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Effect of Earth Pressures and Shear Key on the Structural Stability of Gravity Retaining Wall Supporting Anisotropic Soil

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ABSTRACT

Constructing a retaining wall is very expensive. The cost is directly proportional to the cost of different elements like volume of excavation and concrete. Thus, it is very important for engineers to design optimum and safe retaining walls. The objectives of this study are to evaluate the implications of the two classical lateral earth pressure theories on gravity retaining walls and to determine the effect of shear key depth and location (toe, stem and heel) on the general stability of the wall. The earth pressures acting on a retaining wall supporting an anisotropic cohesionless soil was estimated by the earth pressure theories proposed by Rankine and Coulomb and compared with that obtained by finite element method (FEM) using Plaxis 2D. Results revealed that the method proposed by Coulomb yielded higher active and passive lateral earth pressures than that of Rankine. The values provided by Rankine method are constantly below those of FEM. A peak active and passive earth pressure values of 92.3 kN/m² and 3254.95 kN/m² were obtained. It was observed that the depth of shear key is significant for safety factor against sliding. At wall friction angle factor of 0.6, only the situations with shear key depths of 1.6 and 2.0 m met the minimum safety factor against sliding of 1.5. It was observed that positioning the shear key at the heel or at any position between the stem and the heel is the best practice which was recommended to enhance higher safety against overturning failure of the wall.

1. INTRODUCTION

Soils in their natural state exhibit a certain degree of anisotropy due to stratification associated with soil forming processes such as sedimentation, illuviation, compaction and particle orientation (Nishimura, 2005). Clay deposits, during their formation by sedimentation and subsequent one-dimensional consolidation acquire a fabric that is characterized by particles or particle-units oriented in a horizontal arrangement (Salahudeen et al., 2017). This preferred orientation and the resulting electrochemical bonds among the clay particles are the cause of cross-anisotropic deformational behaviour in clays (Salahudeen et al., 2014). In sands however, the deformational anisotropy arises chiefly from the influence of gravity and particle shape on the deposition process. Experimental investigations by Gazetas (1981) have revealed that sand particles have a strong tendency to adopt preferential orientation with the maximum dimension aligned in a horizontal plane. The isotropic plane in soil deposits frequently coincides with the deposition plane, and its perpendicular direction is often called the axis of anisotropy (Gao and Zhao, 2012).

In addition to the analytical solutions, Finite Element Method (FEM) solutions have also been presented recently for anisotropic elastic media (Tekinsoy et al., 2009; Salahudeen, 2018). The finite element method is one of the mathematical methods in which continuous media is divided into finite elements with different geometries (Salahudeen and Sadeeq, 2017). In this study, the soil was modelled using an elastic-perfectly plastic constitutive model, with the Mohr-Coulomb envelope defining the yield criterion. In classical soil mechanics, plasticity theory has been used for calculating the distribution of lateral earth pressure, which is the central issue in the analysis of earth retaining structures. Osman and Bolton (2004) observed that in this theory, a zone of soil is assumed to reach plastic equilibrium such that plastic collapse occurs when the plastic soil zone slips relative to the rest of the soil mass. The peak soil strength is assumed to be mobilized on the slip surface. The collapse load is then calculated and factors of safety are introduced to allow for uncertainties and to limit movements by ensuring that the stresses are far from their ultimate values. Conventionally, the ratio of resisting forces/moments to the disturbing forces/moments is the factor of safety. This factor of safety should always be greater than 1.5 (for cohesionless soils and 2.0 for cohesive soils) for the retaining structure to be safe against sliding failure or at least 3.0 for safety against overturning failure as a design criterion.

Retaining walls are structures used to retain earth or any other material in a position where the ground level changes abruptly. Retaining walls are also built for aesthetic landscaping purposes. The gravity wall considered in this study generally consists of a vertical stem, and a base slab, made up of two distinct regions, viz. a heel slab and a toe slab. All three components behave like one-way cantilever slabs: the stem acts as a vertical cantilever above the lateral earth pressure; the heel and the toe slabs act as horizontal cantilevers under the action of the resulting soil pressure. The self-weight of the concrete wall together with the weight of the earth on the heel help in maintaining the stability of the wall. They can be of many types such as gravity wall, cantilever wall, counterfort wall and buttress wall among others. Gravity walls are the oldest and simplest type of retaining wall. Gravity walls are heavy, thick and stiff enough that they do not bend. Their movement occurs essentially by rigid body translation or rotation (Yadav et al., 2018). The lateral force due to earth pressure is the main force that acts on the retaining wall which has the tendency to bend, slide and overturn it (Sharma and Baradiya, 2014). The basis for determining the magnitude and direction (position) of the earth pressure are the principles of soil mechanics. The behaviour of lateral earth pressure is similar to that of a fluid, with its pressure magnitude increasing nearly linearly with increasing depth for moderate depths below the surface.

