

**Earth Pressures of Overconsolidated Collapsible Soil Subjected  
to Inundation**

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# ABSTRACT

## **Earth Pressures of Overconsolidated Collapsible Soil Subjected to Inundation**

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Collapsible soils are known as problematic soils, which can be found in many regions around the world. Collapsible soils possess considerable strength when they are dry; however, when they are inundated, they lose their strength and exhibit excessive settlement. The amount of soil collapse increases with the increase of the so-called collapse potential " $C_p$ ", the wetting zone, and the degree of saturation (S). Accordingly, maximum collapse will take place due to full saturation. Collapsible soils can be inundated by heavy and continuous rainfall, excessive irrigation, broken water/sewer lines, or by rising the ground water. Furthermore, collapsible soils can be also found in construction site, in compacted fine soils at low water content (less than the optimum water content). Consequently, it is impossible to avoid construction on collapsible soils, which are potential for excessive settlement, differential settlement, landslides and falls, earth cracks. This type of soil has been responsible for damaging variety of civil engineering structures, loss of lives. Construction on such kind of soil shows extraordinary geotechnical problems, retaining walls are not an exception. With civilization, backfills behind retaining walls made of collapsible soils are widely used in practice. The earth pressures acting on these walls experience radical changes when the backfills of collapsible soils are wetted.

In the literature, there are lack of theories/methods for estimating the earth pressures acting on walls retaining collapsible soils. In this study, experimental investigations on at-rest and passive earth pressures of overconsolidated collapsible soil on retaining walls were conducted. A prototype

model of a vertical wall, retaining horizontal backfill of collapsible soil, was developed in the laboratory. Collapsible soil was prepared in the laboratory by mixing kaolin clay with fine sand. The model was instrumented to measure the earth pressure at strategic points on the wall, the total earth pressure acting on the wall, and the overconsolidation ratio (OCR) of the soil. Tests were conducted on the wall retaining collapsible soil at the dry and at fully saturated conditions for both the at-rest and passive earth pressures. The test results showed that both the at-rest and passive earth pressures increased with the increase of the collapse potential ( $C_p$ ) and overconsolidation ratio (OCR) for the dry soil. At full saturation, the at-rest and passive earth pressures reduced considerably. Generally, the higher the collapse potential of the collapsible soil, the larger the decrease in the at-rest and passive earth pressures of the soil when the soil gets inundated.

In this investigation, for the case of at-rest earth pressure, empirical formulae were developed to determine the coefficient of at-rest earth pressure ( $K_0$ ) of the dry and saturated overconsolidated collapsible soil. For the case of passive earth pressure, analytical model was developed. Accordingly, design theory was presented for estimating the coefficient of passive earth pressure ( $K_p$ ) of overconsolidated collapsible soil at the dry and saturated conditions. Moreover, considering for the case of presence of unsaturated states of collapsible soil in practice, design charts were developed to assist designers in approximating the coefficient of at-rest ( $K_0$ ) and passive ( $K_p$ ) earth pressures of this soil at different degree of saturation.

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## LIST OF SYMBOLS

$b$  = width of the retaining wall

$C_p$  = Collapse potential

$C_c$  = Coefficient of curvature

$C_u$  = Coefficient of uniformity

$c'$  = Cohesion

DAS = Data acquisition system

$e$  = void ratio

$e_0$  = initial void ratio

$e_L$  = void ratio at liquid limit

FS = factor of safety

$G_s$  = Specific gravity

$H$  = height of the retaining wall

$h_0$  = initial specimen height

$I_m$  = maniability index

$K_0$  = Coefficient of at-rest earth pressure

$K_p$  = Coefficient of passive earth pressure

LL = Liquid limit

LVDT = Linear variable displacement transducer

OCR = overconsolidation ratio

PI = Plasticity index

PL = Plastic limit

$P_p$  = total passive earth force

$P_s$  = surcharge loading

$S$  = Degree of saturation

$S_c$  = settlement of the soil layer

$\alpha$  = the slope of the slip surface with the horizontal

$\beta$  = the inclined backfill angle

$\gamma$  = unit weight of soil

$\gamma'$  = effective unit weight of soil

$\delta$  = the wall friction angle

$\sigma_h$  = horizontal earth pressure

$\sigma_v$  = vertical earth pressure

$\sigma_p$  = passive earth pressure

$\phi$  = friction angle of the soil

$\phi_{cr}'$  = critical state friction angle

$\theta$  = angle of back side of wall and a horizontal line

$\mu$  = poisson's ratio

$\omega$  = moisture (water) content

$\omega_0$  = initial water content

$\Delta e$  = change in void ratio

$\Delta h$  = change in specimen height

$\Delta L$  = horizontal wall displacement

# CHAPTER 1

## INTRODUCTION

### 1.1. General

Collapsible soils are in the group of problematic soils, and can be found in many regions around the world including America, Asia, Africa, and Europe; in which, collapsible soils distribution has occupied approximately 17% of the United States, 17% of Europe, 15% of Russia and Siberia, and the large areas of China (Ayadat and Hanna, 2013; Iranpour and Haddad, 2016). Figure 1.1 presents the world distribution of collapsible soil (loess) (Catt, 1986). Collapsible soils exhibit low compressibility and high shear strength at natural water content levels (low water content), and if wetted they collapsed under their self-weight and/or surcharge loads, and loss of strength (Jiang et al., 2012).



**Fig. 1.1.** World distribution of collapsible soil (Loess) (Catt, 1986)

Nowadays, industrial and urban developments have recently exposed geotechnical engineers to relatively significant challenges when collapsible soils are presented in construction fields. The rise of ground water table or water used for farming, industrial development or water from other natural or human activities can cause severe damages to structures rested on collapsible soils. Many structures built on or in collapsible soils failed due to the soil collapse upon becoming wet. Consequently, the structures resting on collapsible soils have been facing to a number of potential problems. Collapse and problems related to collapse such as differential settlement, earth cracks, landslides and falls, instability in the slope supporting the structure, failure occurrence in the side walls of the dam reservoirs, have been responsible for serious damages to the infrastructures constructed on collapsible soil, consisting of loss of human lives (Garakani et al., 2015). For example, in China, Li et al. (2016) presented that a total of about 1505 buildings damaged and about 80km long underground pipelines ruptured due to collapse of collapsible soils were recorded from 1974 to 1975. It was noted that the cost associated with collapsible ground preparation was approximately one third of the total cost of the infrastructure (Li et al., 2016). Another example, the sinkholes developed in the collapsible soil along the northern coast of Egypt, in Al-Ghrabaneyat area, damaged the railway line nearby. It was difficult and costly to keep the railway in a regular working condition (Figure 1.2). Also in that area, two more buildings suffered from large and irregular settlements, seriously damaging the structure (Sakr et al., 2008).

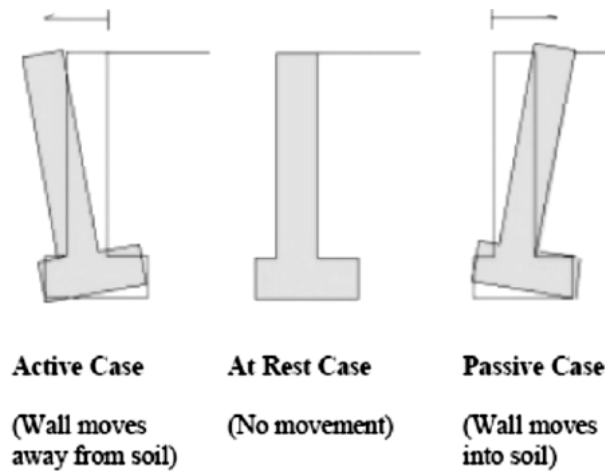
Generally, collapsible soils take a strong reaction to inundation, and it is not easy to predict and control in advance. Once inundated, the soil volume reduction takes place fast and suddenly and no approach can be used to stop the trouble at that time. It is difficult, time-consuming, and costly for repairing infrastructures damaged due to collapse of collapsible soils.



**Fig. 1.2.** (a) Sinkholes developed nearby the railway line; (b) Filling the voids by grouting as soil treatment (Sakr et al., 2008)

In general, all soils including natural soil deposits or compacted soils are susceptible to collapse due to wetting. The amount of collapse or severity of collapse is depended on the so-called collapse potential “ $C_p$ ” of the soil. The problems of industrial and civil structures dealing with collapsible soils are determined by the degree of collapse potential. The higher the collapse potential is, the severer the problem occurs.

Retaining walls are widely used in the United States, Asian countries, and other countries around the world. Retaining walls have being used popularly in structures including basement walls, bulkheads, abutments, tunnels, or flood walls. Retaining walls are primarily designed to counter the lateral earth forces. Lateral earth pressures acting against the retaining wall are one of the most important concerns in the design of retaining wall. Typically, three types of lateral earth pressure (Figure 1.3) can be acted on the wall, including at-rest, active, and passive earth pressures. These types of earth pressure can be developed due to the movement of the wall.



**Fig. 1.3.** Three different types of wall movement (Bowles, 1968)

Active and passive earth pressures are evolved on the wall when the movement of the wall are sufficient to mobilize full shear strength capacity of the soil. The active earth pressure is developed on the wall when the wall is allowed to move freely away from the retaining soil. In this case, the lateral stress in the soil mass decreases sufficiently to reach a minimum value known as the active failure state. Whereas, the passive earth pressure is developed when the wall moves toward the backfill soil, causing soil compression behind the wall. In this case, the lateral stress in the soil mass increases sufficiently to reach a maximum value known as the passive failure state. The passive earth pressure is also known as the resisting force to the active movement of the wall.

