the state of failure. When it comes to "rigid plastic" models, determining the strength of failure is only necessary in plane strain (two-dimensional) conditions. The main requirements for "rigid plastic" models are the RW movement direction and the shear strength parameters (cohesive and internal friction). Rowe & Peaker (1965) stated that the adopting of "rigid plastic" models in the estimation of passive LEP can result in a substantial overestimation. Furthermore, Salazar (2023), stated that case studies have revealed that conventional evaluation techniques used to determine the LEP of an ERS often rely on both the "shear failure band" and the "perfectly plastic soil model". However, the latter is not compatible with "shear failure bands", and as a result, many ERSs tend to fail under earthquakes.

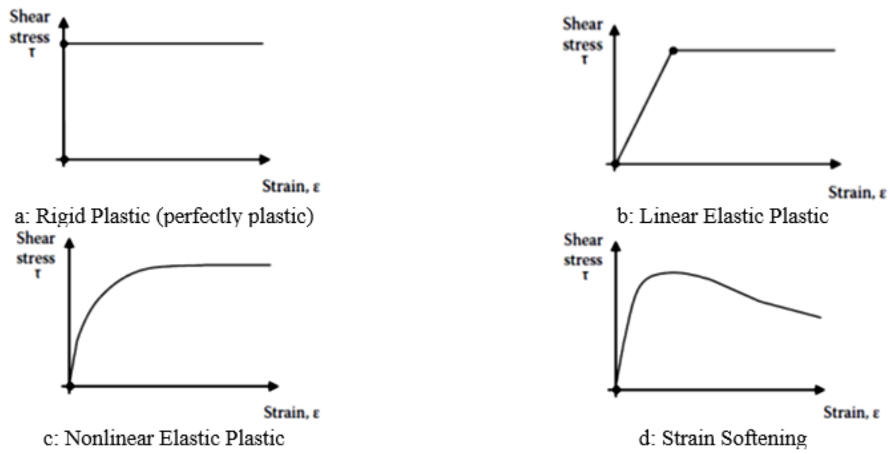


FIGURE 2. Different behaviors of stress and strain relation of soil models

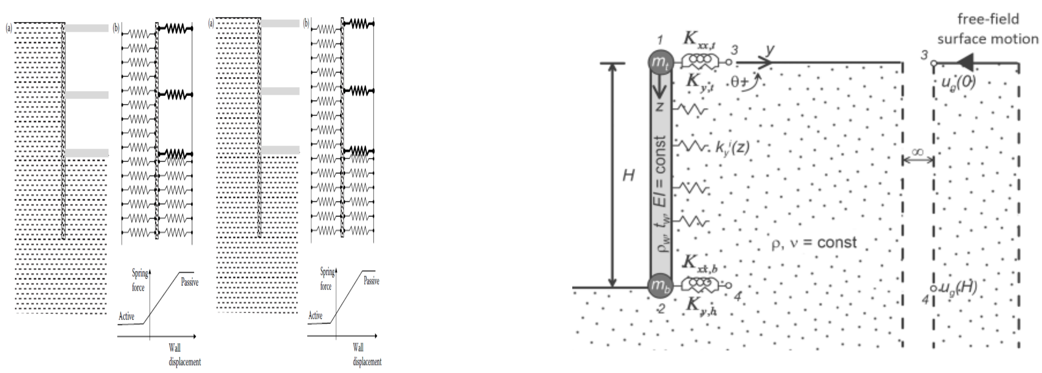
Source: Sørensen (2012); Clayton et al. (2013)

In the analysis of flexible ERSs, Winkler representation can be used for both static and dynamic loading, Figure 3. The Winkler spring model is frequently utilized when analyzing anchored or cantilever ERSs to calculate anchor or prop or anchor loads, bending moments, and shear forces. However, in situations where the geometry of the ground is too intricate to justify the use of full continuum analysis, this model is often preferred. According to the Winkler spring model, the pressure exerted on a RW depends on how far it moves horizontally away from or towards the soil, until it reaches the point of failure, either actively or passively. The finite element can be used to model the RW as a beam. To use the Winkler model, it is necessary to estimate the spring stiffness at various depths along the height of the RW (both sides). The displacement of a loaded area is primarily determined by its size and the stiffness and strength of the material it rests on. Thus, the distribution of overall stress and stress at a specific section of the RW can impact the wall's displacements. Typically, the soil spring stiffness remains constant during analysis. Yet, it's possible for the soil spring coefficient to vary depending on the changes in ground stress and strain during excavation. Nevertheless, it is not recommended to rely on this model for accurately predicting ground movement and surface changes that occur behind embedded ERSs (Sakamoto & Katsura 2012; Clayton et al. 2013; Essam 2018).

On the other hand, Brandenberg et al. (2020) found a solution to address the impact of shear waves (vertically propagating) on flexible ERSs in inhomogeneous viscoelastic or elastic soil. In their solution, they used a weak form of the motion governing differential equation linked with the Winkler representation of LEP. The model takes into account the relative displacement between the free-field soil and the RW, as well as various inputs such as RW flexural stiffness, distribution of mass along the RW, the boundary conditions (elastic) at the bottom and top of the RW, surface motion at the retained soil, and shear wave velocity profile.