In determining the external stability of retaining walls, failure modes like bearing failure, sliding and overturning are normally considered in design. Overturning occurs because of unbalanced moments, when overturning moment about toe due to lateral pressure is larger than the resisting moments of the self-weight of walls and weight of soil above the heel slab. The resistance against sliding is essentially provided by the friction between the bottom surface of the base slab and the soil beneath it. Resistance provided by the passive earth pressure on the front face of the base gives some contribution, but since this material is often backfill against the face, its resistance cannot be guarantee and is usually ignored. If the wall is found to be unsafe against sliding (usually high walls, H>4m) and overturning, shear key below the wall base is provided (Sarath et al., 2011). In considering the criterion of sliding, the sliding resistance of retaining walls is derived from the base friction between the wall base and the foundation soils. To increase the sliding and overturning resistance of retaining walls, other than providing a large self-weight or a large retained soil mass, shear keys are to be installed at the wall base. A shear key is an optional structural element installed at the bottom of the base slab. It is used to provide extra safety against sliding.

Gravity retaining walls with shear key provide long-standing stability and serviceability. The principle of shear key is such that it increases the passive resistance developed by the height of shear keys. However, active pressure developed by shear keys also increases simultaneously but disproportionately. The success of shear keys lies in the fact that the increase of passive pressure exceeds the increase in active pressure, resulting in a net improvement of sliding resistance. On the other hand, friction between the wall base and the foundation soils is normally about a fraction of the angle of internal resistance of foundation soil. When a shear key is installed at the base of the retaining wall, the failure surface is changed from the wall base/soil horizontal plane to a plane within foundation soil. Therefore, the friction angle mobilized in this case is higher than a situation of no shear key and therefore a higher sliding resistance can be enhanced.

Retaining wall is widely use in slope stabilization at residential area; mountainous road and highways; bridge abutment; river walls and coastal region. The cost of construction of retaining wall is very expensive. Thus, it is very important for engineer to design optimum and safe retaining walls. The cost of the wall structure is directly proportional to the volume of different quantities like volume of excavation, filling and concrete. Thus, the optimum cost of construction depends on an optimum design. The objectives of this study are to evaluate the implications of the two classical lateral earth pressure theories on gravity retaining walls and to determine the effect of shear key depth and location on the general stability of the wall. In order to reach conclusions of the widest possible applicability with actually encountered soil deposits, this study idealizes soil as a layered medium, with each layer being a linearly elastic cross-anisotropic continuum having a vertical axis of symmetry.

2. MATERIALS AND METHODS

2.1. Problem Description

A gravity retaining wall with dimensions shown in Figure 1 was analysed in this study. The finite element model used for the numerical modelling in Plaxis is shown in Figure 2 while the material properties of the layered cohesionless soil and gravity retaining wall concrete are given in Table 1. The soil obtained from a trial pit by disturbed method naturally exist in layers with distinct geotechnical properties which is the cause of the anisotropy. The earth pressure distribution generated behind the gravity retaining wall supporting an anisotropic cohesionless soil was estimated by the classical lateral earth pressure theories proposed by Rankine (1857) and Coulomb (1776) and compared with that obtained by finite element method using Plaxis 2D numerical modelling program. The active and passive lateral earth pressures together with the estimated factors of safety were plotted against varied wall friction angle (δ) factor values. The wall friction angle (δ) values were obtained by multiplying an average value of the soils internal friction angle (ϕ) by certain arbitrary factors of 0.6, 0.8, 1.0, 1.2, 1.4 and 1.6 (i.e., $\delta = 0.6\phi$, 0.8ϕ, 1.0ϕ, 1.2ϕ, 1.4ϕ and 10.6ϕ).

