Passive Earth Pressure on Walls Retaining Over-Consolidated Collapsible Soils Subjected to Partial Inundation

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ABSTRACT
This paper presents a numerical investigation on the passive earth pressure acting on walls retaining collapsible soils at various degrees of saturation and over-consolidation ratios. A 2D finite element model was developed using the software ABAQUS. The model employed the theory of unsaturated soil mechanics, including the soil water characteristic curve (SWCC) and the variation in the soil's parameters with respect to matric suction. It was concluded that the coefficient of passive earth pressure significantly decreases as the degree of saturation increases, and this amount of reduction decreases as the over-consolidation ratio of the collapsible soil increases. Practical charts were produced to estimate the coefficient of passive earth pressure for a given collapse potential, over-consolidation ratio, and degree of saturation.

RÉSUMÉ
Cet article décrit l'investigation numérique de pression passive de terres agissant sur des murs qui retiennent des sols effondrables à différents degrés de saturation et de rapports de surconsolidation. Un modèle d'éléments finis 2D a été développé à l'aide du logiciel ABAQUS. Ce modèle utilise la théorie de la mécanique des sols non saturés incluant la courbe caractéristique de l'eau dans le sol (SWCC) ainsi que la variation des paramètres de résistance du sol en fonction à la succion matricielle. Il a été conclu que le coefficient de pression passive des terres diminue significativement lorsque le degré de saturation augmente. Néanmoins, cet effet de diminution est moins important lorsque le rapport de surconsolidation du sol effondrable augmente. Des tableaux pratiques pouvant être utilisés pour estimer le coefficient de pression passive pour un certain potentiel d'effondrement, un rapport de surconsolidation et un degré de saturation donnés ont été établis

1 INTRODUCTION

Collapsible soils, as defined by Dudley (1970) and Bara (1976), are unsaturated soils that exhibit high sustainability to loads when dry but experience a sudden volume reduction when subjected to inundation. Collapsible soils have been known to have partially unstable, partially saturated structure, where the coarse particles are bonded by weak cementing agents of fine particles resulting in a high void ratio and low density. Many of the soils that have been classified as collapsible soils were aeolian, alluvial, colluvial, or residual soils.

Due to today's constant growth in construction and urban development, the exposure to different sites having collapsible soils has become more common, causing significant challenges to geotechnical engineers, especially that the collapse could be caused by multiple factors like a rise in the groundwater table, irrigation, or heavy rainfalls. Many structures found on collapsible soils experience severe damages such as differential settlements, earth cracks, landslides, or slope stability problems. One of these common structures is retaining walls.

Retaining walls are commonly used in deep excavations, dams, and tunnels. They are mainly used to hold the soil in place, eliminating horizontal displacements by counteracting any present lateral pressure. There are three types of lateral pressure that soils exert on retaining walls: at-rest, active and passive. This article focuses on the passive lateral pressure which represents the lateral pressure that soils exert on walls displaced horizontally towards the soil.

Thus, this article analyzes the change in the coefficient of passive earth pressure for walls retaining collapsible soils subjected to partial inundation. A 2D finite element model was developed, that incorporated the gradual collapse that occurs in the soil when subjected to inundation, in which the change in the coefficient of passive earth pressure was analyzed at variable values of degree of saturation, collapse potential, and over-consolidation ratio.

1.1 Overview of previous studies

1.1.1 Collapsible Soils Identification

Jennings and Knights (1975) introduced a relation, presented in Equation 1, to estimate the soil's collapse potential as a function of the change in the void ratio before and after inundation
This relation could be used to evaluate the severity of the soil's collapse potential based on the values presented in Table 1.

$$C_p = \frac{\Delta e_o}{1 + e_o} * 100\% \quad [1]$$

Where C_p is the collapse potential, Δe_o is the difference between the initial void ratio and the void ratio after inundation, and e_o is the initial void ratio.

Table 1 Collapse potential and severity of foundation problem (Jennings and Knight, 1975*)*

Collapse Potential C_p (%)	Severity of problem
0-1	No problem
1-5	Moderate trouble
5-10	Trouble
10-20	Severe trouble
>20	Very severe trouble

Basma and Tuncer (1992) suggested an empirical formula to predict the soil's collapse potential based on the difference in the percentage of clay content to sand content, the initial water content, the dry unit weight, and the pressure applied on the soil during inundation, as given in Equation 2.

$$C_p = 47.506 - 0.072(S - C) - 0.439w_{ci} - 3.123\gamma_d + 2.851\, ln(p_w) \quad [2]$$

Where $(S - C)$ is the difference between the sand and clay percentage, w_{ci} is the initial water content (%), γ_d is the dry unit weight (kN/m^3), and p_w is the pressure at wetting (kPa).