In practice, backfills behind bridge abutments or plate anchors are heavily under passive earth pressure.

At-rest earth pressure is occurred in the situation where the retaining walls have no lateral movement or the lateral movement of the wall is not adequate to develop the soil shear strength capacity. Basement walls or tunnels are typical examples of earth retaining structures facing to at-rest earth pressure.



## **1.2. Statement of the problem**

With civilization, it is impossible to avoid construction on/in collapsible soils. Construction on such kind of soils has involved a number of potential problems, the retaining wall is not an exception. Earth pressures evaluation plays an important role in geotechnical engineering. An accurate prediction of the earth pressure acting on the retaining wall is significant to the safe design of many infrastructures.

Unfortunately, in the literature, there is lack of studies about the case of retaining wall with backfill of overconsolidated collapsible soil. Consequently, no design theory for predicting the at-rest and passive earth pressures acting on the wall retaining overconsolidated collapsible soil subjected to inundation are found. In the literature, Jaky, Coulomb, and Rankine's theories are the most common and reliable methods for calculation of lateral earth pressures acting on the wall. Besides that, numerous theories have been developed and reported. However, those methods and attempts have been limited for cases of backfills of normally and overconsolidated cohesionless soils or cohesive soils. They cannot be applied for the case of inundation of backfill collapsible soil because the soil will experience reduction in strength and excessive settlement upon inundated, which further influence the earth pressure of this soil on the walls.

It is no doubt that a remarkable decrease in the earth resistance of the soil after inundation can result in severe problems to many infrastructures.

## **1.3. Research Objectives**

The main goal of this research is to provide methods to predict the at-rest and passive earth pressures of overconsolidated collapsible soil at the dry and saturated states. It is also to establish

the effect of the collapse potential and the overconsolidation ratio of the soil on the at-rest and passive earth pressures. Therefore, the objectives of this thesis are listed as follows:

- a. To conduct a literature review and to prepare a state-of-the-art report on collapsible soils and theories / studies on the at-rest and passive earth pressures.
- b. To develop a prototype model of a vertical wall retaining horizontal backfill of overconsolidated collapsible soil in the laboratory to investigate the effect of inundation due to the rise of ground water table on the at-rest and passive earth pressures. This model is instrumented to measure the at-rest and passive earth pressures at selected points on the wall, the total earth pressure acting on the wall, and the overconsolidation ratio (OCR) in the soil mass.
- c. To examine the role of the soil collapse potential  $C_p$  and stress history (overconsolidation ratio - OCR) on the at-rest and passive earth pressures.
- d. To develop theories to estimate the values of coefficients of at-rest ( $K_0$ ) and passive ( $K_p$ ) earth pressures for both dry and saturated soil conditions.
- e. To develop design charts for predicting the values of  $K_0$  and  $K_p$  of unsaturated collapsible soil at different degree of saturation.

## **CHAPTER 2**

### **LITERATURE REVIEW**

#### **2.1. General**

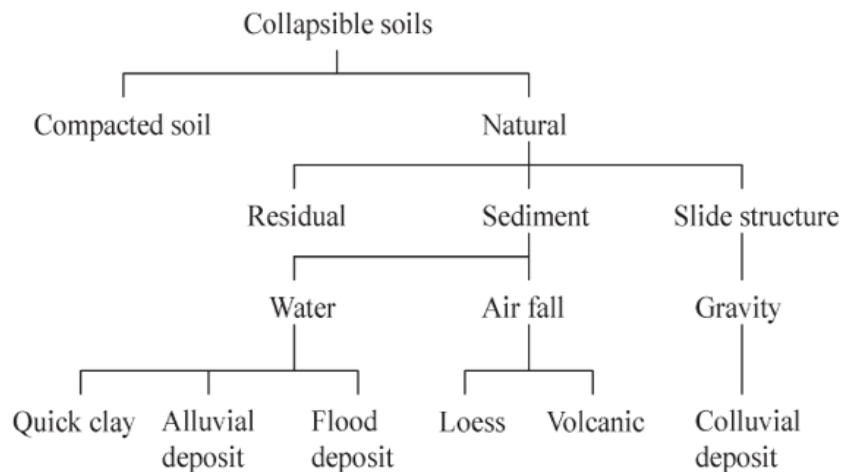
This chapter presents the state-of-the art literature review with emphasis on the background about collapsible soils and lateral earth pressure acting on the retaining wall. The theories and studies on the at-rest and passive earth pressures are also reported.

#### **2.2. Literature Pertinent To Collapsible Soil**

Collapsible soils are mostly found in form of unsaturated soil at low initial water content, which are susceptible to loss of strength due to decrease in bulk volume upon becoming wet. Generally, collapsible soil deposits share two typical features. Firstly, they are loose and cemented deposits. Secondly, they are naturally near dry. Collapsible soils own loose structure with particles joined together commonly by chemical cement or clay bond. In their natural unsaturated condition, collapsible soils are strong and stable thanks to the cemented or bonded nature. However, due to their unstable soil fabric and weak inter-particle bond strength, the unsaturated collapsible soils are susceptible to significant volumetric decrease upon wetting. Bond strength is easy to be weakened by water, which causes collapse and loss of strength. (Ali, 2015; Haeri and Garakani, 2016).

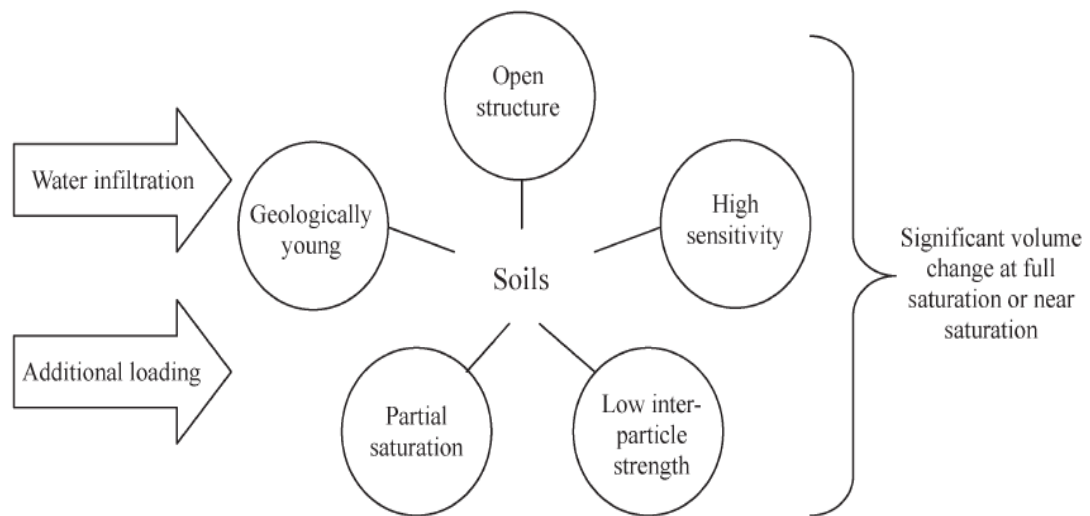
There are plenty of sources causing collapsible soils wetted, including but not limited to heavy and continuous rainfall, excessive irrigation, broken water or sewer lines, other kinds of artificial flooding from the surface, or upward water saturated from rising ground water table (Haeri, 2016; Iranpour and Haddad, 2016).

Collapsible soil is basically classified into two groups including natural occurring collapsible soils and man-made soils (Clemence and Finbarr, 1981; Mossaad et al., 2006). Natural deposits involve Aeolian deposits, water-laid deposits, and residual soils. The most substantial natural deposits of collapsible soils are Aeolian or windblown deposits –well known as loess. Loess is thick, yellowish or brown deposit of windblown dust. It is a product of past glacial activity in an area. Water laid deposits can form alluvial fans, flows, and flowslides. Resident soils are the product of weathering of rocks. The particle size of the residual materials can be various from large fragments to gravel, sand, silt, colloids, and even organic material. Other soil types such as volcanic tuff, gypsum, loose sands cemented by soluble salts, dispersive clays, or sodium-rich montmorillonite clays may exhibit collapsible behavior upon wetting. Man-made soils consisting of compacted soils, construction debris, mine tailings, and coal ash fills may also exhibit collapse. Figure 2.1 shows classification of different types of collapsible soils (Rogers, 1995).