In practice, the shear key is placed an any position in the base of the wall. It is a common practice to put the key beneath the stem in cantilever walls for convenience of extending the stem reinforcement through the base and directly into the key. However, for gravity retaining walls that has no reinforcements, there is need to determine the best location for a shear key in order to achieve the highest possible safety factor against sliding which is the actual aim of installing the key. In this paper, the effect of shear key against overturning failure was equally examined. Shear key has been considered in three locations herein: below the heel, stem and toe. A situation of no shear key was also sufficiently examined. The dimensions of the shear key considered are width of 0.5 m and varied depth of 0.4, 0.8, 1.2, 1.6 and 2.0 m.

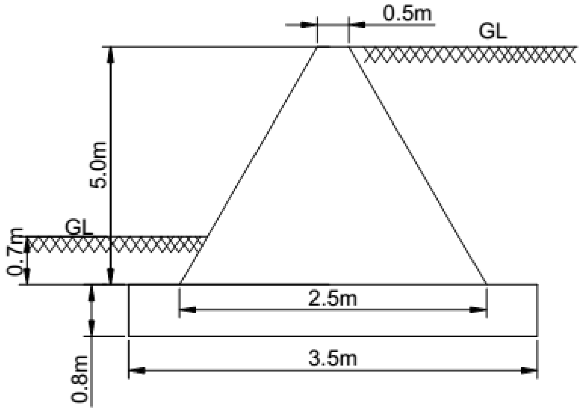

Figure 1: Dimensions of the gravity retaining wall

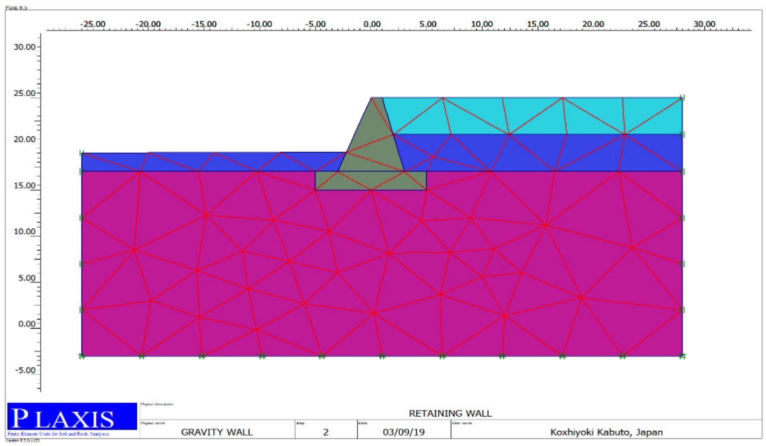

Figure 2: The finite element model used for numerical modelling

Table 1: Properties of the layered soil and retaining wall concrete

Parameter	Unit	Soil			Concrete
		Layer I	Layer II	Layer III	
Depth	m	2.0	2.0	>2.5	-
Bulk unit weight	kN/m^3	17	18	20	24
Friction angle	Degree	30	35	28	-
Young's modulus	kN/m^2	2.54 x 10^4	3.26 x 10^4	2.38 x 10^4	2.74 x 10^7
Poisson's ratio	-	0.321	0.398	0.296	0.2
Soil model	-	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	-
Soil behaviour	-	Drained	Drained	Drained	-
Material behaviour	-	Elasto-plastic	Elasto-plastic	Elasto-plastic	Linear (Isotropic)

2.2. Finite Element Modelling

The finite element method (FEM) provides an alternative way to design retaining structures. The method is generally used with the aid of computer programs as the calculations usually involve large matrix operations. The program used in this study is Plaxis 2D, a software that has in the last decades gained high acclaim in finite element analysis of geotechnical problems (Valsson, 2011). The limit designs calculated with

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conventional Coulomb and Rankine methods were evaluated with Plaxis and the results studied. Plaxis 2D is an axisymmetric finite element package used for two-dimensional analysis of deformation and stability in geotechnical engineering. It uses advanced soil constitutive models for the simulation of the non-linear, time dependent and anisotropic behaviour of soils and rocks. Plaxis 2D portfolio models the structure, the soil and the interaction between the structure and the soil (Salahudeen et al., 2019). Soil layers and structure parameters are inputted into Plaxis and the construction stages, loads and boundary conditions are defined in an already defined geometry cross-section containing the soil model then Plaxis automatically generates the unstructured 2D finite element meshes with options of global and local mesh refinements. Using its calculation facilities, Plaxis 2D undergoes a calculation process and presents the calculation and model outputs which can be accessed in animation and/or numerical forms. The input data in numerical modelling are index, elastic and strength parameters obtained from the processed laboratory results. In modelling stability of retaining wall numerically, the actual soil properties and conditions are considered together with the actual wall position, dimensions and material properties, process and stages. The applied boundary conditions used in numerical analysis are conditions in which the soil model bottom is restricted from movement in all directions (fixed in all of x, y and z-axes), the two sides are horizontally fixed and restrained from movement but vertically freed to move (fixed in x and z axes but free in y-axis) while the soil surface is totally unrestrained.