Ayadat and Hanna (2012) presented an extensive evaluation of existing collapse prediction criteria. They modified the limitations associated with some of them and changed the format of others. They also introduced a new method to identify the collapse potential based on the soil's bulk unit weight and the individual unit weight of each of its constituents, as presented in Figure 1.

This method was derived from existing prediction methods introduced by *Feda (1966), Markin (1969), Denisov (1969), and Minkov (1977).*

1.1.2 Characteristics of the Collapse

In the literature, there has been an increasing effort to identify the behavior of unsaturated or partially saturated soils, such like collapsible soils, using two important independent stress state variables, the net normal stress $(\sigma_f - u_a)$ and the matric suction $(u_a - u_w)$, where σ_f is the total normal stress, u_a is the pore air pressure and u_w is the pore water pressure (*Bishop et al.,1960; Fredlund and Rahardjo, 1993; Fredlund and Morgenstern, 1977, Fredlund, Morgenstern, and Widger, 1978; Jotisankasa, 2005).*

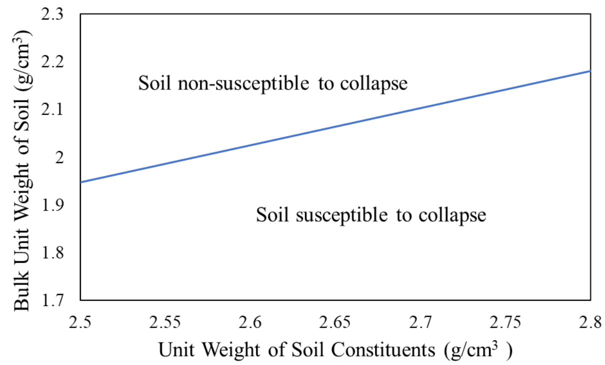

Figure 1 Identification of Collapsible Soils (Ayadat and Hanna, 2012)

Matric suction, as defined by *Jotisankasa (2005),* is the tensile stress measured through a porous tip making intimate contact with the soil's water content and can be calculated as the difference between the pore air pressure and the pore water pressure as given in Equation 3.

$$S = u_a - u_w \quad [3]$$

The impact of matric suction has recently been given a lot of importance when dealing with unsaturated or partially saturated soils, as it highly affects the soil's shear strength and physical properties, such as the effective angle of friction φ' and effective cohesion c'. Thus, Futai *et al.* (2006) reported that there is a variation in the angle of friction and cohesion with matric suction, that can be determined using Equations 4 and 5.

$$\varphi(s) = \varphi' + \left(\varphi_{(u_a-u_w=\infty)} - \varphi'\right)\left(1 - 10^{b(u_a-u_w)}\right) \quad [4]$$

$$c(s) = c' + (c_{(u_a-u_w=\infty)} - c')(1 - 10^{a(u_a-u_w)}) \quad [5]$$

Where φ' and c' are respectively the effective angle of friction, and effective cohesion for soils in saturation conditions, $\varphi_{(u_a-u_w=\infty)}$ and $c_{(u_a-u_w=\infty)}$ are respectively the maximum value of the soil's angle of friction and cohesion, and a and b are adjustment factors. This variation was explained by the fact that the soil's angle of friction and cohesion depend on the type of minerals that make up the soil, the inter-particle sliding resistance, geometric interlocking, particle interpenetration, the grain break-up and dilation, and the water content. As the matric suction increases, it induces an increase in the interlacing bonds between the soil particles because of retraction caused by the process of drying, which will consequently increase the friction between the particles resulting in an increase in the soil's effective angle of friction and a decrease in the soil's cohesion. Measuring soil parameters, like matric suction, in unsaturated soils can be challenging due to the difficulty in separating the various soil-water interactions found in the soil's micro-pores and macro-pores. It is also costly and time-consuming to identify the hydro-mechanical behavior of collapsible soils during inundation.

Thus, some estimation techniques were introduced in the literature that facilitate the identification of unsaturated soil parameters. One of these estimation techniques is the Soil Water Characteristic Curve (SWCC), that was used by many researchers in different studies, such as *Vanpalli, Fredlund and Pufahl (1999); Lins and Shanz (2005); Jotisankasaa (2005); Elsharief and Abdulaziz (2015),* which is defined as a graph that determines the relationship between the amount of water in the soil, whether gravimetric, volumetric, normalized, or dimensionless water content, or degree of saturation, and the matric suction.

Poterasu (2013) concluded that the collapse potential C_p of the soil decreases as the soil's initial water content increases, and as the soil's clay content increases.