The most basic connection that can be suggested is a direct correlation between stress and strain, indicating a steady proportionality between the increase in overall stress and the increase in strain. The elastic model is frequently utilized to estimate the ground and wall movements for embedded ERSs. This model is typically used in conjugation with the plastic behavior of soils. The elastic model can be used to model different soil conditions including nonhomogeneous stiffness, nonlinear elasticity, linear elastic, and cross anisotropic stiffness.

To use linear elastic models, one needs to estimate stiffness parameters. These parameters depend on various factors in natural soils, such as strain level, stress path, effective stress, and location. Also, if these models are to be used, it is necessary to estimate a characteristic strain. Such estimation can either vary depending on location and depth or be done for the entire ERS. While to create an isotropic linear elastic model, two stiffness parameters are needed: either Poisson's ratio and Young's modulus, or shear modulus and bulk modulus (Perloff & Baron, 1976; Wood 2004; Clayton et al. 2013). Nam & Thao (2013) stated that the deformation results from the numerical simulation of cantilever ERS are highly affected by the selected soil model. They found that the outcomes produced using the linear elastic model do not accurately represent the actual characteristics of soils, including the impact of stress state on the deformation. This is due to the linear model's utilization of fixed Young's moduli.



(a) (b)

FIGURE 3. Winkler representation of flexible ERS in A: static earth pressures, B: dynamic earth pressure
Source: Clayton et al. (2013); Brandenberg et al. (2020)

In even seemingly uniform soils, nonhomogeneous stiffness is typically necessary due to the fact that soil stiffness is dependent on effective stress, which augments with depth. The degree of stiffness increase with depth can be calculated through tests on soils at varying depths or by analyzing similar ERS types in comparable ground conditions. The following equation was proposed by Hooper (1973) to calculate the degree of stiffness increase with depth for London clay (z, in meters):

$$E = 10 + 5.2\, z \qquad (1)$$

Where
E : is Young's modulus (N/mm^2)
z : the depth (m)

On the other hand, soil layering can cause non homogeneity, which must be considered in any analysis to obtain reasonable estimates of ground movement. It's common for soils to have different stiffness and strength characteristics in each layer due to this layering. Based on laboratory tests done by Hetland (2015), it has been revealed that the soil's modulus of elasticity is directly proportional to the effective overburden pressure, following a power rule. This means that the modulus of elasticity of soil increases with depth, as demonstrated in Figure 4. It is worth stating that the linear elastic model used in numerical analysis with geotechnical software can only take into account a linear augment in stiffness as the depth increases. Accordingly, to accurately capture the soil modulus of elasticity, the soil profile can be divided into a number of subsequent layers as stated by Mohamed et al. (2020).

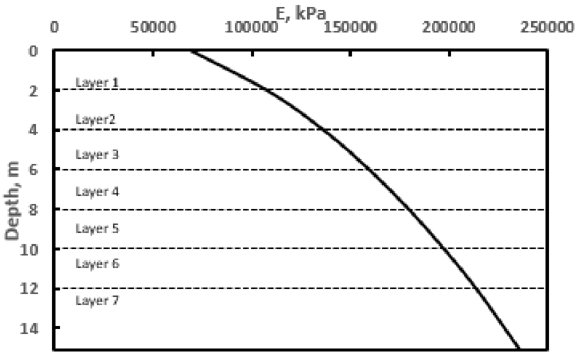


FIGURE 4. Soil layering effect on Young's modulus Source: Hetland (2015)

Simply put, one–dimensional deposition results in a stress and deformation anisotropy, which leads to anisotropy in particle arrangement and fabric. This, in turn, is expected to give rise to cross–anisotropic deformation properties (Wood 2004). Typically, sediments of sand and clay display anisotropic behavior as a result of gravitational deposition and consolidation, respectively (Beskou et al. 2018). Barden (1963) discovered that the cross–anisotropy type varies in different soils. The analysis of cross–anisotropic behavior in clayey soils is considered less reliable than in sandy soils.

Based on the results of Khaleghi et al. (2020), it has been found that cross–anisotropy significantly affects the response of RWs. Therefore, it is imperative to take this factor into account during the design process of ERSs. When analyzing embedded RW in heavily overconsolidated deposits, the cross–anisotropic stiffness is crucial. This is mainly due to the in–situ stresses near the ground surface. Before excavation, the in–situ horizontal stresses are higher than the vertical stresses, in other words, the horizontal stiffness is greater than the vertical stiffness. This, in turn, will reduce the strains in the horizontal direction. On the other hand, in order to establish a cross–anisotropic stiffness model, it is necessary to have the elastic modulus and shear modulus in both horizontal and vertical directions in addition to Poisson's ratio in the horizontal direction (Clayton 2011; Nguyen and Koseki 2019; Niemunis & Staszewska 2022). In the vertical direction, the measured value of Young's modulus is used, while it is estimated in the horizontal direction. However, the matrix's elements for cross–anisotropic stiffness can be determined using both static and dynamic data (Lings et al. 2000).