**Fig. 2.1.** A family of collapsible soils (Rogers, 1995)

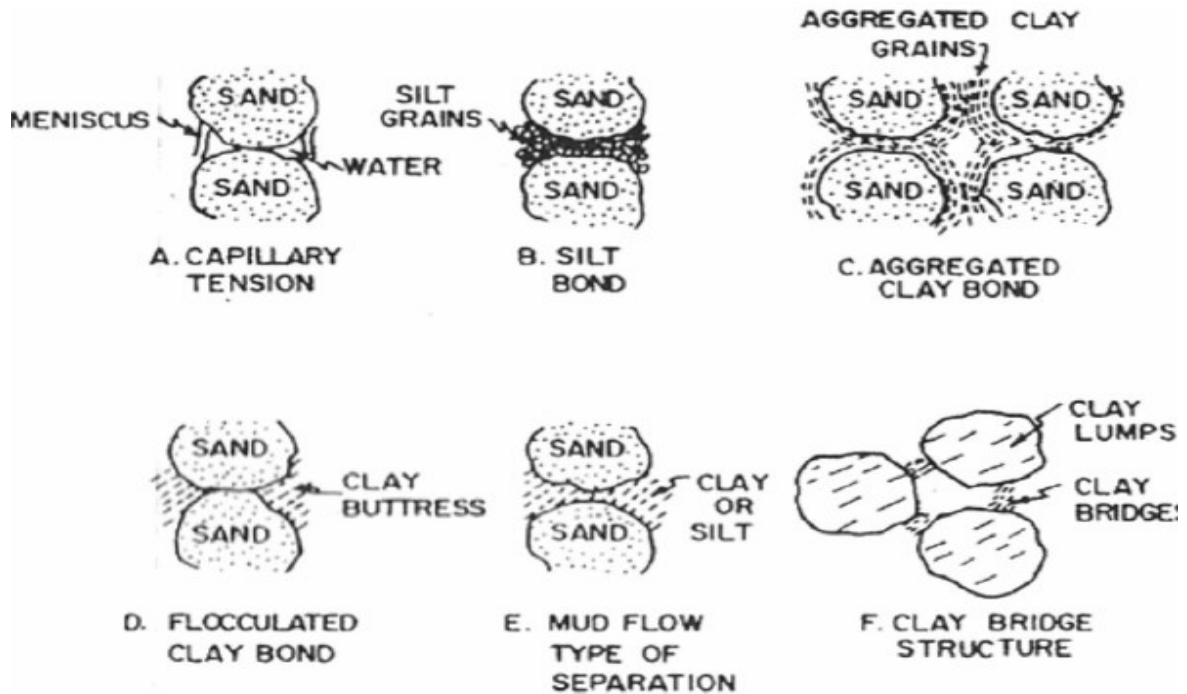
Collapsible soils are generally characterized by some specific engineering properties including high void ratio, low natural density, low natural water content, great dry strength and stiffness, zero or slight plasticity, and high percentage of fine grained particles (Howayek et al., 2011/12; Haeri, 2016). Figure 2.2 illustrates the schematic view of main characteristics of collapsible soils.



**Fig. 2.2.** Schematic view of key characteristics of collapsible soils (Howayek et al., 2011/12)

Collapsible soil basically possesses a honey-comb, open, and unstable structure of bulky shaped grains with the grains held in place by inter-particle bonding material such as cementation, chemical, physical attraction or soil suction (negative pore pressures) (Mossaad et al., 2006). When the support is taken off by addition of water, the grains slide over each other, moving into void spaces. In general, the greater the water content, the lower the bond strength. Physically, all collapsible soils are weakened by adding water. Immediate collapse in strength is found in the case where the grains are held together by capillary suction, slow collapse in strength in the case of chemical cementing, and much lower collapse in strength in the case of clay buttresses (Clemence

and Finbarr, 1981). Figure 2.3 presents different types of inter-particle bonds for collapsible soils that reported by Clemence and Finbarr (1981).



**Fig. 2.3.** Different inter-particle bonds in collapsible soil (Clemence and Finbarr, 1981)

The collapse mechanism accompanying wetting is based on the reduction of inter-particle bonds, followed by the collapse of the soil structure. The collapse occurs in the soil with three main phases (summarized by Li et al. 2016). The first phase (pre-collapse phase) occurs when the soil is subjected to high values of matric suction. In this phase, the soil experiences small volumetric deformations due to matric suction decrease, no slippage occurs between the particles and the structure of the soil still remains intact. The second phase (collapse phase) occurs when the soil undergoes intermediate values of matric suction. In this phase, the soil experiences a significant volume decrease due to matric suction decrease. The structure of the soil is also adjusted

due to bonding break. The third phase (post-collapse phase) occurs when the unsaturated soil reaches full saturation. In this phase, the soil is not subjected to further volume decrease due to matric suction decrease. Klukanova and Frankovska (1995) proposed the process of collapse mechanism, consisting of three phases. Phase 1: the original microstructure is demolished due to increasing humidity (soil suction reduced) and external total stress. Phase 2: it is involved the disintegration of microstructure. Phase 3: a new microstructure is formed after the collapse is completed.

Identification of collapsible soils is very important to the geotechnical field. The practical engineers often require a fast approach of indicating the collapsible soils. Geotechnical zonation maps can be used for identifying the collapsible soils preliminarily. These zonation maps have been established and developed based on actual boreholes, laboratory tests, and field tests. The maps may provide helpful information about the location and depth of collapsible deposits. (Ayadat and Hanna, 2013)

#### *Collapse criteria based on soil parameters*

The methods available in the literature can be grouped into four categories (Ayadat and Hanna, 2012) as follows:

- Methods based on void ratio, density, and water content relationship
- Methods based on water content and Atterberg limits relationship
- Methods based on density and Atterberg limits
- Methods based on particle size distribution of soils

1. Methods based on void ratio, density, and water content relationship

Denisov (1951) introduced a coefficient of collapse  $K$  as:  $K = \frac{e_L}{e_0}$  (2.1)

Where:  $e_0$  = void ratio at nature, and  $e_L$  = void ratio at liquid limit

The degree of soil collapsibility could be determined as follows:

$K=0.5-0.75$ : highly collapsible soil

$K=0.75-1.0$ : moderate collapse

$K>1.0$ : non-collapsible loam

$K=1.5-2.0$  non-collapsible soil

Clevengar (1958) stated the criterion based on natural dry unit weight as follows:

$\gamma_d < 12.6 \text{ kN/m}^3$  : highly collapsible

$12.6 \text{ kN/m}^3 \leq \gamma_d \leq 14.0 \text{ kN/m}^3$  : may be collapsible

$\gamma_d > 14.0 \text{ kN/m}^3$  : non-collapsible

2. Methods based on water content and Atterberg limits relationship

Freda (1964) introduced the criteria based on collapse index as follows:  $K_L = \frac{m - PL}{PI}$  (2.2)

Where:  $m$  = natural moisture content

$S_r$  = degree of saturation

$PL$  = plastic limit

$PI$  = plasticity index

The soil is susceptible to collapse if the collapse index  $K_L > 0.85$



### 3. Methods based on density and Atterberg limits

Beckwith (1979) proposed use of typical index properties to identify collapsible soils. Collapsible soils could be demonstrated by a value of plasticity index  $PI < 10$ , the unit weight  $\gamma_d < 14.9 \text{ kN/m}^3$  and moisture content  $\omega < 4\%$  to  $8\%$ .

### 4. Methods based on particle size distribution of soils.

Handy (1973) introduced the criteria for Iowa loess in terms of clay content ( $< 0.002\text{mm}$ )

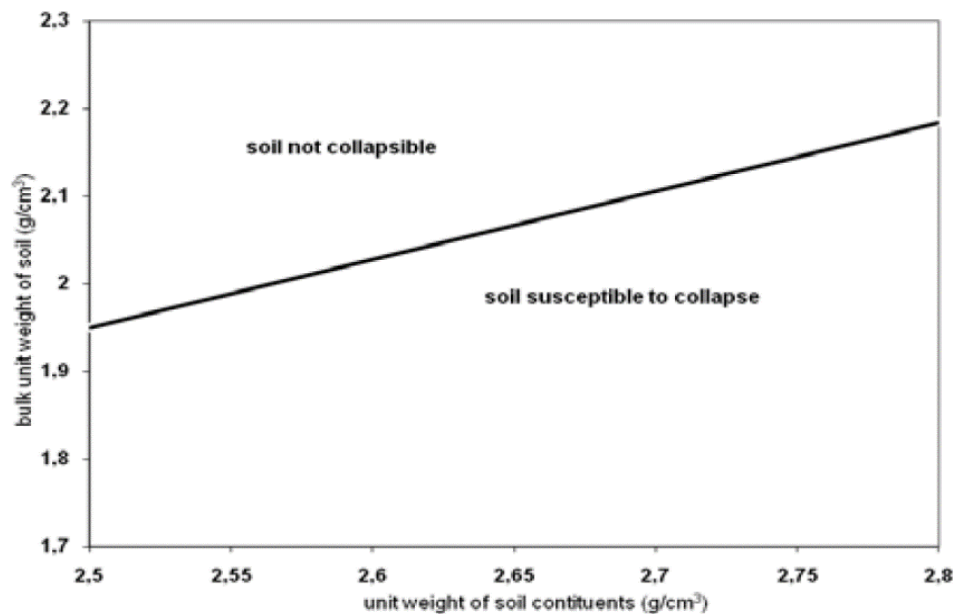
Clay content  $< 16\%$  : the probability of collapse is high

Clay content:  $16\text{-}24\%$  : the probability of collapse is moderate

Clay content:  $24\text{-}32\%$  : the probability of collapse is less than  $50\%$