Nguyen (2018) concluded that the collapse mechanism is governed by the collapse potential C_p of the soil, which depends on the clay content. Based on the experimental investigation that was performed in this study, it was observed that the collapse settlement of the soil increases with an increase in the collapse potential C_p, but decreases with an increase in the over-consolidation ratio OCR. The collapse settlement was found to increase rapidly up to 80% of the degree of saturation, after which it continued to increase at a lower rate, up to 20-40% of the collapse settlement that was achieved at 100% saturation. It was also concluded that the collapse settlement is relatively faster at higher values of collapse potential C_p.

1.1.3 Coefficient of Passive earth pressure

Rankine's theory introduced in 1875 is one of the most used theories to evaluate the passive earth pressure, from which the coefficient of passive earth pressure can be calculated using Equation 6. Rankin's theory was used to validate the results obtained from this analysis.

$$K_p = \frac{1 + sin\varphi'}{1 - sin\varphi'} = \tan^2\left(45° + \frac{\varphi'}{2}\right) \quad [6]$$

Poterasu (2013) performed experimental and numerical investigations on passive earth pressure acting on walls retaining collapsible soils. The influence of the soils collapse potential C_p on the passive earth pressure was studied at both dry and fully saturated conditions. It was reported that, when the soil is dry, the passive earth pressure increases with an increase in the soil's collapse potential C_p, but it decreases significantly when the soil is fully saturated.

Hanna and Nguyen (2018) performed an experimental investigation on the passive earth pressure acting on walls retaining over-consolidated collapsible soils subjected to inundation. They analyzed the influence of the soil's collapse potential C_p, and over-consolidation ratio OCR on the passive earth pressure. The analysis was performed on collapsible soils of variable magnitudes of cohesion c' and angle of friction φ'. They reported that, in dry conditions, the coefficient of passive earth pressure K_p increases with the increase in collapse potential C_p, and over-consolidation ratio OCR. However, when the soil is fully inundated ($S_r = 100\%$), they reported that the coefficient of passive earth pressure significantly decreases with the increase in collapse potential C_p, but still increases with the increase in over-consolidation ratio OCR. Design charts were also developed to estimate the coefficient of passive earth pressure K_p for dry, and fully saturated collapsible soils at various over-consolidation ratios OCR and collapse potentials C_p, assuming a linear behavior between dry and fully saturated conditions.

Thus, the following sections will present more details about the change in the coefficient of passive earth pressure during the inundation process.

2 MODEL DEVELOPMENT

A 2D numerical model was developed, using the finite element software ABAQUS, to perform this investigation in which the coefficient of passive earth pressure was evaluated for unsaturated soils of variable collapse potentials C_p, at different degrees of saturation S_r, and over-consolidation ratios OCR.

The numerical model was validated by comparing the results obtained from the numerical analysis with the results obtained from the laboratory experiment performed by *Hanna and Nguyen (2018).*

2.1 Model geometry

The model consisted of two geometric parts. These two parts were chosen to be 2-D planar, deformable shell elements. The width of the wall was neglected and only displacements and rotations about the horizontal and vertical axes, represented by the x and y-axis respectively, were taken into consideration. The model represented a deep excavation, supported by 6 m walls retaining a collapsible soil backfill. Figure 2 presents the dimension used in the model.

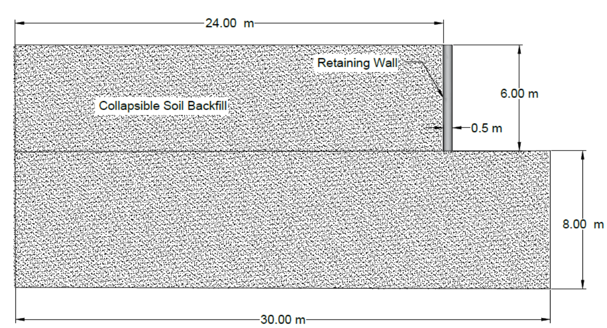

Figure 2 Numerical Model Geometry

2.2 Material properties

2.2.1 Retaining wall

The retaining wall was assumed to be made of concrete and it was chosen to behave elastically since its behavior was not in the scope of the study. Table 2 presents a summary of the input parameters obtained from the experimental investigation of *Hanna and Nguyen (2018).*

Table 2 Retaining wall elastic material properties

Geometric Part	Unit Weight γ (kN/m^3)	Modulus of Elasticity E (kPa)	Poisson's Ratio υ
Retaining Wall	27	6.89E07	0.3

2.2.2 Properties of the backfill at initial conditions ($S_r = 20\%$)

Four variations of soil parameters were used in this model. The parameters were obtained from the experimental investigation done by *Hanna and Nguyen (2018),* and they are summarized in Table 3. All four soils were classified as clayey sands (SC), or poorly graded sands (SP) according to the Unified Soil Classification System (USCS). According to *Jennings and Knights (1975),* the soils with collapse potentials C_p of 4.2%, 9%, 12.5%, and 18% were classified as moderate trouble, trouble, severe trouble, and severe trouble respectively, summarized in Table 3.
All four soils were assigned an initial degree of saturation of 20% according to their initial moisture content of 5%.