3. RESULTS AND DISCUSSIONS

3.1. Evaluation of Active and Passive Earth Pressures

The empirical methods proposed by Coulomb (1776) and Rankine (1857) were evaluated and results compared with those of infinite element method (FEM) using Plaxis. As presented in Figures 3 and 4, three important observations are obvious. Firstly, the method proposed by Coulomb yielded higher active and passive lateral earth pressures than that of Rankine. This may not be unconnected with the anisotropic nature of the soil. Coulomb approximated the curve failure plane to a straight line for mathematical simplicity and considered soil to be homogeneous, isotropic and ideally plastic material. These approximations and considerations provide computational comfort but consequently results in higher values of pressured above the values provided by FEM which was used as yardstick in this study. Secondly, the values provided by Rankine method are constantly below those of FEM. Rankine's method is based on the general state of elastic equilibrium. That is, the lateral earth pressure of a body of soil in a state of plastic equilibrium will be constant if every part of it is in incipient failure condition. However, failure may be imminent only in a small portion of the soil mass and not necessarily in the whole component. Thirdly, In the active zone, the FEM yielded the highest pressure values for wall friction angle factor up to 1.0 but constantly fall between the two empirical methods at the passive zone.

A peak active earth pressure value of 92.3 kN/m^2 was observed at wall friction angle factor of 0.6 and decreased with increase in wall friction angle factor to a minimum value of 19.0 wall friction angle factor of 1.6. A minimum passive earth pressure value of 235.7 kN/m^2 was observed at wall friction angle factor of 0.6 and increased with increase in wall friction angle factor to a peak value of 3254.95 kN/m^2 at wall friction angle factor of 1.6. This observation is similar to that of Sarath et al. (2011) who evaluated the role of shear keys in cantilever retaining wall with detailed analyses of the active and passive pressures acting on the wall considering Coulomb (1776) and Rankine (1857) methods. Comparing the values provided by Eurocode 7 (2004) to Coulomb's method, Valsson (2011) observed that the active pressure values superimposed, meaning they overlapped and exactly the same. The reason provided to explain the situation was that the wall roughness is defined in the same way in both of them. However, the Coulomb method yielded very high values for passive pressures which was blamed on the unrealistic assumptions of the method.