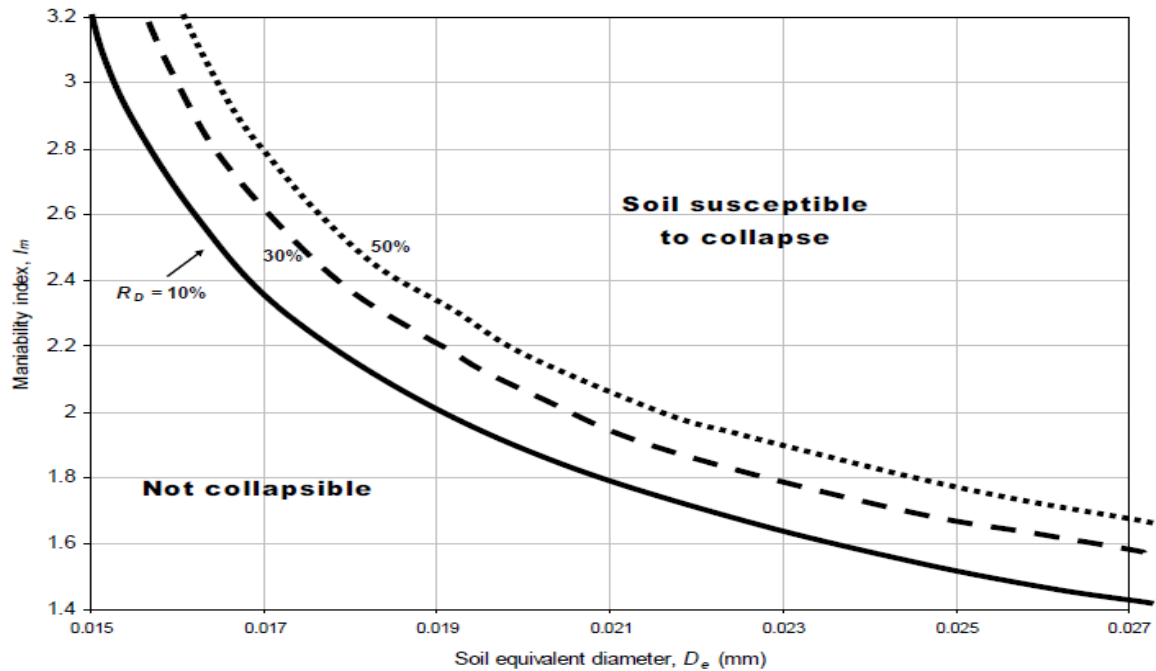
Clay content :  $> 32\%$  : the probability of collapse is nearly zero

Ayadat and Hanna (2012) suggested the criterion for the susceptibility of soil to collapse based on bulk unit weight of soil and unit weight of soil constituents. This criterion was presented by a chart as below:



**Fig. 2.4.** A chart of bulk unit weight against unit weight of soil constituents (Ayadat and Hanna, 2012)

In addition, the collapse criterion based on relative density, soil equivalent diameter, and the maniability index which was defined as  $I_m = \frac{w_L - w_0}{IP}$  (2.3) (IP: plasticity index,  $w_0$ : initial water content,  $w_L$ : liquid limit) was developed for practical application by Ayadat and Hanna, (2013), as shown in Figure 2.5.



**Fig. 2.5.** Design chart for identification of soil collapse behavior (Ayadat and Hanna, 2013)

*Prediction methods based on laboratory tests*

The most common way for measurement of collapsibility is to conduct single oedometer test (Vakili, 2013; ASTM D5333-03). The single oedometer test consists of several steps. Firstly, soil specimen (within the oedometer ring) at natural water content is placed in the oedometer apparatus. Secondly, a predetermined vertical stress is gradually applied to the soil specimen until reaching a maximum load of 200 kPa. Thirdly, the soil specimen is inundated while the load is still maintained on the soil specimen. The relative soil compression (in terms of void

ratio) is determined from the measurements of vertical settlements of the specimen. The collapse potential ( $C_p$ ) is equal to the deformation of soil due to the addition of water, divided by the initial height of the specimen.

The collapse potential ( $C_p$ ) is the collapse strain due to inundation of the specimen under 200 kPa

pressure in oedometer apparatus: 
$$C_p = \frac{\Delta h}{h_0} \cdot 100\% = \frac{\Delta e}{1 + e_0} \cdot 100\% \quad (2.4)$$

Where

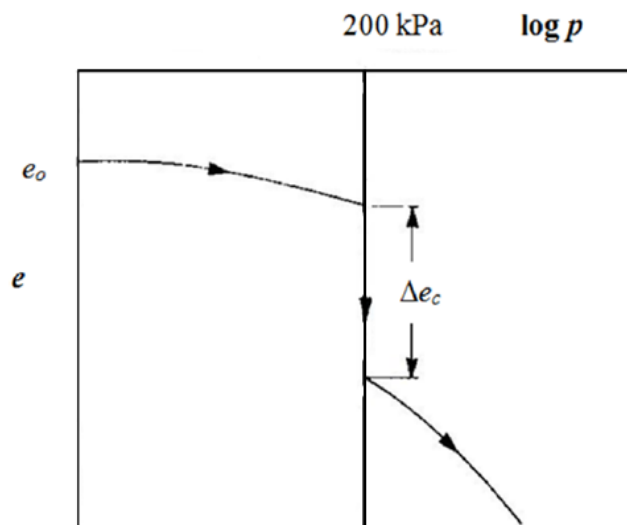
$\Delta h$  = change in specimen height due to wetting under 200 kPa pressure (mm)

$h_0$  = initial specimen height (mm)

$\Delta e$  = change in void ratio due to wetting under 200 kPa pressure

$e_0$  = initial void ratio

Figure 2.6 depicted the typical collapse test result which presented by Jennings and Knight (1975).



**Fig. 2.6.** Typical collapse potential test result (Jennings and Knight, 1975)

According to ASTM D5333-03, the classification of collapse potential is illustrated in Table 2.1.

**Table 2.1.** Classification of collapse potential (ASTM D5333-03)

Degree of collapse	Collapse potential (%)
None	0
Slight	0.1 to 2.0
Moderate	2.1 to 6.0
Moderately severe	6.1 to 10.0
Severe	>10

Jennings and Knight (1975) proposed a classification for severity of foundation problems with respect to collapse potential  $C_p$  as table below:

**Table 2.2.** Collapse potential and severity of problems of foundation (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

Collapse potential ( $C_p$ ) can be employed to roughly calculate settlement that may occur in a collapsible soil layer at a certain place (ASTM D5333-03). The settlement of the soil layer might be approximated as:

$$S_c = C_p \frac{H}{100} \quad (2.5)$$

Where:  $S_c$  = settlement of the soil layer upon wetting

$C_p$  = collapse potential

$H$  = thickness of the soil layer.

Many treatment methods for collapsible soils have been studied and suggested by a number of scientists and researchers (Mossaad et al., 2006; Houston et al., 2001; Soliman and Hanna, 2010; Ayadat and Hanna, 2005; Jefferson et al., 2008; Mohamedzein and Al-Rawas, 2011; Abbeche et al. 2010; Mohamed and Gamal, 2012; Ali, 2015). The mitigation methods differ in both concept and methodology to go well with a certain problem. To choose which approach should be used depends on many factors including the site condition, the depth of the collapsible soil layer, and the thickness of the collapsible soil layer.

These treatment methods consist of, but not limited to, removal and replacement, removal and presume compaction, avoidance of wetting, pre-wetting, controlling wetting, preloading, stone columns, encapsulated stone columns, dynamic compaction, pile or pier foundation, chemical stabilization (e.g. cement, salts, sulfur cement, sodium silicate solution, lime, and phosphoric acid), grouting, and reinforcement using geosynthetics.

- Removal and compaction: this method can be applied before construction, with the collapsible soils are situated shallowly.

- Pre-wetting (increase in moisture content) simply means that the collapsible soils are wetted before constructing structures, so that the settlement due to soil collapse will be small. This method is often used in cases where the collapsible soils are identified prior to structures built.

This approach is not very effective in the case where collapsible soil layers are located at a great depth (more than 5m).

- Controlling wetting: the method is similar to pre-wetting, except that it is used when the structures are constructed.

- Chemical stabilization or grouting can strengthen the collapsible soils by introducing a chemical compound to add cementation that enhances inter-particle bonds. This can reduce the soil collapse upon becoming wet. This approach may not be economical if heavy structures will be constructed in the soil improvement area.

- Dynamic compaction can lessen the void ratio but it is not sufficient to eliminate the collapse problem. This technique can provide a significant improvement up to 2.5 to 3 m in the area with deep ground water level.

- Geosynthetics materials benefit in increasing bearing capacity and decreasing settlement of foundation on/in collapsible soils.

- Piles, piers, stone columns, or encapsulated stone columns have been employed as a mitigation alternative. The load is transferred through the collapsible soil layer to a good soil layer such as stiff clay, dense sand, or bedrock. The approach can be used effectively for heavy structures built on the collapsible soil area, but costly.

The choice of one or combination of these techniques depends on numerous parameters consisting of timing of mitigation, source of loading, source of wetting, nature of ground to treat, environmental conditions, and economic considerations.

Haeri, Khosravi et al. (2014) attempted to assess the effect of soil disturbance on the hydro-mechanical behavior of the highly collapsible loessial soil during wetting by conducting modified suction-controlled triaxial tests on undisturbed and reconstituted loessial soil specimens taken from Golestan province of Iran. The natural composition of the soil matrix, the non-homogenous distribution of macro or micro pores, and the weak inter-particle bonding between grains were listed as a group that was significantly influenced by the disturbance of highly collapsible loessial soils samples. The authors reported that due to differences in the soil pore size and pore size distribution, the capacity for water retention and consequently the stress state of the soil could be strongly different for tests conducted on undisturbed and reconstituted samples. An increase in volumetric strain was recorded during the wetting process due to the decrease in matric suction. They also found that for the low mean net stress testing, the deformation of measurements for undisturbed samples were lower than that for reconstituted samples. For the higher mean net stress testing, a similar wetting-induced volumetric strain behavior were found for both undisturbed and reconstituted samples. The author believed that this behavior was resulted from the effect of the mechanical stress on the natural composition of the soil matrix, which could break the cemented bonds between soil particles and decrease the effect of non-homogeneity in the distribution of pores on the deformation behavior of the loessial soils.