Other models frequently employed in many countries are the non–linear elasticity models. They are employed in numerical modeling especially when ground movement close to ERSs is crucial. It is well recognized that the pre–failure behavior of stiff soils is inelastic and very non–linear from the first loading stages. In addition, analysis of the strain created in the soil ground during engineering construction demonstrates that geotechnical constructions function under working loads only within this non–linear range. Consequently, it is vital to take into account the behavior of the soil within this range of strains (Figure 5) in order to produce correct estimates of the movement (Grammatikopoulou et al. 2008). According to Figure 5, it is clear that, under working condition, for ERSs, the typical range for strains are from 0.01% to 0.1%. Grammatikopoulou et al. (2008) stated that the selection of a typical strain level for the derivation of an appropriate linear elastic pre–failure stiffness is difficult. Gaba et al. (2003) proposed a maximum value of 0.1% for the increment of shear strain for ERSs, as these structures experience little deflections.

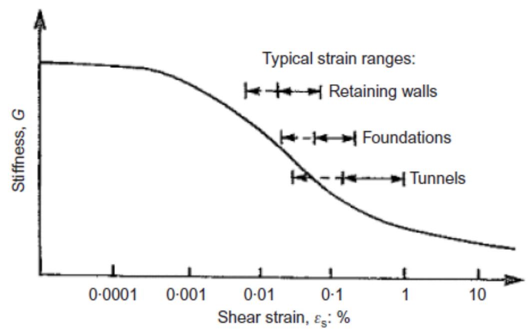


FIGURE 5. Strain ranges (typical values) for different geotechnical structures, and typical stiffness vs. strain behavior (for stiff clay)
Source: Mair (1993)

The observed nonlinearity in the behavior of soils is typically a sign of plasticity. Once the elasto–plastic characterizations of soil non-linearity are presented, turning to finite element numerical analysis with the assistance of a computer becomes necessary. Both cohesive and non-cohesive soils can be described using "elasto–plastic models". However, more complicated models have been required to successfully simulate the behavior of sand in comparison to clay. Furthermore, while dealing with stiff clay, it is important to pay close attention to the nature of the expected elastic response (Wood, 1991a). A majority of geotechnical modeling software regularly contains numerous failure and yield functions. In general, knowing that soils have yield surfaces means that the reaction to changes in stress within a current yield surface is elastic. When a stress shift interacts with a current yield surface, both elastic and plastic responses happen. Choosing the type of plastic deformations is crucial (the magnitudes of different plastic deformation components, the relative magnitudes of these components, and the relation between the deformation component and the yield surface changing size (Wood 1991b).

The stresses used in the analysis of these programs are in 3-D, which requires 3-D space to work. However, the analysis may be simplified into a plane strain or a 2-D problem. There are a number of failure criteria used to model the failure conditions in geotechnical applications, these are "Mohr–Coulomb criteria", "Tresca criteria", and "Modified Tresca criteria". Additionally, for drained conditions, volumetric strain effects can be modeled using "CamClay" (work-hardening plasticity). Since only a little volume of the soil will be yielding, the using of plasticity in the model should have a negligibly minor impact on the majority of ERSs for which ground movements are a concern. Since estimations of the effective strength parameters (cohesion and internal friction) are typically made as part of any ERS design, the use of a Mohr-Coulomb criterion is practical (Clayton et al. 2013).

The constitutive relations of the behavior of soils were molded numerically in numerous models defined in the literature. The presentation and development of hypotheses and principles of the complete equations for the constitutive models are seeming daunting, although it appears conceptually rather simple. The calculations of geotechnical designs need an aware selection of models of soil behavior. Some calculations (as in bearing capacity) consider that the behavior of soil is perfectly plastic and rigid, while other ones (like settlement calculations) consider the linear-elastic behavior. Actually, it is not likely for the perfect plasticity condition to be attained under working loads for geotechnical constructions, for the same loading condition, these constructions will surely have proceeded well beyond the range of linear-elasticity. In selecting an appropriate model, it is important to consider the effect of soil history and the changes in stress that may the soil experience in the future. However, the elaboration required for the model is related to the number of effects to be considered. Among these effects are the soil nature, the type of loading, the location project, and the site investigation quality. With consideration of more effects, more parameters of soils are needed to identify the model, and as a result, the more complicated the tests that are required to obtain their values become (Wood 1991c).

BACKFILL QUALITY EFFECTS ON ERSS

The behavior of ERSs is a function of many factors including the properties and conditions of backfill soils. As a matter of fact, the type of backfill materials significantly impacts the values of movement required to reach the plastic state of equilibrium (Mikola & Sitar 2013). ERSs backfilled with cohesionless materials require considerably less lateral movement to reach the plastic equilibrium in comparison to cohesive backfill materials (Das & Sivakugan 2019). The relative state of the cohesionless backfill highly affects the LEP coefficient value. Dense backfill soil provides more internal friction than loose material, this, in turn, implies a higher passive coefficient (kp) and lower active coefficient (ka). Accordingly, dense backfill applies less LEP to RW subjected to an active state than loose backfill, as the dense material is more stable and capable of supporting itself. Well-knowing of the engineering properties of the backfill soil is essential to assure the ability of the RW to sustain the augmented loading (Al-Taie 2013; McCombie et al. 2016; Holtz et al. 2023).