Mohr-Coulomb constitutive law was chosen to represent the behavior of the soil backfill in ABAQUS. The plasticity parameters include the soil's angle of friction φ' and dilation angle Ψ, which is equal to $(\varphi' - 30)$ (*Bolton, 1986; Kakoli $et\,al.$, 2009*). The cohesion parameters include the soil's cohesion c, and the absolute plastic strain. The elastic parameters include the soil's modulus of Elasticity E, and Poisson's ratio ν.

The initial modulus of elasticity used for each soil was calculated using Equation 7 introduced by *Reznik (2007),* which is a function of the soil's degree of saturation S_r, and void ratio e.

$$E(e, S_r) = 10^{-rS+q}\ (MPa) \qquad [7]$$

Where, E is the modulus of Elasticity, e is the void ratio, S_r is the degree of saturation, and r & q are coefficients that can be obtained from Table 4 reproduced from *Reznik, (2007).*

The obtained values for the modulus of elasticity E are shown in Table 5. Poisson's ratio was chosen to be equal to 0.4, which is a common value used for clayey sands (*Bowles, 1996; Kulhawy et al., 1990; Hanna et al. 2013).*

Table 3 Soil properties

Soil Properties	Soil A	Soil B	Soil C	Soil D
Clay Content (%)	6.00	8.00	10.00	14.00
Unit Weight γ (kN/m^3)	16.28	16.25	16.20	15.30
Angle of Friction (degree)	40.00	38.50	35.00	31.00
Cohesion c (kPa)	9.00	12.50	15.50	18.00
Void Ratio e_o	0.67	0.69	0.70	0.80
Water Content w_c (%)	5.00	5.00	5.00	5.00
Liquid Limit LL	-	-	15.90	24.70
Plastic Limit PL	-	-	13.35	17.30
Plasticity Index PI	-	-	2.55	7.40
Coefficient of Uniformity C_u	4.00	5.40	21.90	30.00
Coefficient of Curvature C_c	1.27	1.65	6.47	8.53
Collapse Severity (Jennings and Knight 1975)	Moderate trouble	Trouble	Severe trouble	Severe trouble

Table 4 Values for (-rS + q) at different void ratios (Reznik, 2007)

Soil	Void Ratio, e	$-rS + q$
Loess	>1.0	$-0.910S + 0.995$
	0.9 – 1.0	$-0.860S + 1.240$
	0.8 – 0.9	$-0.830S + 1.425$
Loessial Loam	0.9 – 1.0	$-1.040S + 1.402$
	0.8 – 0.9	$-0.920S + 1.477$
	0.7 – 0.8	$-0.833S + 1.558$
	0.6 – 0.7	$-0.785S + 1.627$
	< 0.6	$-0.757S + 1.710$

Table 5 Calculated moduli of elasticity using Equation 2.0

Soil Properties	Soil A	Soil B	Soil C	Soil D
Modulus of Elasticity E (MPa)	29592	29472	24731	20794

2.2.3 Properties of the backfill during inundation (20%< S_r < 100%) -Soil Collapse

Modeling the soil collapse in ABAQUS was mainly dependent on the change in the soil's strength and stiffness parameters during inundation, in which the variation in four parameters were considered: the soil's modulus of elasticity E, angle of friction φ', cohesion c, and dilation angle Ψ.

Changing the soil parameters was done by calculating new values for each parameter at each degree of saturation S_r using **Equations 4, 5 and 7**. An increment of 8% was used for the degree of saturation S_r. Values of matric suction $(u_a - u_w)$ were obtained from the Soil Water Characteristic Cure (SWCC) for collapsible soils provided by *Lins and Shanz (2005).* Each variation of parameters calculated at each increment of degree of saturation S_r was then input in ABAQUS as a separate set of materials. The collapse was then introduced in multiple steps that incorporated the incremental increase in degree of saturation S_r along with the variation in the material properties.

It should be noted that the decrease in both the soil's cohesion c and angle of friction φ' was relatively insignificant, but it was still considered in this study to ensure accuracy in the numerical representation of the soil's behavior.

2.2.4 Boundary Conditions and Interface Interaction

To best represent the field conditions, the soil backfill was only allowed to move vertically to account for the collapse settlement, in which it was completely fixed from the bottom, and horizontally fixed from both the right and left side.

The retaining wall was modelled as a rigid body, eliminating any deformations resulting from horizontal displacement.