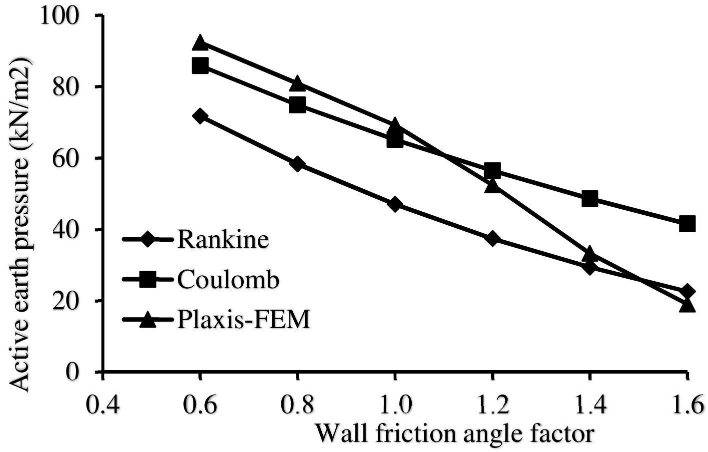

Figure 3: Variation of active earth pressure with wall friction angle factor

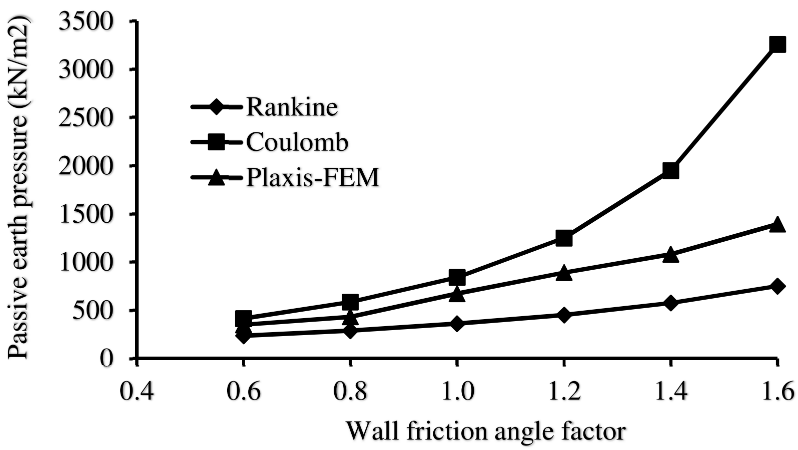

Figure 4: Variation of passive earth pressure with wall friction angle factor

3.2. Effect of Shear Key Position and Depth on Safety Factor Against Sliding Failure

The effect of shear key position on the safety factor against sliding failure of a gravity retaining wall was evaluated and results presented in Figure 5. Situations of no shear key and shear key placed at the toe, centre of stem and at the heel were considered. It was observed that for safety against sliding, the effect is insignificant for the wall system analysed. Studies have revealed that shear key position is most significant for safety against sliding in cantilever retaining walls whose elements are far slender than those of gravity retaining walls. That is, shear key position is not an issue in the design of gravity retaining walls as far as safety against sliding is concerned due to the self-weight of the walls (Green and Olgun, 2008). Installing the shear key at the at the toe, centre of stem and at the heel yielded the same results of safety factor against sliding for any particular depth of the key. However, it was observed that the depth of the shear key is significant for safety factor against sliding even for gravity retaining walls for different depth of the key. At wall friction angle factor of 0.6, only the situations with shear key depths of 1.6 and 2.0 m met the minimum safety factor against sliding of 1.5. It was observed that safety factor against sliding increases with increase in was friction angle factor and the longer the key the higher will be the safety factor against sliding. That is, using a longer shear key in the footing of retaining wall, reduces wall's sliding possibility considerably. The reason is that in wall with flat bottom, only the friction between the footing and the foundation soil resists wall's sliding. But in wall with shear key, passive pressure acting on the key reduces sliding too (Sichani and Bargi, 2012). The passive pressure acting on the key is greater than the friction under the wall's footing, so walls with shear key experience less sliding and the longer the shear key the higher the mobilized resistance against sliding.

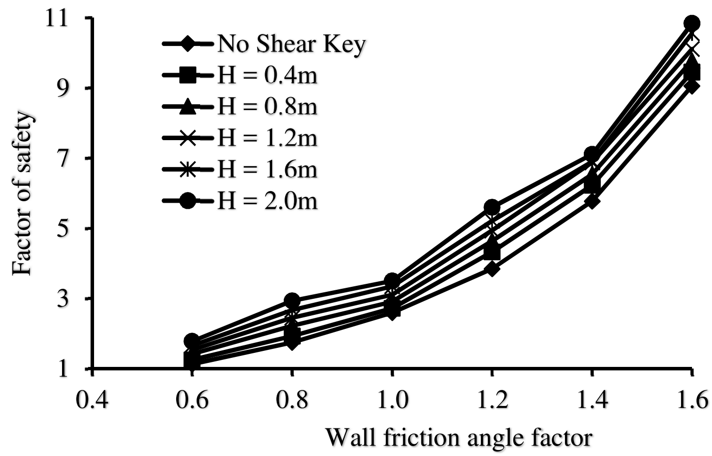

Figure 5: Variation of safety factor against sliding with wall friction angle factor