According to the quality of backfill materials, and based on the literature reviewed, ERSs can be divided into two main categories. The first category includes the group of RWs that require large quantities of high-quality backfill materials, and the second category includes ERSs that require small quantities of high-quality materials as backfill soil. In general, mechanically stabilized RWs require high-quality backfill materials, while concrete RWs (including concrete cantilever RW, concrete gravity RW, and concrete counterfort RW) require small quantities of high-quality materials in comparison to the mechanically stabilized types. Actually, the site-specific cost of mechanically stabilized RWs is highly affected by the quality of backfill materials. The backfill materials affect the selection of reinforcement type and the performance of the facing units. Furthermore, the quality of the backfill material impacts the soil interaction with the reinforcement. Therefore, it is necessary to confirm the suitability of the backfill soil by conducting a series of laboratory tests including grading, electrical resistivity, etc. (Chonkar 2001; INDOT 2012).

From an economic point of view, the cost of backfill soil represents one of the major components of total cost for some ERSs. There are some requirements (e.g. permeability, shear strength, gradation, etc.) are specified in different standards regarding the properties of backfill soils for particular ERS. Soils, even native soils, that meet these requirements can be considered as backfill materials for RWs. In fact, the availability of good quality backfill soil in abundance close to a specified construction area is questionable. However, there are exceptions that can be adopted according to the conditions of the site and based on the project specifications (Al-Taie 2011; Al-Taie & Mohammed 2014; Chauhan & Dasaka 2022).

In engineering practice, the predominated backfilling soils are cohesionless materials with moderate fine content (silt and clay). In reinforced soil ERSs, soils with less than

25% fine materials are considered suitable backfill. According to the literature, for a good performance free from trouble, the RWs must be backfilled with a high-quality backfill material (well-graded cohesionless soil of high shear resistance and free drainage). Also, in some cases, it is possible to recycle waste stone (including fragments of gravel-sized) into the backfilling behind drystone gravity RWs. In reality, it is not preferred to use cohesive soils as backfill materials, in cases where cohesive soils cannot be avoided, high-quality backfills must be supplied at least for the area directly at the back face along the entire length and depth of the wall (Kaniraj, 1998; PWRC 2003; WSDOT 2009; Suzuki et al. 2015). It is important to note that the LEP from cohesive backfill soils is higher in comparison to non-cohesive backfill. Accordingly, the minimum LEP values proposed in international standards (e.g. ASCE /SEI 7-10) and literature (e.g. Brooks, 2010) in the design of ERSs are higher for the cohesive group of soils (silt and clay) than the non-cohesive soils (gravel and sand) as shown in Figure 6.

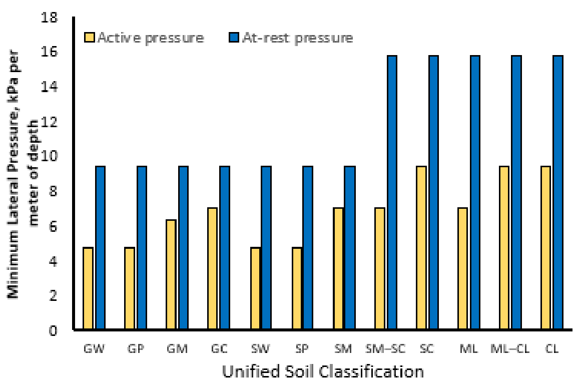


FIGURE 6. Minimum design lateral earth pressure
Source: Brooks (2010; ASCE/SEI 7-10)

Observations of case histories recorded by researchers showed that there is a critical effect of the grain size of backfill materials on the design of the ERSs. Casagrande (1973) noted that if the grain size of the granular backfill material behind the anchored type sheet pile RWs is predominantly finer than the coarse sand grain size, then, the LEP may change from active state to at-rest state after construction. This can cause an increase in the value of LEP with time, as a result, the force in the anchor is largely increased as shown in Figure 7. Also, field observations for anchored sheet pile walls backfilled with fine sand revealed the recorded values of lateral active pressure from field observation are higher than that predicted from theoretical approaches (Das & Sivakugan 2019). The type of backfill soil can affect the pressure distribution behind a RW. It demonstrated in studies the impact of backfill soil properties on the distribution of lateral soil pressure and the moment on the walls, especially under dynamic conditions. Backfill soils with adequate strength have a yielding acceleration of about 0.8 g. Below this acceleration, e.g. for seismic active pressure on a rigid RW, the formation of the failure wedge is precluded. Also, it was recorded that, below 0.8g acceleration, the backfill soil interaction with wall motions is influenced by the dynamic response of the ERS (Alampalli & Elgamal, 1990; Veletsos & Younan, 1997; Dewoolkar et al. 2001; Gazetas et al. 2004; Wilson & Elgamal 2015).