Zhou et al. (2014) attempted to examine the soil behavior and failure mechanism of loess slopes subjected to water infiltration in Heifangtai Plateau in Gansu province of China. They reported that landslides in the area could be triggered by water infiltration through two different processes including wetting of soil at a shallow depth to due short term rainfall or irrigation, and fully saturating due to rising of groundwater table in deep soil resulting from prolonged irrigation. They conducted a series of laboratory tests such as undrained compression on the anisotropically

consolidated saturated samples, drained shear by decreasing mean effective stress at constant axial load on saturated samples, and wetting by decreasing matric suction at constant axial load on unsaturated samples. Based on their experiments, they stated that under constant axial load condition, the saturated loess experienced two modes of failure with increasing pore water pressure. At a lower stress level, the initially drained deformation might evolve into an undrained sudden failure; whereas, at the higher stress level, progressive failure was found. They also supposed that in saturated loess, at low stress, a rapid increase in pore water pressure at constant axial load was due to the main triggering mechanism of loess landslides. They additionally suggested that the improvement of the flooding irrigation method and the construction of a vertical drainage system penetrated to the pebble stone layer could be taken as a serious consideration in remediation measures for the area.

Haeri, Garakani et al. (2014) attempted to characterize the deformation behavior of a highly collapsible undisturbed loessial soil under changing mean net stress or matric suction conditions, using a modified triaxial test device with suction and saturation controls. The soil specimens were taken from the Hezar Pich Hills area near the city of Gorgan, Iran. They found that the hydro-mechanical behavior of collapsible soil was considerably stress-path dependent. With the same mean net stress values, the deformation measurements of samples subjected to isotropic compression were frequently larger than that subjected to wetting-induced collapse. The soil specimens subjected to isotropic compression underwent a decrease in their volumetric strain as the mean effective stress increased; whereas, the soil specimens subjected to wetting-induced collapse experienced a decrease in volumetric strain when the effective stress was decreased during the wetting process. They also presented that for the isotropic compression tests, the soil water



retention curve of the loessial soil depended on the mean net stress; while less sensitivity to the mean net stress were recorded for the specimens subjected to wetting-induced collapse.

Garakani et al. (2015) investigated the hydro-mechanical behavior of loessial soil during isotropic and shear loadings using the modified triaxial test device. Tests were conducted on undisturbed specimens taken from loessial deposits in Gorgan city, Iran. The authors reported that the hydro-mechanical behavior of loess was essentially affected by the extent of applied mean net stress and the degree of suction. The magnitude and extent of collapse were different, depending on the applied state of the stress and the hydro-mechanical loading path. They also presented that the collapse phenomenon in natural loess was mostly a continuous – stepwise reduction in volume rather than a sudden reduction as water enters the voids. It was found that any increase in the magnitude of mechanical stresses (confining net stress or shear stress) or degree of saturation (wetting process) led to an increase in the magnitude of collapse. Generally, an increase in isotropic loads, suction, and shear loads caused an increase in the amount of collapse.

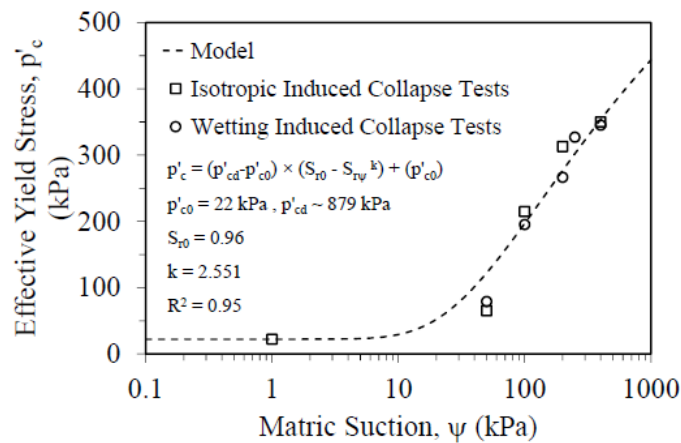
Ali (2015) presented the field plate load tests aiming to understand the behavior and performance of compacted sand replacement over treated collapsible soil by pre-wetting and compaction. The field tests were conducted on collapsible soils of Borg Al-Arab area, Egypt. The author reported that replacement soil improved the stability of collapsible soil by uniform distribution of water on surface. Increase in thickness of compacted sand replacement layer led to increase in the bearing capacity of inundated compacted collapsible soil. Significant settlements were found in foundations of structures on collapsible soils after being saturated, resulting in serious structure damage. It was suggested that in order to densify and stabilize collapsible soils, the collapsible soil should be flooded before construction. The author also found that the total amount of collapsibility potential depended on initial moisture content, extent of wetting depth,

duration of wetting, and the pattern of moisture migration. Finally, it was concluded that saturation and preloading of collapsible soil before construction could be helpful to stabilization of collapsible soils.

Haeri (2016) presented the studies taken place at Advance Soil Mechanics Laboratory of Sharif University of Technology, Tehran, Iran to study the intact behavior of the collapsible loessial soils taken from Hezar Pich Hill in the city of Gorgan, Iran. The author conducted tests using automatic unsaturated oedometer and fully automatic unsaturated triaxial devices. The author reported that the hydro-mechanical behavior of intact collapsible loessial soils was essentially different from that of the reconstituted samples. The water absorption and associated volume change or wetting collapse of the intact samples was stepwise and generally greater, while the wetting collapse of the reconstituted samples experienced almost linearly increasing upon becoming wet and smaller. The author also stated that the shear tests on reconstituted samples overestimated the shear strength of the natural loess. Thus, the author recommended that the reconstituted specimens of loess should not be used to predict the behavior of natural loessial soils. It was recommended to conduct undisturbed sampling and tests on intact natural loess to predict accurately the collapse potential, hydro-mechanical behavior, and shear strength of this type of collapsible soil.

Haeri and Garakani (2016) studied the hardening behavior of the collapsible loessial soil by conducting modified suction-controlled triaxial tests on undisturbed loess samples taken from Hezar Pich Hill in Gorgan city, Iran. The authors stated that collapse and reduction in volume of the specimen was due to suction decrease, and an elastic rebound happening due to the mean effective stress decrease. Additionally, the amount of volume reduction due to pore collapse was significantly higher than the amount of dilation and elastic rebound due to the effective stress

decrease. Consequently, the resultant was volume decrease while effective stress decreased at the same time. The authors also found that during the test, the soil samples could undergo hardening or softening, depending on the level of applied mean net stress or matric suction. They also provided a coupled constitutive model to show the relation between the values of mean effective yield stress, degree of saturation, and matric suction for unsaturated collapsible loessial soils (Figure 2.7).



**Fig. 2.7.** Predicted and tested values of  $P_c'$  with matric suction ( $\psi$ ) (Haeri and Garakani, 2016)

Haeri et al. (2016) attempted to study the role of sample disturbance and soil structure in the hydro-mechanical behavior of collapsible soils by conducting modified suction-controlled triaxial tests on intact and reconstituted specimens of the loessial soil taken from Golestan province of Iran. The author found that the collapse phenomenon in tact loessial samples was mostly a stepwise reduction in volume rather than a continuous decrease with a constant rate or a sudden drop in volume as water added. They supposed that this behavior could be explained by the non-homogenous distribution of macrospores or microspores in intact specimens which led to the presence of void spaces with different levels of collapse potential within soil matrix. They also presented that the intact samples were highly contractive and experienced mostly ductile shear

strength behavior; whereas, the reconstituted samples seemed to undergo strain softening and dilatancy after failure. The authors additionally reported that the shear strength values of reconstituted samples were higher than that of intact samples when sheared at the same applied mean stresses and matric suctions. Thus, it was concluded that the specimen disturbance could result in the underestimation of volume change due to collapse and overestimation of shear strength. The authors stated that the shear strength from tests conducted on reconstituted specimens was overestimated for the natural soil, due to the different structures of the intact and reconstituted samples. Based on their experiments, the hydro-mechanical behavior of loessial soils was significantly affected by specimen disturbance and the soil structure.

Li et al. (2016) summarized a state of the art review on collapse mechanism with special reference to loess soil deposits, focusing on three different categories including traditional approaches, microstructure approaches, and soil mechanics-based approaches. The traditional approaches for interpreting the wetting-induced collapse mechanism consisted of loss of capillary tension, solution of soluble salts, shortage of clays, and under compaction. However, the traditional or conventional approaches were found to be unsatisfactory for universally explaining the collapse behavior of loess soils. These approaches were valuable to better understand collapsibility of local loess soil from simple tests. Microstructure approaches for interpreting wetting-induced collapse behavior were focused on four factors including particle pattern, contact relation, pore form, and bonding material. Among these factors, pore form and bonding material were proposed as the two dominant factors which have more influence on the collapse behavior. It was concluded that microstructure approaches were widely acknowledged to play an important role in controlling the mechanical behavior of loessial soils. However, the microstructure approaches could not provide quantitative descriptor. The soil mechanics-based approaches for interpreting the collapse

behavior provided the elastoplastic modeling for understanding collapse behavior. The elastoplastic models could define the yield surface for unsaturated soils, which obviously divided elastic and plastic deformations of unsaturated soils. Thus, these models could be a useful tool to explain the collapse phenomenon as soil yields or the stress path crosses the yield surface due to loading or wetting or both. It was found that these approaches based on concepts of elastoplastic breakage mechanics, such as the binary medium model and the block structural model, were validated to provide reasonable prediction of collapse behavior for natural loess soils. However, it was not easy to apply these models in conventional practice due to the difficulty in determining the parameters required for the models from time-consuming and resource-consuming experiments. Finally, it was recommended that future research studies should be focused on providing much simpler models with less parameters and parameters which are easy to determine from conventional tests.