The vertical interaction between the retaining wall and the soil was assumed to be frictionless, and the horizontal interaction was assumed to be hard, restricting any vertical penetration of the wall into the soil.

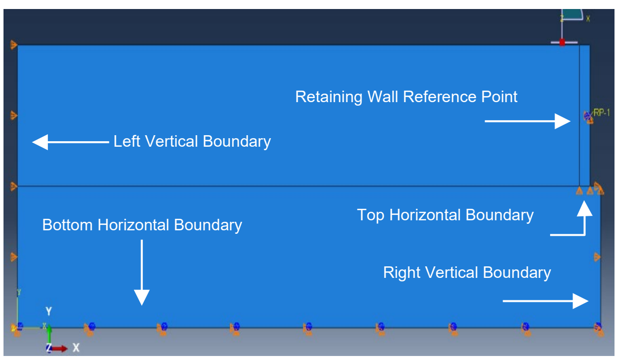

Figure 3 Boundary conditions of the numerical model

2.2.5 Mesh Analysis

According to the uniform geometry of the model, a structured-type mesh was used, since it offers a better mesh control and provides more accurate results, especially for shell elements. *Kumar and Rao (2018)* reported that "*this technique offers a better mesh control to the user compared to sweep meshing technique*". Both the soil backfill and the retaining wall were assigned as structured quadrilateral elements. The type of mesh assigned to the wall is insignificant because the behavior of the wall is not in the scope of this study. However, it was assigned the same mesh type of the soil backfill, in which any possible calculation divergence was eliminated. The soil backfill was assigned an 8-node biquadratic plane strain quadrilateral mesh.

To confirm the accuracy of the mesh size, a mesh analysis was performed by evaluating K_p at dry and fully saturated conditions for $C_p = 4.2\%$ at OCR= 1. using three different mesh sizes: 0.25x0.25, 0.5x0.5, and 0.7x0.75 m. The results are summarized in Table 6.

Table 6 Results obtained from the mesh size analysis for K_p

Mesh Size (m)	K_p (kPa)	
	$S_r = 20\%$	$S_r = 100\%$
0.25x0.25	5.181	0.673
0.50x0.50	5.165	0.662
0.75x0.75	5.100	0.575

It can be seen from Table 6 that there is a slight difference between the results obtained from each mesh size. Accordingly, mesh size 0.5x0.5 m was used in this analysis, as it provided both accuracy in results and efficiency in time.

3 MODEL VALIDATION

The modelling technique provided in previous sections was first implemented in a smaller scale model that represented the same model geometry and conditions used in the experimental investigation of *Hanna and Nguyen (2018).* The results obtained from this preliminary model was compared and validated with the results obtained by *Hanna and Nguyen (2018)*. The values of the coefficient of passive earth pressure K_p were validated for the multiple soil types presented in Table 3 at three different values of OCR, and at initial $(S_r = 20\%)$ and full saturation $(S_r = 100\%)$ conditions. Figures 4 and 5 present a comparison between the results obtained from this study and the results presented in the experimental investigation of Hanna and Nguyen (2018) at initial $(S_r = 20\%)$ and full saturation conditions $(S_r = 100\%)$ respectively, from which the same trend can be noted with a relatively good agreement in the results.

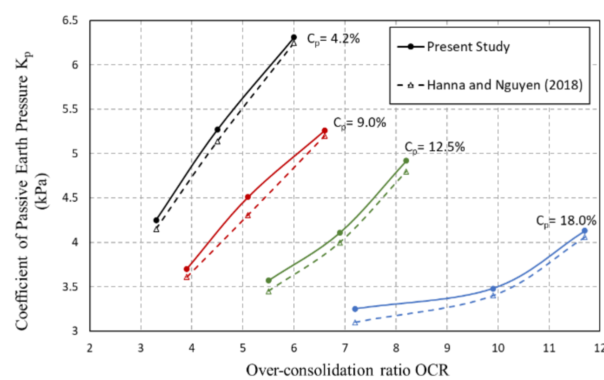

Figure 4 Comparison between the test results of K_p from the present study and the experimental investigation of *Hanna and Nguyen (2018)* at initial conditions $(S_r = 20\%)$

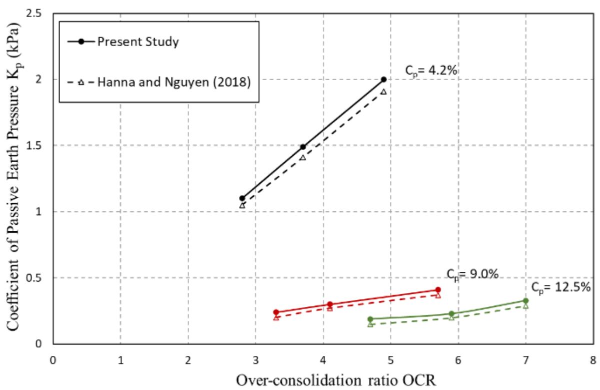

Figure 5 Comparison between the test results of K_p from the present study and the experimental investigation of *Hanna and Nguyen (2018)* at full saturation ($S_r = 100\%$)

4 TEST RESULTS AND ANALYSIS OF PASSIVE EARHT PRESSURE K_p

Figure 6 presents an overview of the plastic shear strain failure plane obtained at failure full saturation (S_r= 100%) for four soils. The following sections present an analysis of all the results attained in this study.