Finally, the fact of depletion of clean granular non-cohesive materials is of important concern for engineers in different engineering applications. Anyway, this led to exploring and examining the alternatives as filling materials from locally available resources. Among the explored materials are naturally occurring aeolian and sedimentary soils, byproduct materials from industries, and various waste materials from construction and domestic wastes (Al-Taie 2002; Al-Taie et al. 2013; Al-Taie & Al-Shakarchi 2016 2017; Li et al. 2020; Al-Taie et al. 2020; Jelani,et al. 2021; Onyelowe et al. 2021; Al-Yasir & Al-Taie 2022; Prajapati & Rangwala 2022; Ramli et al. 2022; Shaikh et al. 2022)

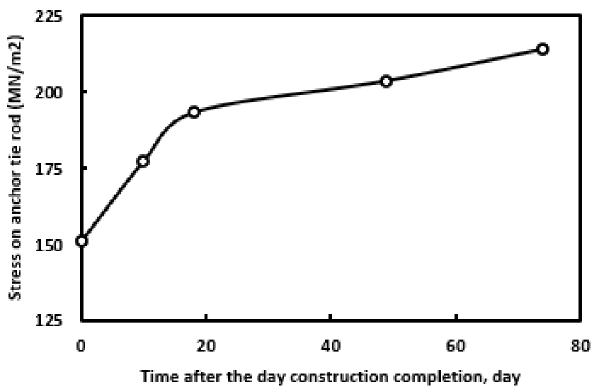


FIGURE 7. Variation of stress on anchor tie rod of anchored sheet pile backfilled with fine sand
Source: Modified after Casagrande (1973)

EFFECT OF TOLERATED SETTLEMENT ON ERSS

The design of ERSs includes the calculation of possible deformations related to serviceability limit design states. These deformations are related to the foundation soil's settlement, and/or the settlement of soil in the wall itself (for the case of internal stability walls like reinforcement soil RWs). In design, the tilt failure is critical for the external stability of ERSs in the ultimate limit states. ERSs exhibit different abilities to sustain deformations. The selection of a suitable retaining system to fit the design requirements is directly concerned with settlement issues (both the total and differential settlements). The considerable differential settlement at the foundations of the RWs may cause large deformation for these walls. Due to the inherent flexibility of some types of retaining systems (e.g. mechanically stabilized RWs, prefabricated modular RWs, and reinforced slopes), these systems can tolerate a considerable settlement without structural damage. While the types of RWs that are inherently rigid cannot sustain settlements or can sustain a very low settlement (Koseki et al. 2006; Dean & White 2010).

RWs can be classified according to their abilities to sustain settlement into four main categories, these categories are "RWs cannot tolerate settlement", "RWs can tolerate little settlement", "RWs can tolerate moderate settlement", and "RWs can tolerate large settlement". The group of ERSs that cannot tolerate settlement includes mortar-rubble masonry RWs, and rockeries or rock RWs. While a little settlement is tolerated in some types of reinforcement soil RWs like steel reinforced-soil RW (with concrete face). Also, RWs like concrete cantilever walls and concrete counterfort RWs cannot tolerate much settlements. Moderate settlement is tolerated in concrete bin RW and gabion RW. While the allowed settlement of the group of crib walls (concrete, metal, and timber), the steel reinforcement soil RWs (with modular concrete panels), reinforcement soil segmental RWs, and reinforced soil slopes are large. It is worth noting that there are some types of RWs that can tolerate large short-term settlements but they cannot tolerate large long-term settlements, such as steel reinforced-soil RW (with welded wire and concrete panels, steel reinforced-soil RW with welded wire facing, and geosynthetic RW with a concrete face). Figure 8 shows the variation of tolerating differential settlement for different types of retaining walls. Each value plotted in Figure 8 was calculated from the net vertical settlement for two selected points on the RW divided by the distance between them (Dean & White 2010; INDOT 2012; McCombie et al. 2016).

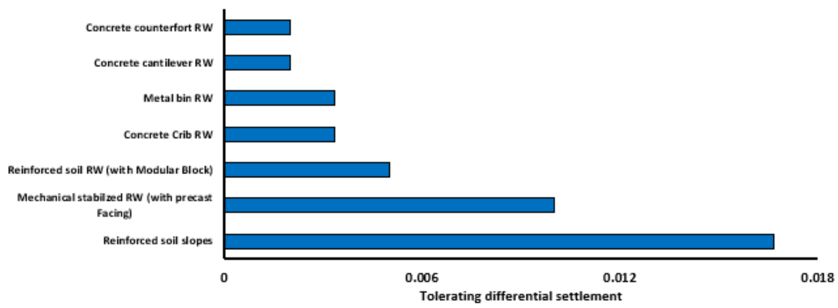


FIGURE 8. Tolerating differential settlement
Source: Dean & White (2010); INDOT (2012)

Considering the settlement toleration in the design of the retaining walls aims to ensure the good performance of these walls without damage. For example, the settlement for mechanically stabilized RWs impacts the selection of the wall's facing units. Flexible-facing units (like geosynthetic and welded wire facings) sustain more settlement in comparison to lesser flexible units (like concrete facings). Also, in rock walls or rockers, much settlement can cause shafting and falling of the rock. In addition, the selection of RWs is controlled by the properties of the foundation soil and its profile. Soft ground or very deep weak ground cannot support the retaining system without shear failure of excessive settlement. In such a case, using a pile foundation and/or anchor with a suitable RW (can be supported by piles e.g. concrete (cantilever or semi-gravity) RWs, or anchored RWs) is the appropriate solution. The susceptibility of ERS to settlement can be improved by implementing a variety of methods depending on the expected magnitude of the settlement. Improving the foundation soil using a suitable technique (e.g. stone columns, soil replacement, dynamic compaction, wick drains, and rigid inclusions), selecting lightweight backfilling soils, and increasing the tolerated settlement of the walls are among these methods. However, the best method selection depends on the height of the RW, the availability of backfill materials, and the constraints of construction time (Koseki et al. 2006; Al-Busoda et al. 2017; RECo 2020). On the other hand, it was proved in different studies that the lateral displacement for some ERSs, like mechanically stabilized RWs, is highly controlled by the length and spacing of the reinforcement materials. Actually, the whole retaining system displacement can be reduced to a limited value significantly as these parameters increase. However, the displacement of ERSs, even the mechanically stabilized RWs, is negatively affected under seismic loading (Latha & Manju 2016; Wu et al. 2017; Zhu et al. 2022).