Iranpour and Haddad (2016) conducted laboratory investigation aiming to understand the impacts of nanomaterials on collapsible soil behavior. The collapsible soil specimens were taken from arid and semi-arid regions of Iran. Then, the specimens were treated with four different types of nanomaterials, including nanoclay, nanocopper, nanoalumina, and nanosilica, and combined under different percentages of the total dry weight of the soil. Soil tests were carried out in natural water content and density. The authors stated that enhancing the resistance of clay and silt bridges as well as cementation factors in the structure of collapsible soils could reduce the danger likelihood of collapse because soil collapse highly depended on the structure of soil. They presented that the dry unit weight and the water content significantly affected the soil collapse potential. Based on their experiments, they found that the combination of soil and nanomaterials was very sensitive and the amount and the type of nanomaterials added to the collapsible soil could

get not only positive impact but also have negative impact on improvement of collapsible soils. It was concluded that the combination of nanomaterials with collapsible soils should be used at the most appropriate percentage to avoid negative effects resulting from adding nanomaterials more than the optimum value. The author also suggested that the negative effect could be reduced by combining nanoparticles with collapsible soil in the form of colloid solutions.

## **2.3. Literature Pertinent to At-Rest and Passive Earth Pressures**

### **2.3.1. General**

Earth retaining structures including, but not limited to, basement walls, bulkheads, abutment, and retaining walls are often met in foundation engineering. They are primarily designed to counter the lateral earth loads. Design of retaining structures requires knowledge and comprehension of lateral earth pressures that cause the lateral forces on the retaining structures.

**At-rest earth pressure:** The wall is static and it does not move toward or backward of its initial position (Figure 2.8). In this case, the earth pressure refers to the at-rest earth pressure ( $K = K_0$ : coefficient of at-rest earth pressure), and corresponds to a state of static equilibrium in the backfill soil mass. The resultant of at-rest earth pressure exerted on the wall is  $P_0$ .

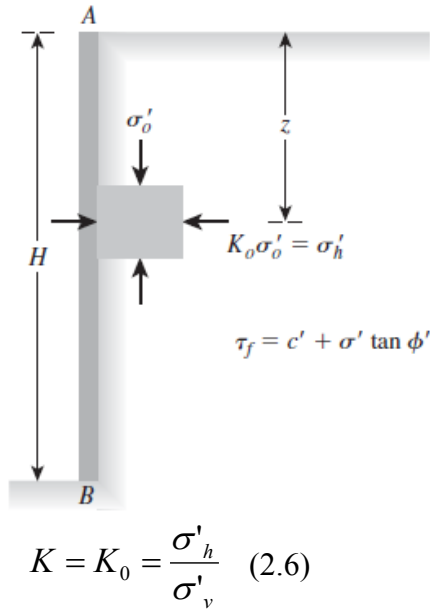


Fig. 2.8. At-rest earth pressure (Das, 2009)

**Passive earth pressure:** The wall moves toward the retaining soil (Figure 2.9). The earth pressure increases from at-rest to passive earth pressure ( $K = K_p$ : coefficient of passive earth pressure) due to soil compression behind the wall. The resultant of passive earth pressure exerted on the wall is the passive thrust  $P_p$ .

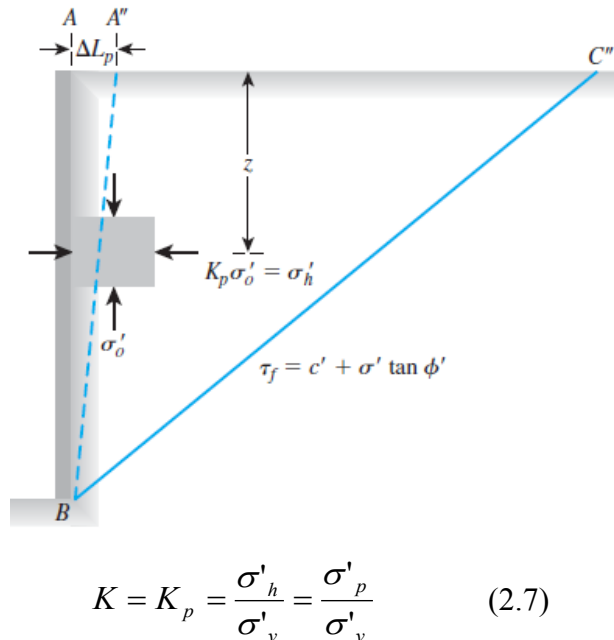
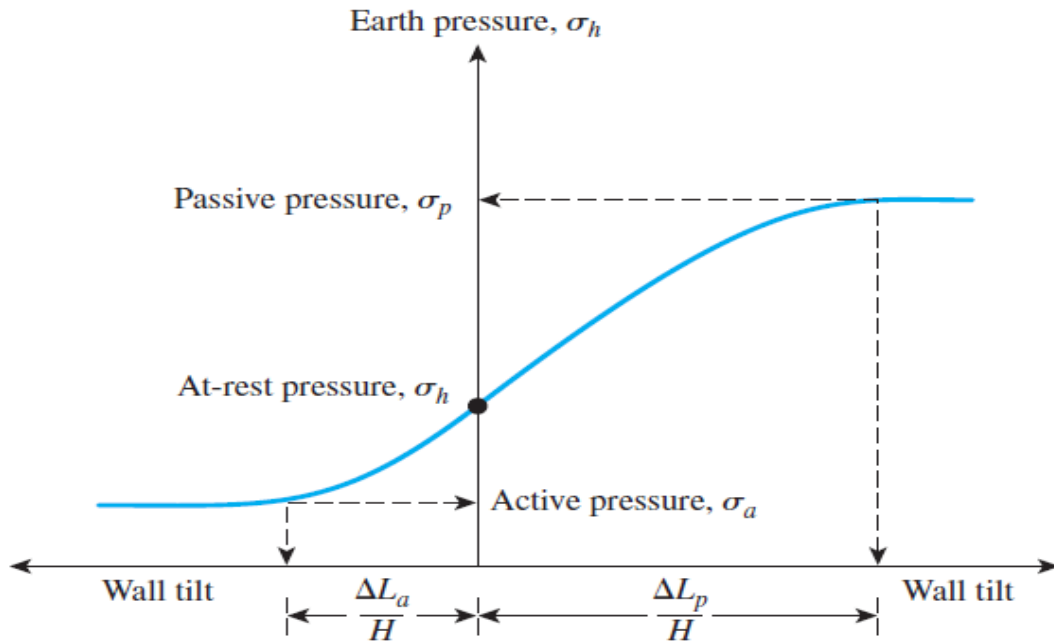


Fig. 2.9. Passive earth pressure (Das, 2009)

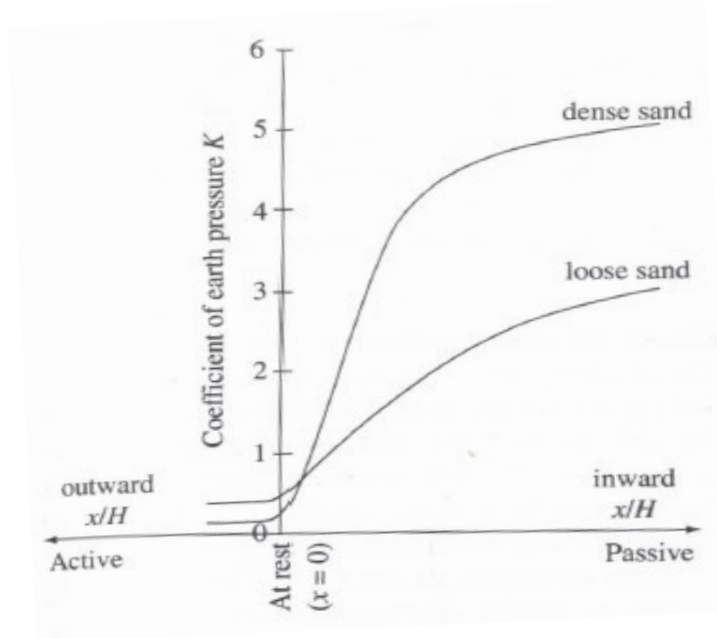
The relationship between wall movement and variation of the magnitude of lateral earth pressure is presented in Figure 2.10.