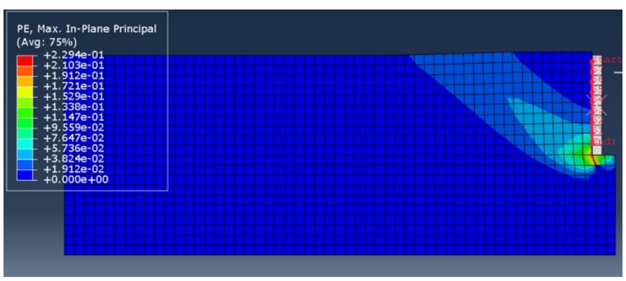

Figure 6 Plastic strain failure plane at full saturation (S_r= 100%)

4.1 Initial Conditions ($S_r = 20\%$)

Table 7 presents the test results for the coefficient of passive earth pressure K_p at initial conditions ($S_r = 20\%$) along with the theoretical values of *Rankine (Das and Sobhan, 2010)*, for normally consolidated soils (OCR= 1), in which a good agreement can be noted.

Figure 7 presents the results for K_p at initial conditions ($S_r = 20\%$), for different values of collapse potential C_p at various over-consolidation ratios OCR It can be noted that K_p decreases with the increase in collapse potential C_p, which can be explained by the fact that increasing the soil's clay content (higher C_p) decreases the soil's effective angle of friction φ', resulting in a decrease in K_p. It can also be noted that K_p increases as the over-consolidation ratio OCR increases, since at higher values of over-consolidation ratio, the soil becomes denser increasing the angle of friction φ', between the soil particles, which results in an increase in K_p.

Table 7 Comparison between values of K_p obtained from this study and from Rankine's theory

Model Number	C_p(%)	OCR	K_p (kPa) Present Study	K_p (kPa) Rankine (1857)
1	4.2	1	5.14	4.60
5	9	1	4.70	4.30
9	12.5	1	4.35	3.69
13	18	1	3.86	3.12

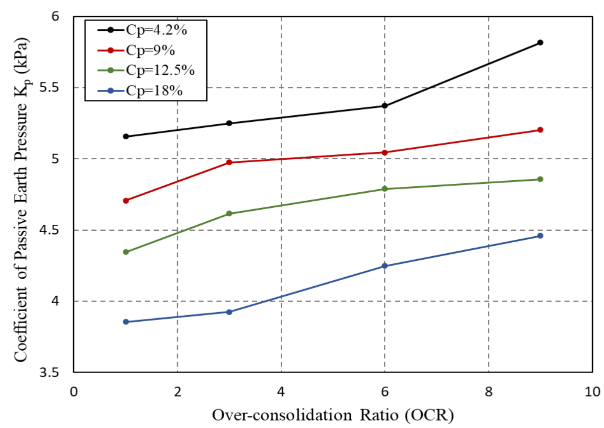

Figure 7 Coefficient of passive earth pressure K_p versus over-consolidation ratio OCR at initial conditions ($S_r = 20\%$)

4.2 Partial Inundation (20% < S_r < 100%)

Figure 8 presents the test results for the coefficient of passive earth pressure K_p versus degree of saturation S_r, for normally consolidated collapsible soils (OCR= 1). It can be noted that K_p sharply decreases as the degree of saturation S_r increases, up to about S_r =70% at which the water dominates the overall behavior of the soil, and the reduction in K_p becomes relatively insignificant. It is also noted that the amount of reduction in K_p caused by the increase in the degree of saturation S_r is greater for higher values of collapse potential C_p.

Figures 9 to 11 present the test results for the coefficient of passive earth pressure K_p versus degree of saturation S_r for values of OCR= 3, 6, and 9 respectively.

The same trend can be observed for all values of OCR, in which K_p decreases non-linearly as the degree of saturation S_r increases. However, it can be noted that the amount of reduction in the coefficient of passive earth pressure K_p decreases with the increase in over-consolidation ratio OCR, since the void ratio is smaller for highly over-consolidated soils, resulting in smaller collapse settlements. Thus, at high values of over-consolidation ratio OCR, the value of the soil's collapse potential C_p becomes insignificant to the amount of reduction in K_p.