ERSs are constructed either with a "horizontal backfill slope" or with an "inclined backfill slope". The "inclined backfill slope", in turn, is either "planer or sloping" or "broken" (Murthy 2003). It was verified in the literature that the angle of inclination of the backfill greatly affects the design of ERSs for both static and dynamic loading. Where the value of this angle must be lesser than the angle of internal friction of the backfill material behind the RW (McCombie et al. 2016; Sun et al. 2022; Holtz et al. 2023). There is an essential relation between the earth pressure coefficients and the angle of inclination of the backfill material. Figure 9 shows values of LEP coefficients (for active and passive states from Rankine method) for combinations of internal friction of soil and backfill slope angle. As provided, when the slope angle increases, the active earth pressure coefficient (K_a) increases. If the other parameters remain constant, this means that the increase in the backfill slope angle causes an increase in LEP that has to be exerted on ERS. More deformation is required to reach the plastic state of failure in the case of ERSs that are constructed with horizontal backfill slopes in comparison to inclined backfill. In reality, the inclination of backfill introduces more soil mass and this will cause the backfill material to reach the unstable state much sooner, as a result, the soil mass will fail with smaller deformation. In such a state, the Mohr circle will not become bigger as much before the condition of failure, and this means lateral earth pressure will be greater at failure, corresponding to the mentioned higher values of K_a.

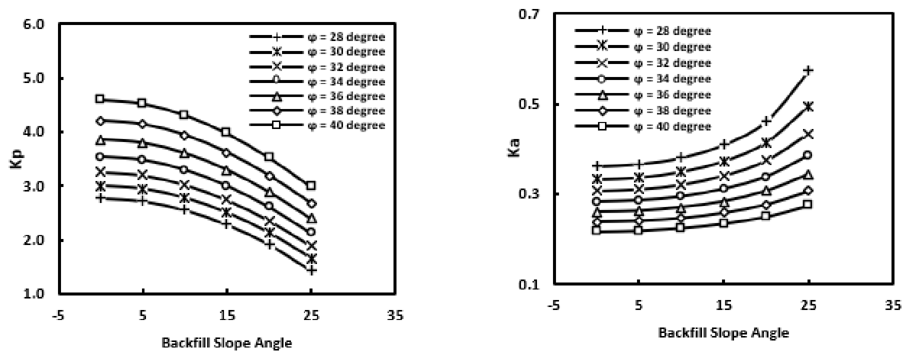


FIGURE 9. Variation of lateral earth pressure coefficients with backfill Slope angle
Source: Modified after Holtz et al. (2023)

The inclination of the backfill materials is affecting both the distribution of LEP and the value of the resultant force behind the RW. Inclined backfill produces inclined LEP distribution and resultant force, where the inclination of these two components is assumed to be the same as that of the backfill slope. Actually, in the case of the inclined backfill slope, the alignment of the resultant earth pressure force will be closer to the plane of failure, and this in turn will make the resultant force more efficient at enhancing failure over that plane.

There are some approximations that can be adopted for practice purposes, e.g. for inclined backfill with broken slope shown in Figure 10, the simplest method to obtain the LEP diagram behind cantilever walls is to calculate the total force and full height (measured from the bottom of the excavation to top of the slope, H_1), then they are applied to the height of the wall, H_2 (Turner 2009).

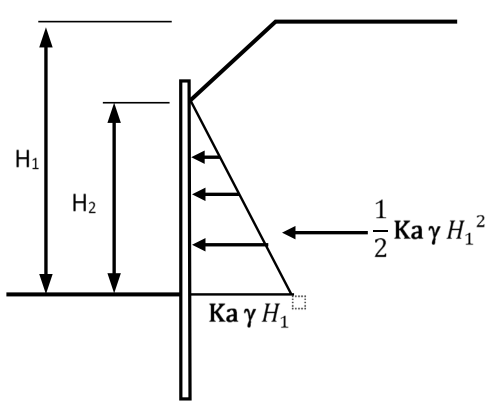


FIGURE 10. LEP distribution for cantilever RW with sloped bank
Source: Modified after Turner (2009)

It is worthily to mentioned that there is a case in which the inclination of the backfill slope is not downward to the wall, but is in the upward direction, i.e. it is not above the horizontal slope, it is below it. Such a case can be well noted in the approaches to the bridges. This case produces lesser ka values and, of course, lesser lateral earth pressure (Holtz et al. 2023). On the other hand, the angle of inclination of the backfill material has a main effect on the earth pressure coefficients of dynamic loading conditions, Kae, as shown in Figure 11. As can be seen in Figure 11 the K_{ae} is highly influenced by the slope of backfilling, the implications of the angle of inclination of the backfill material on the design are very clear. For example, for horizontal acceleration, k_h =0.3, increasing the backfill slope from 0 to 10 degrees can cause approximately a 100% increase in the value of K_{ae}, which in turn causes a doubling of the dynamic component of the soil thrust (Seed & Whitman, 1970; Elms & Richards, 1979; Brooks & Nielsen 2013).