3.3. Effect of Shear Key Position and Depth on Safety Factor Against Overturning Failure

In the literatures, there has not been much study on the effect of shear key on safety of retaining walls against overturning failure. That is because, the safety against sliding is more difficult to attain compared with safety against overturning (Sichani and Bargi, 2012). Situations of no shear key and shear key placed at the toe, centre of stem and at the heel were considered as shown in Figures 6-10. It was observed that for safety against overturning, the effect is significant for the wall system analysed. That is, shear key position is a considerable issue in the design of gravity retaining walls as far as safety against overturning is concerned. It was observed that installing the shear key at the toe yielded the same results with situation of no shear key at all. Positioning the shear key at the heel or at any position between the stem and the heel is the best practice to enhance higher safety against overturning failure of the wall. However, there was no significant increase in safety factor against overturning with increase in shear key depth. It is noteworthy that in designing retaining walls, this observed effect of shear key position on the safety factor against overturning should only be considered in situations where the slip surface passes through any part of the wall. If the slip surface passes below the base of the shear key then this observation is irrelevant. That is because, in this case only the soil's shear strength properties come to play. All the situations considered met the conventionally recommended minimum value of 3.0 safety factor against overturning failure. This could be as a result of the anisotropic advantages associated with layered soils. Fabric anisotropy has long been recognized to significantly affect the peak strength of both sands and clays. A prominent example is the bearing capacity test of a strip footing on sand examined by Azami et al. (2010), wherein the bearing capacity measured with the loading direction perpendicular to the deposition plane was around 25 - 34% higher than that with a loading case parallel to the plane. The more rigorous Finite Element Method (FEM) has allowed the relevance of anisotropy to be further confirmed for numerous types of geotechnical applications (Salahudeen and Aghayan, 2018).

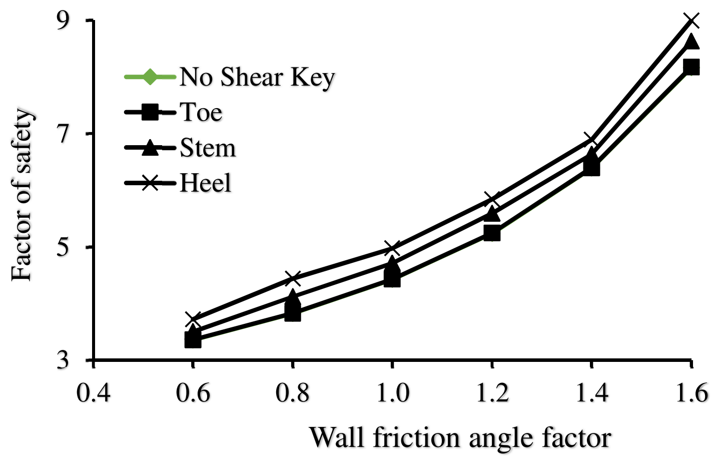

Figure 6: Variation of safety factor against overturning with wall friction angle factor for H=0.4 m

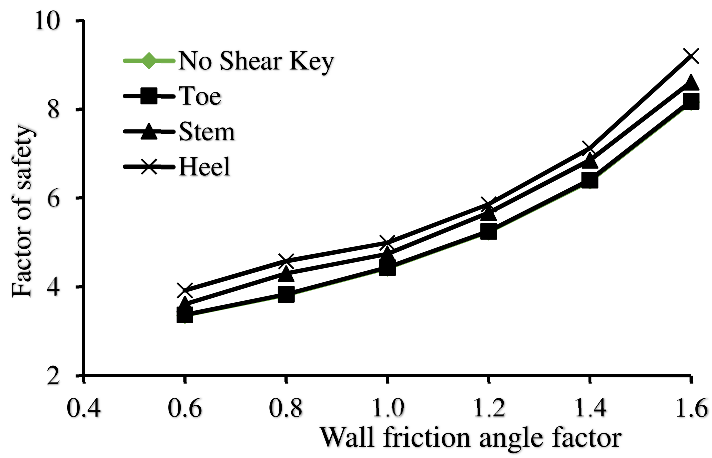

Figure 7: Variation of safety factor against overturning with wall friction angle factor for H=0.8 m

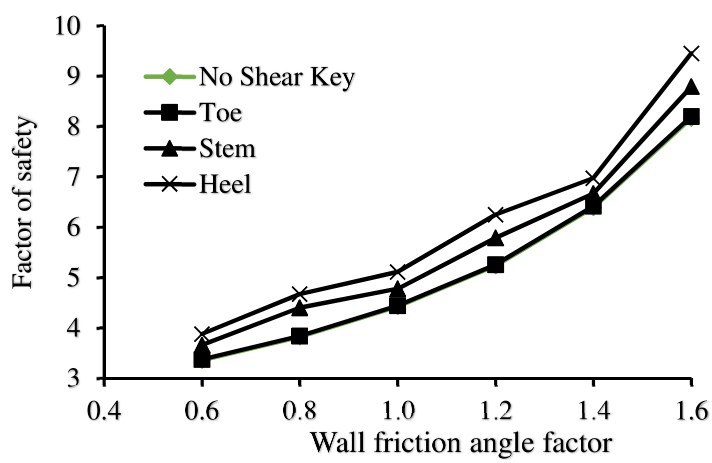

Figure 8: Variation of safety factor against overturning with wall friction angle factor for H=1.2 m