**Fig. 2.10.** Variation in lateral earth pressure with wall movement (Das, 2009)

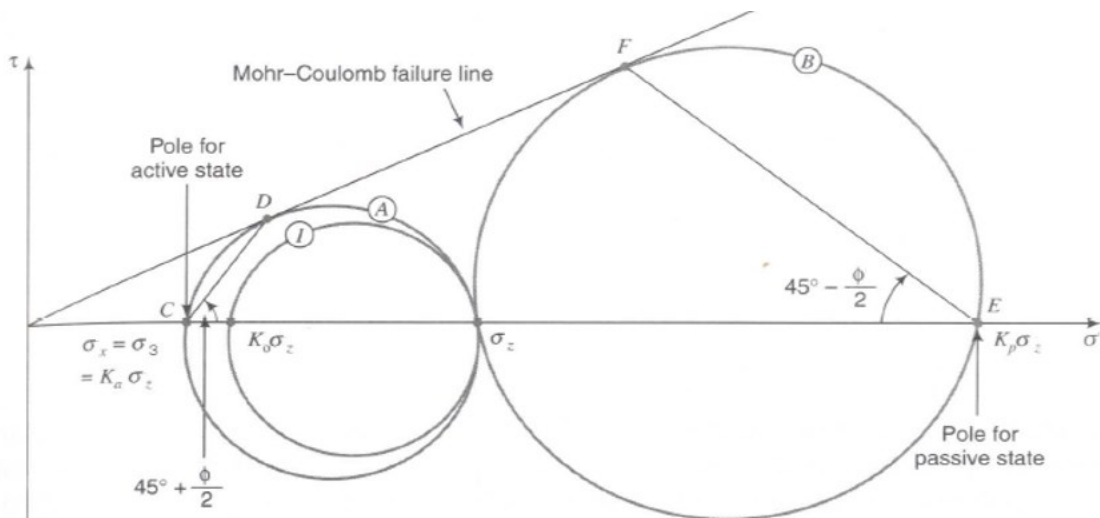
Typical values of coefficient of active and passive earth pressures of loose and dense cohesionless soils (Barnes, 2010) are depicted in Figure 2.11





**Fig. 2.11.** Typical values of earth pressure coefficient ( $K$ ) of cohesionless soils with wall movement (Barnes, 2010)

Mohr's circles for at-rest, active, and passive states are shown in Figure 2.12, in which: circle  $(I)$  presents for the at-rest state, circle  $(A)$  presents for the active state, and circle  $(B)$  presents for the passive state.



**Fig. 2.12.** Mohr's circles at-rest, active, and passive states (Budhu, 2008)

### 2.3.2. At-rest earth pressure

Generally, the at-rest earth pressure coefficient ( $K_0$ ) depends on the angle of shearing resistance ( $\phi'$ ), plasticity index (PI), and stress history - overconsolidation ratio OCR (Liu and Evett, 2008).

Jaky (1944) proposed the empirical equation for estimation of at-rest earth pressure coefficient ( $K_0$ ) for soils as follows:

$$K_0 = 1 - \sin \phi'_{cr} \quad (2.8)$$

Where  $\phi'_{cr}$  = critical state friction angle

However, the above Jaky's relationship just gives good results for backfill loose sand. The Jaky's relationship may grossly underestimate  $K_0$  for backfill dense sand, resulting from the process of compaction backfill (Das, 2009).

Brooker and Ireland (1965) investigated experimentally the influence of stress history on the at-rest earth pressure coefficient ( $K_0$ ) of remolded cohesive soils. Based on experimental results, the authors concluded that the at-rest earth pressure coefficient ( $K_0$ ) depended on the angle of shearing resistance ( $\phi'$ ), the plasticity index (PI), and the soil's stress history as stated by the preconsolidation load or the overconsolidation ratio OCR. They confirmed that Jaky's equation was more accurate for the calculation of  $K_0$  of cohesionless soils. For normally consolidated clay, a new empirical relationship was suggested as follows:

$$K_0 = 0.95 - \sin \phi'_{cr} \quad (2.9)$$

Mackey and Kirk (1967) conducted the laboratory investigation into at-rest pressures acting on a rigid steel wall. In this investigation, three different sands which ranged from the loose to dense states were used. The authors indicated that the at-rest earth pressure in loose sand was in reasonable agreement with those of Jaky theory  $K_0 = 1 - \sin \phi_{cr}'$ ; whereas, the at-rest earth pressure in dense sand was roughly equivalent to those of the simple Rankine passive pressure. It was confirmed again the Jaky's equation should not be used for backfill dense sand.

Alpan (1967) suggested a reasonably reliable procedure to roughly calculate the value of the at-rest earth pressure coefficient ( $K_0$ ). He suggested that the following relationship should be used for clays:  $K_0 = 0.19 + 0.233 \log(\text{PI})$  (PI – plasticity index – in %). (2.10)

Wroth (1973) proposed the empirical formula for estimation of at-rest earth pressure for overconsolidated sand.

$$K_{0(OCR)} = K_{0(NC)} OCR - \left[ \frac{\mu}{1 - \mu} \right] (OCR - 1) \quad (2.11)$$

Where:  $K_{0(NC)} = 1 - \sin \phi_{cr}'$ ;  $\mu$  = Poisson's ratio ( $\mu = 0.1 - 0.3$  for loose sands;

$\mu = 0.3 - 0.4$  for dense sands)

Massarsch (1975) presented the new method for measuring in situ lateral earth pressure in soft clay. The method developed at the Royal Institute of Technology in Stockholm by using Glootzl measuring system. A spade-like, very thin (4mm) cell with large surface area (10x20cm) was installed at greater depths thanks to the protection of a steel casing. In this method, the at-rest

earth pressure coefficient  $K_0$  was estimated from the stress and pore pressure measurements. They also reported that the at-rest earth pressure coefficient  $K_0$  at 5 m depth was from 0.58 to 0.62.

Meyerhof (1976) proposed the following equation for estimating at-rest earth pressure coefficient ( $K_0$ ):  $K_{0(OC)} = (1 - \sin \phi')\sqrt{OCR}$  (2.12)

Massarsch (1979) introduced the empirical formula for predicting coefficient of at-rest earth pressure of normally consolidated clays as:  $K_0 = 0.44 + 0.42 \left[ \frac{PI(\%)}{100} \right]$  (2.13)

For overconsolidated clays, the coefficient of at-rest earth pressure can be calculated by the following equation:

$$K_{0(OC)} = K_{0(NC)}\sqrt{OCR} \quad (2.14)$$

Mayne and Kulhawy (1982) proposed the formula for estimation of at rest earth pressure coefficient of overconsolidated coarse-grained soils as follows:

$$K_{0(OC)} = (1 - \sin \phi')OCR^{\sin \phi'} \quad (2.15)$$

Sherif et al. (1984) suggested for backfill dense sand, the following relationship should be employed:

$$K_0 = (1 - \sin \phi') + \left[ \frac{\gamma_d}{\gamma_{d(\min)}} - 1 \right] 5.5 \quad (2.16)$$

Where:  $\gamma_d$  = actual compacted dry unit weight of the backfill sand,

$\gamma_{d(\min)}$  = dry unit weight of the sand in the loosest state

Duncan et al. (1991) stated that compaction of soil in layers adjacent to retaining walls induced lateral earth pressure above its pre-compaction values (normal at rest values). They developed charts for calculating  $K_0$  by using a computer program, namely, EPCOMP2, based on the theory developed by Duncan and Seed called the hysteretic theory. These charts should be used for backfill cohesionless soil. They also expressed that for backfill clay soil, the high lateral pressures induced by compaction would be decreased gradually over time to normal at-rest values. Therefore, these charts should only be used to approximate the horizontal pressures in backfill clay soil immediately after compaction.

Mesri and Hayat (1993) performed laboratory investigation on undisturbed specimens of series of soft clay deposits. They introduced that an end-of-primary (EOP) coefficient of at-rest earth pressure,  $K_{op}$ , could be defined in terms of the constant-volume friction angle  $\phi'_{cv}$ . The values of  $K_{op}$  for clay compositions and granular soils ranged from 0.31 to 0.67, corresponding to the value of  $\phi'_{cv}$  from  $19^\circ$  to  $44^\circ$ . They suggested the following equation for predicting  $K_{op}$  of sedimented, normally consolidated clays and granular soils:  $K_{op} = 1 - \sin \phi'_{cv}$  (2.17)

In case of overconsolidated soil, they proposed the empirical formula as follows:

$$K_0 = (1 - \sin \phi'_{cv}) OCR^{\sin \phi'_{cv}} \quad (2.18)$$

Hamouche et al. (1995) reported the results of series of field tests (the self-boring pressuremeter, the Marchetti flat dilatometer, and the hydraulic fracturing) that were applied for three different clay deposits of eastern Canada. For an overconsolidated clay deposit, the values of  $K_0$  from the experimental results were greater than that from using empirical relationship in theory:

$$K_{0(OC)} = K_{0(nc)} OCR^\alpha \quad (2.19)$$

where:  $\alpha = \sin \phi'$ ,  $K_{0(nc)} = 1 - \sin \phi'$

They proposed that the equation (2.19) could be used to approximate the value of  $K_0$  of overconsolidated clays, with a value of  $\alpha$  between 0.75 and 1.15. It was necessary to determine  $K_0$  directly by in situ tests for cases where accurate values of  $K_0$  were required.

Landva et al. (2000) introduced a new apparatus, the split ring, to conduct one dimensional compression tests on municipal waste samples in order to measure at-rest earth pressure. They compared their measured values to typical values of granular materials and tire chips. They reported that the values of coefficient of earth pressure at-rest  $K_0$  determined from the split ring test ranged from 0.23 to 0.40. The long term at-rest earth pressure coefficient of waste materials would be 0.47-0.49. They found that the at-rest earth pressure coefficient of waste materials decreased with an increasing amount of fibres (exposed belts). Therefore, the critical condition for coefficient of at-rest earth pressure of waste materials was the long term state when decomposable fibres was no longer available.