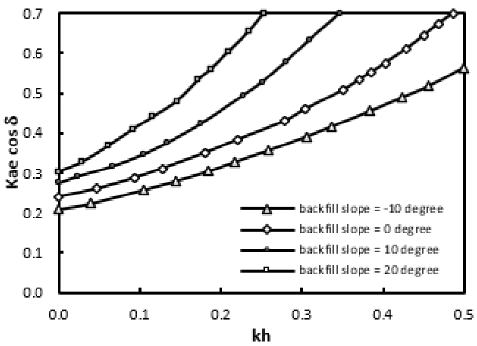


FIGURE 11. Variation of dynamic lateral earth pressure coefficient with backfill slope angle
Source: Modified after Seed & Whitman (1970)

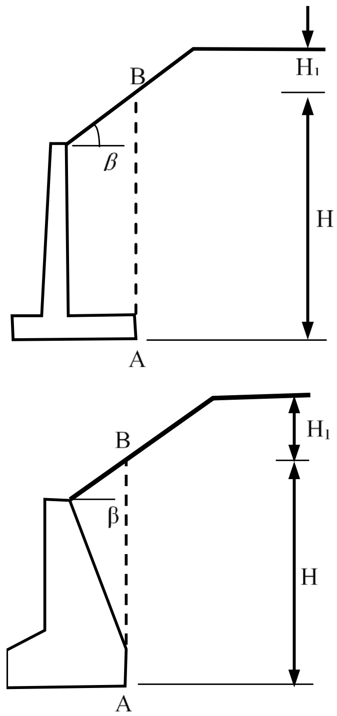


FIGURE 12. Cantilever and gravity RWs with sloped backfills surface
Source: Murthy (2003)

TABLE 1. Types of Backfill for Retaining Walls

Type number	Soil description	Soil properties	Maximum slope angle (horizontal to vertical)	
			with plane surface	with break surface
1	Coarse-grained cohesionless soil without admixture	Free drainage clean soil (sand or gravel)	1.5:1	1.5:1
2	Coarse-grained cohesionless soil with an admixture	Low permeable soil with silt-sized admixture	1.5:1	1.5:1
3	Granular material with clay, and fine silty sand residual soil (with stones)	-	1.5:1	1.5:1
4	clay, silty clay, or organic silts	Soil with very soft to soft consistency	3:1	3:1
5	Clay soil	Soil of medium stiff to stiff consistency	2:1	2:1

Source: Modified after Terzaghi et al. (1996)

The angle of inclination of the backfill materials behind RWs is contorted by factors like type and height of ERS, type of backfill soil, the available backfilling space, water pressure, seismic activity, and site restrictions (INDOT 2012; Sun et al. 2022). The inclination angle of the backfill can be greatly influenced by the type and height of the RW. For example, Figure 12 presents cantilever and gravity RWs supporting upward-sloped backfill surfaces. As shown, there is a clear difference between the magnitude of height H_1, it is higher in the case of the gravity wall. This indicated that gravity walls can be constructed with a steeper backfill slope in comparison to the slope in the case of cantilever walls. Substantially, the backfill slope for shorter walls can be steeper than the backfill slope of taller ones. However, for installing backfill soils behind cantilever and gravity RWs, the angle of slope of inclination should be not more than 1 horizontal to 1 vertical, otherwise, the methods adopted in the design of the RW should be more vigorous (Murthy 2003; INDOT 2012).

The second important factor that affects the angle of inclination of the backfill materials is the properties of the backfill soil. Different soil properties were presented by Terzaghi et al. (1996) to represent a variety of backfill soils behind RWs as shown in Table 1. Five types of soil were presented with different properties ranging from clean free drainage cohesionless soil to impermeable stiff cohesive soil. According to Terzaghi et al. (1996), a semi-empirical method to estimate the lateral earth pressure on RWs with different backfill materials was developed with design charts. These charts were developed to design concrete RWs with inclined backfill slopes of different backfill surfaces (plane surface and broken surface). The maximum values of the slope angle (horizontal to vertical) from these charts are shown in Table 1, it is clear that RWs backfilled with cohesionless soils can be constructed with a steeper backfill slope (plane and break surfaces) in comparison to RWs that backfilled with cohesive soils. A reexamination of Table 1 shows that retaining walls with cohesive soil of stiff consistency (higher shear strength) can support a steeper slope in comparison to that with soft cohesive soil

The available backfilling space behind RWs has a direct impact on the angle of inclination of the backfill slope behind RWs. Obtaining more backfilling space implies the angle of inclination of the backfill slope can be shallower. The pressure on the wall reduces when a large area is available to spread out the backfill soil, this allows a flatter inclination. Additionally, the available backfilling space behind RWs has an effect on the type of inclined backfill slope as it is directly related to the height of the RW. A broken inclined backfill slope may be utilized behind the RWs that supported inclined backfill with a break at a horizontal distance of less than twice the wall height, otherwise, a horizontal backslope can be used (INDOT 2012). Also, both the failure surface and failure mode are affected by the backfill space. Wider backfill space allows full development of the failure surface. The potential of rotation of the ERS increases with a decrease in the backfilling space (Muktinutalapati & GuhaRay 2021).