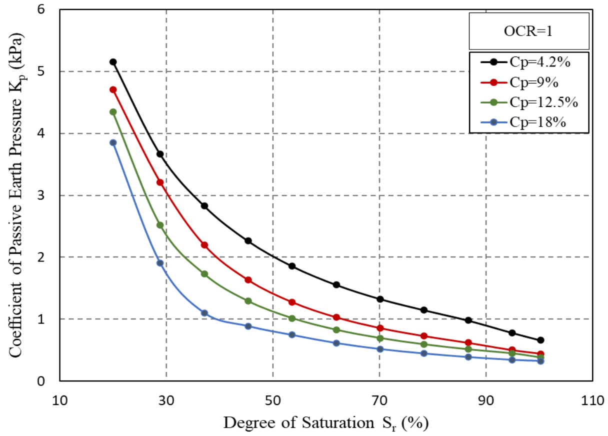

Figure 8 Coefficient of passive earth pressure K_p versus degree of saturation S_r at OCR=1

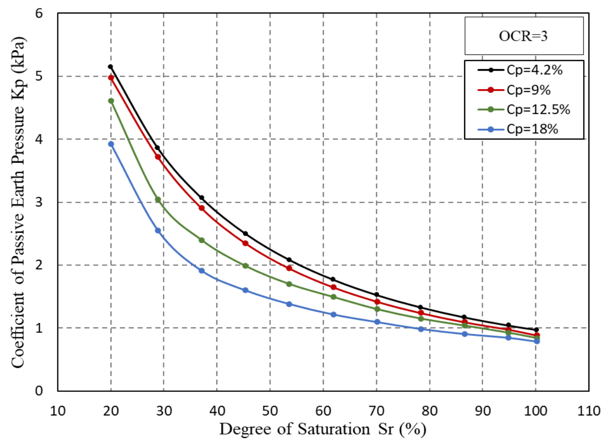

Figure 9 Coefficient of passive earth pressure K_p versus degree of saturation S_r at OCR=3

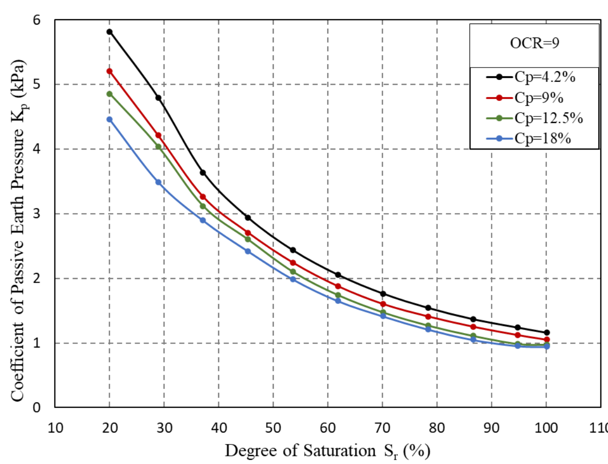

Figure 10 Coefficient of passive earth pressure K_p versus degree of saturation S_r at OCR=6

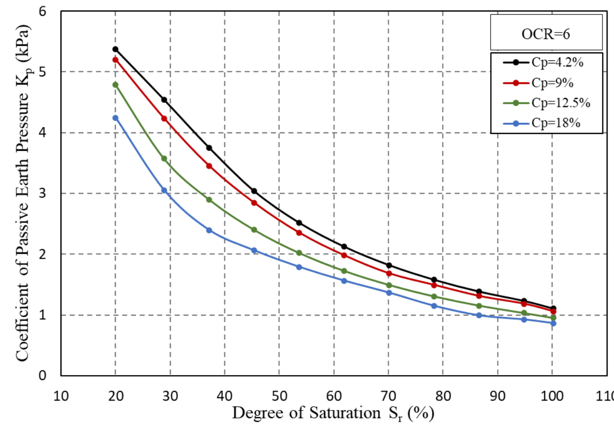

Figure 11 Coefficient of passive earth pressure K_p versus degree of saturation S_r at OCR=9

4.3 Full Saturation ($S_r = 100\%$)

Figure 12 presents the test results for the coefficient of passive earth pressure K_p at full saturation ($S_r = 100\%$) versus the over-consolidation ratio OCR for each of C_p = 4.2%, 9%, 12.5%, and 18%. It can be noted that the coefficient of passive earth pressure K_p decreases significantly as the collapse potential C_p increases.
This is because soils of higher collapse potential have a higher percentage of clay content (cementing agents between coarse particles) that get dissolved by water resulting in a reduction in the soil's strength and collapse settlement. It can also be noted in Figure 12 that K_p increases rapidly from OCR = 1 to 3 for all values of collapse potential, beyond which the increase continues at a relatively slower rate.