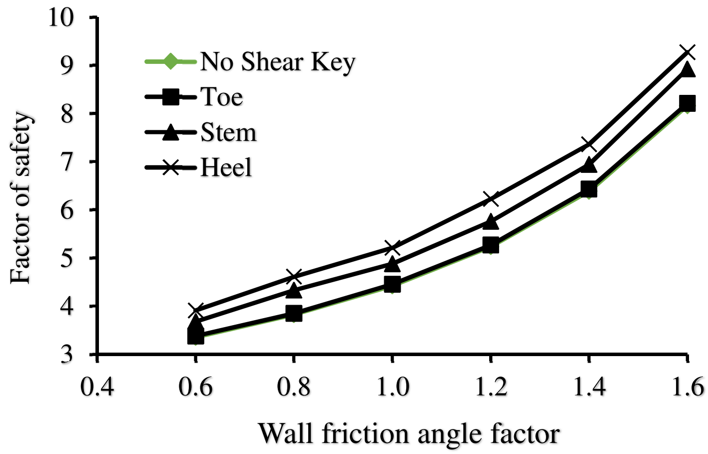

Figure 9: Variation of safety factor against overturning with wall friction angle factor for H=1.6 m

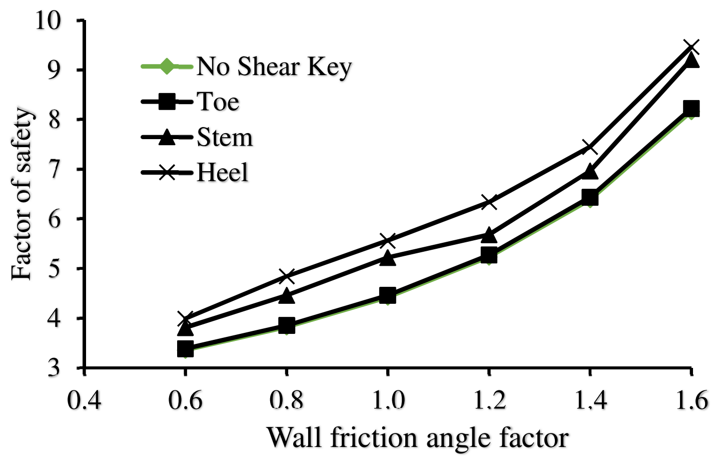

Figure 10: Variation of safety factor against overturning with wall friction angle factor for H=2.0 m

4. CONCLUSION

In this study, the earth pressure distribution generated behind the gravity retaining wall supporting an anisotropic cohesionless soil was estimated by the two classical lateral earth pressure theories proposed by Rankine (1857) and Coulomb (1776) and compared with that obtained by finite element method using Plaxis 2D. The effects of shear key against sliding and overturning failures were examined. Shear key was considered in three locations: below the heel, stem and toe. A situation of no shear key was also sufficiently examined. The following conclusions were drawn:

- The method proposed by Coulomb yielded higher active and passive lateral earth pressures than that of Rankine. The values provided by Rankine method are constantly below those of FEM.
- In the active zone, the FEM yielded the highest pressure values for wall friction angle factor up to 1.0 but constantly fall between the two empirical methods at the passive zone. A peak active earth pressure value of 92.3 kN/m^2 was observed at wall friction angle factor of 0.6. and a minimum value of 19.0 at wall friction angle factor of 1.6. A minimum passive earth pressure value of 235.7 kN/m^2 was observed at wall friction angle factor of 0.6 a peak value of 3254.95 kN/m^2 at wall friction angle factor of 1.6.

- For safety factor against sliding, the effect of shear key position is insignificant for the gravity wall system analysed. Installing the shear key at the at the toe, centre of stem and at the heel yielded the same results of safety factor against sliding for any particular depth of the key. However, it was observed that the depth of the shear key is significant for safety factor against sliding even for gravity retaining walls. It was observed that the longer the shear key the higher will be the safety factor against sliding.
- For safety factor against overturning, shear key position is a considerable issue in the design of gravity retaining walls. It was observed that installing the shear key at the toe yielded the same results with situation of no shear key at all. Positioning the shear key at the heel or at any position between the stem and the heel was observed to be the best practice to enhance higher safety against overturning failure of the wall.

5. CONFLICT OF INTEREST

There is no conflict of interest associated with this work.

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