The water pressure behind retaining walls and seismic activity can have a significant effect on the angle of inclination of the backfill slope behind the wall. The water pressure can cause the backfill slope to become steeper, as the pressure pushes the soil outward. This causes the backfill slope to become unstable, leading to a potential failure of the wall. To prevent this, the backfill slope should be designed with a shallow angle of inclination to reduce the effects of water pressure. Also, the seismic activity in the area can also affect the angle of inclination of the backfill slope. Construction of RWs in areas with higher seismic activity requires flatter backfill slopes than in areas with lower seismic activity. In areas with seismic activity, the effect of seismic loading on the angle of inclination of backfilling of RWs is highly dependent on the type of backfill soil. In general, it is thought that the active earth pressure coefficient will highly decrease with the presence of cohesion in backfill soil, (Wilson & Elgamal 2015; Osouli & Zamiran 2017; Zamiran & Osouli 2018). Authors like Al-Atik & Sitar (2010) showed that with inclined cohesive soil backfill behind the RWs, the value of the dynamic coefficient of earth pressure is much greater amount than its value in the case of horizontal cohesionless backfilling. This in turn demonstrated that there is a significant influence of the angle of inclination of backfilling of the wall on the value of the dynamic coefficient of earth pressure. In point of fact, the state in which a backfill slope is made behind a retaining wall is perhaps the most occurring in civil engineering practice. Furthermore, in relation to horizontally backfilled materials, more considerable residual bending moment and residual deformation are caused in walls with inclined backfill slopes behind the wall. This being is due to the possibility of a downward slide and the deformation of backfill slopes under the excitation of the earthquake (Sun et al. 2022).

CONCLUSION

Earth retaining structures, ERSs, are used in many engineering fields including architectural, coastal, bridge, and road engineering. Special considerations and technical knowledge in structural and soil mechanics should be adopted in the modeling, analysis, and design of ERSs'. Some of these considerations are related to the soil models, and (quality, settlement, and inclination) of the backfill and foundation soils of the retaining wall, RW, as presented in this critical review.

The calculations of geotechnical designs need an aware selection of models of soil behavior. In selecting an appropriate model, it is important to consider the effect of soil history and the changes in stress that may the soil experience in the future. However, the elaboration required for the model is related to the number of effects to be considered. Among these effects are the soil nature, the type of loading, the location project, and the site investigation quality. With consideration of more effects, more parameters of soils are needed to identify the model, and as a result, the more complicated the tests that are required to obtain their values become.

The adoption of "rigid plastic" models in the estimation of passive LEP can result in a substantial overestimation. The use of the linear elastic model does not accurately represent the actual characteristics of soils, including the impact of the stress state on the deformation. On the other hand, the cross–anisotropy significantly affects the response of RWs. Therefore, it is imperative to take this factor into account during the design process of ERSs.

The design for some types of ERSs is highly affected by the quality of backfill materials. As the backfill materials affect the selection of material type and the performance of the ERSs, furthermore, they impact the soil interaction with RW. Therefore, it is necessary to confirm the suitability of the backfill soil by conducting specified experimental tests (permeability, shear strength, gradation, etc.). In reality, it is not preferred to use cohesive soils as backfill materials, in cases where cohesive soils cannot be avoided, high-quality backfills must be supplied at least for the area directly at the back face along the entire length and depth of the RW. Also, the design of some anchored ERSs with backfill of predominantly finer than the coarse sand grain size should be conducted with precautions due to the possibility of LEP changing from active state to at-rest state after construction.

The selection of a suitable ERS to fit the design requirements is directly concerned with settlement issues (both the total and differential settlements). The effect of settlement on the design of the earth system is related to the flexibility of ERSs themself and the magnitude of their tolerating settlement. It is worthwhile to consider both short-term and long-term settlements in the design of ERSs as there are specific types of these systems that can tolerate large short-term settlements but cannot tolerate large long-term settlements. Finally, the design of the retaining system is affected by the allowable limits of displacement, which in turn is negatively impacted under seismic loading.

The angle of inclination of the backfill soil greatly affects the design of ERSs (for both static and dynamic loadings). More deformation is required to reach the plastic state of failure in the case of active stress on ERSs that are constructed with horizontal backfill slopes in comparison to inclined backfill. The angle of inclination of the backfill materials affect both the distribution of LEP and the value of the resultant force behind the RW (under dynamic and static loadings). This angle, however, is contorted by factors like the type and height of ERS, type of backfill soil, available backfilling space, water pressure, seismic activity, and site restrictions.

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DECLARATION OF COMPETING INTEREST

None

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