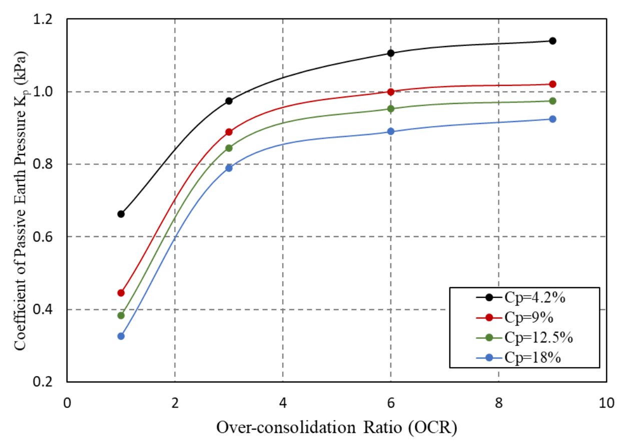

Figure 12 Coefficient of passive earth pressure K_p versus over-consolidation ratio OCR at S_r = 100%

Table 8 presents the test results of the coefficient of passive earth pressure K_p at $S_r = 20\%$ and at full saturation ($S_r = 100\%$), along with the amount of reduction in K_p. It is noted that beyond OCR= 6 the values of K_p do not considerably change. It is of interest to report herein that increasing the soil's over-consolidation ratio OCR can relatively minimize the amount of reduction in K_p.

Table 8 Test results for the coefficient of passive earth pressure K_p at initial conditions ($S_r = 20\%$) and at full saturation ($S_r = 100\%$)

Model Number	C_p(%)	OCR	Present Study		
			$K_{p,dry}$ (kPa)	$K_{p,sat}$ (kPa)	Amount of reduction in K_p
1	4.2	1	5.16	0.66	87%
2	4.2	3	5.25	0.97	81%
3	4.2	6	5.37	1.11	79%
4	4.2	9	5.82	1.16	79%
5	9	1	4.71	0.45	91%
6	9	3	4.97	0.89	82%
7	9	6	5.04	1.00	80%
8	9	9	5.20	1.02	80%
9	12.5	1	4.35	0.38	91%
10	12.5	3	4.62	0.85	82%
11	12.5	6	4.79	0.95	80%
12	12.5	9	4.86	0.98	80%
13	18	1	3.86	0.33	92%
14	18	3	3.93	0.79	82%
15	18	6	4.25	0.89	80%
16	18	9	4.46	0.93	80%

5 CONCLUSIONS

A numerical model was developed using the finite element software ABAQUS to investigate the passive earth pressure K_p, on walls retaining collapsible soils subjected to partial and full inundation, at variable values of the over-consolidation ratio.

The model was validated by an experimental investigation provided in the literature. The main objective of this study was to determine the change in the values of K_p as the soil's degree of saturation increases at variable values of OCR. Also, to develop practical charts that can be used to estimate the passive earth pressure of over-consolidated collapsible soils at any degree of saturation. Based on the results obtained in this study, the following can be concluded:

1. The behavior of the coefficient of passive earth pressure K_p, at dry state and full saturation, agrees well with the findings of Hanna and Nguyen (2018), which state that K_p, increases as the soil's collapse potential increases, and the over-consolidation ratio increases.rel Whereas it decreases as the soil's degree of saturation increases,
2. The decrease in the values of the coefficient of passive earth pressure with respect to the increase in the degree of saturation is non-linear,
3. The coefficient of passive earth pressure K_p sharply decreases as the degree of saturation S_r increases. The amount of decrease is a function of the soil's collapse potential C_p and over-consolidation ratio OCR,
4. The amount of decrease in the value of K_p, caused by the increase in the degree of saturation, is highest when the soil is normally consolidated (OCR = 1), and has a high value of collapse potential C_p. However, it reduces as the over-consolidation ratio OCR increases, up to a certain over-consolidation ratio OCR, where it becomes almost the same for values of collapse potential in the range of 4.2%-18%,
5. The value of collapse potential C_p at high values of over-consolidation ratio OCR, is insignificant to the amount of reduction in K_p caused by the increase in the degree of saturation,
6. The amount of reduction in K_p, caused by the increase in the degree of saturation S_r, increases as the collapse potential C_p increases.
7. The rate of reduction in K_p, caused by the increase in the degree of saturation S_r, is higher at low values of over-consolidation ratio and high values of collapse potential,
8. Practical charts were introduced to predict K_p at any degree of saturation that falls within the range of 20% to 100% for a given over-consolidation ratio, for C_p= 4.2%, 9%, 12.5% and 18%.

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