Michalowski (2005) revisited the Jaky's coefficient of at-rest earth pressure  $K_0$ , focusing on the problem of stress distribution in a wedge-shaped prism of sand. The author proposed the empirical relationship for the coefficient of at-rest earth pressure  $K_0$  as follows:

$$K_0 = (1 - \sin \phi') \frac{1 + \frac{2}{3} \sin \phi'}{1 + \sin \phi'} \quad (2.20)$$

The author also stated that the coefficient of at-rest earth pressure for overconsolidated soil  $K_{0(OC)}$  was typically represented as a function of overconsolidation ratio OCR, but it was not dependent

on the magnitude of the maximum consolidation stress. The  $K_{0(OC)}$  not only was affected by OCR but also was affected by some dependence on history including geologic features such as compaction bands.

Teerachaikkulpanich et al. (2007) introduced a method for estimation of at-rest earth pressure coefficient  $K_0$  of kaolin clay by using the modified Oedometer covered with a pressurized chamber known as the COWK triaxial apparatus. Based on their study, the  $K_0$  of normally consolidated clays was determined by using the equation below:

$$K_0 = \frac{CP - u_1}{\sigma'_{v0}} \quad (2.21)$$

Where: CP = cell pressure,  $u_1$  = pore water pressure measured by pressure transducers,  $\sigma'_{v0}$  = consolidation pressure during the preceding consolidation process.

They reported that the values of  $K_0$  of normally consolidated kaolin used ranged from 0.68 to 0.75. Most of the values of  $K_0$  of both normally and overconsolidated clays using empirical equations proposed by many researchers in the literature were lower than that from the experimental study.

Kalman (2008) attempted to determine  $K_0$  of overconsolidated clay (Kiscelli clay) by two in situ measurements, namely, the Stresses Monitoring Station (borehole cells) and the flat jack. The results from these field tests supported their assumption which the values of  $K_0$  of overconsolidated clay were more than 1. They demonstrated that the empirical relationship for estimation of  $K_0$  proposed by Jaky always gave the result of  $K_0$  less than 1. Jaky's formula should be only applied for geologically young and normally consolidated soils. Typically, for soft and plastic soils, the values of  $K_0$  could be equal to 1. For overconsolidated soil/rock, the values of  $K_0$  were greater than 1.

Hanna and Al-Romhein (2008) conducted an experimental investigation of the at-rest earth pressure of overconsolidated cohesionless soil acting on a vertical rough retaining wall with horizontal backfill. They proposed an empirical formula for determination of the at-rest earth pressure coefficient of overconsolidated cohesionless soil as follows:

$$K_{0(OC)} = (1 - \sin \phi') OCR^{(\sin \phi' - 0.18)} \quad (2.22)$$

Where: OCR = the overconsolidation ratio, which is computed as  $OCR = \frac{\sigma_v}{\gamma h}$  ( $\sigma_v$  = vertical pressure measured by the respective transducer at a given location,  $\gamma h$  = theoretical overburden pressure calculated at that location).

They stated that the OCR was a function of the degree of the soil particles interlocking, stress history, and sand placing techniques employed in the laboratories. An increase of the OCR would result in significant increase of  $K_{0(OC)}$ . They also found that the friction angle between the wall and the sand ( $\delta$ ) had no effect on the measured coefficient of at-rest earth pressure.

Cai et al. (2011) attempted to assess the at-rest earth pressure coefficient  $K_0$  based on the measurement of shear wave velocities from the seismic piezocone test (SCPTU). The empirical relationship in terms of  $K_0$  was proposed as follows:

$$\frac{V_s(HV)}{V_s(HH)} = \frac{C_s(HV)}{C_s(HH)} \left( \frac{1 + K_0}{2K_0} \right)^{n'} \quad (2.23)$$

Where:  $V_s(HV)$  = the shear waves propagate in vertical direction,  $V_s(HH)$  = the shear wave propagate in horizontal direction;  $C_s(HV)$  and  $C_s(HH)$  = dimensional material constants, which could reflect the fabric anisotropy of the soil structure.



For granular soils, the ratio  $\frac{V_s(HV)}{V_s(HH)} = 0.93$ , and  $n_t = 0.25$ , while  $\frac{C_s(HV)}{C_s(HH)} = 0.85$  for cohesive soils.

The author concluded that the shear wave velocity ratio  $\frac{V_s(HV)}{V_s(HH)}$  was slightly influenced only by changes in the in-situ effective stress conditions.

El-Emam (2011) attempted to study at-rest lateral earth pressure of overconsolidated sand experimentally and numerically. The author reported that  $K_0$  increased significantly by increasing overconsolidation ratio with repeated vibration compaction. Jaky's formula led to underestimate  $K_0$  for overconsolidated sand. The author also found that the resultant of at-rest lateral earth pressure  $P_0$  was located closer to  $0.4H$  ( $H$  was the backfill height) from the wall base, which was above the  $0.3H$  assumed by the classical earth pressure theory.

### 2.3.3. Passive earth pressure

#### Coulomb's theory

Coulomb theory (Das, 2009) for determination of lateral earth pressure based on limit equilibrium method, considering the wall soil friction angle  $\delta$ . It is assumed that the soil is isotropic and homogeneous. The soil mass will slip along a plane inclined an angle  $\alpha$  to the horizontal line. The slip plane can be determined by looking through the plane on which the maximum thrust acts. Coulomb's passive earth pressure coefficient ( $K_p$ ):

$$K_p = \frac{\cos^2(\phi' + \theta)}{\cos^2 \theta \cos(\delta - \theta) \left[ 1 - \sqrt{\frac{\sin(\phi' + \delta) \sin(\phi' + \beta)}{\cos(\delta - \theta) \cos(\beta - \theta)}} \right]^2} \quad (2.24)$$

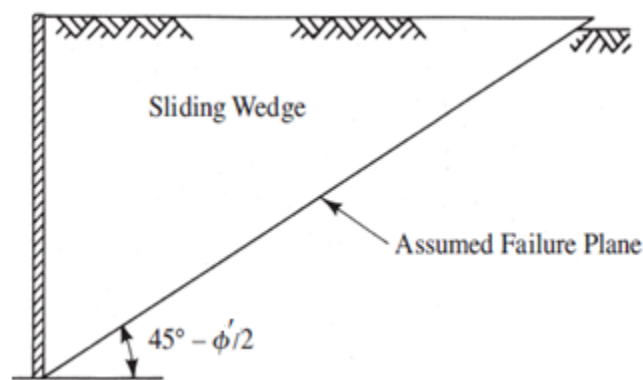
In which:  $\phi'$  = the angle of shearing resistance of the backfill soil,  $\theta$  = the angle of back side of wall and a horizontal line,  $\beta$  = the inclined backfill angle, and  $\delta$  = the wall friction angle.

The passive thrust  $P_p$  is commonly assumed to act approximately a distance of  $(H/3)$  above the wall base. It is of interest to note that for a frictionless ( $\delta=0$ ) and vertical ( $\theta=0$ ) wall with a horizontal backfill ( $\beta=0$ ), the above equation of  $K_p$  reduces to the Rankine's equation.

### Rankine's theory

Rankine's theory (Liu and Evett, 2008; Das, 2009) for determination of lateral earth pressure is based on the stress state of soil and several assumptions. These assumptions are as follows:

- The wall is frictionless (no friction between wall and soil) and has to be vertical.
- The assumed failure plane is a function of the angle of shearing resistance ( $\phi'$ ), and the sliding wedge is along the assumed failure plane (Figure 2.13).
- The lateral earth pressure is various linearly with depth.
- The resultant force acts approximately a distance of  $(H/3)$  above the wall base, and it is parallel to the backfill surface.



**Fig. 2.13.** Assumed failure plane for Rankine theory: Passive state (Liu and Evett, 2008)

Rankine's passive earth pressure coefficient:  $K_p = \frac{1 + \sin \phi'}{1 - \sin \phi'} = \tan^2 \left( 45 + \frac{\phi'}{2} \right)$  (2.25)

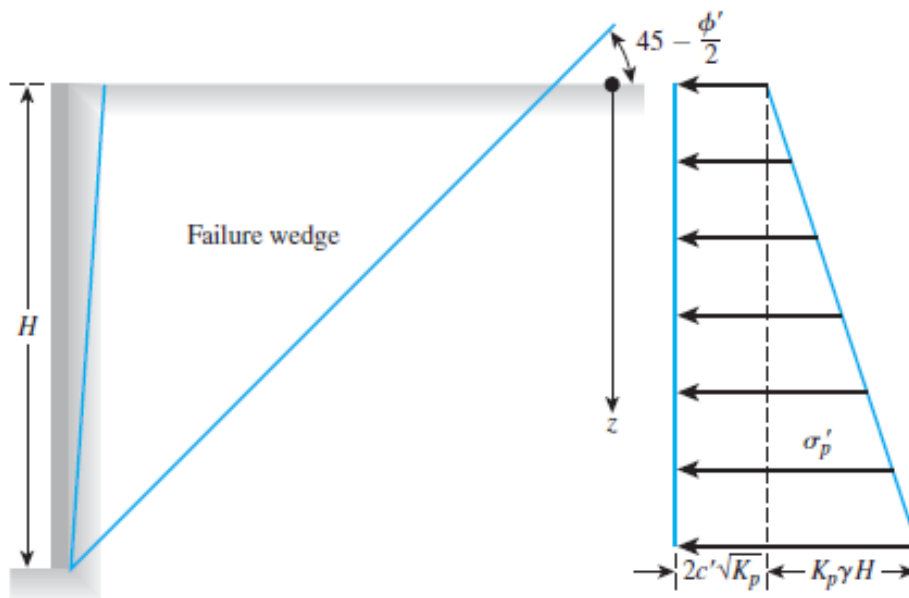
The total force per unit length of the wall:  $P_p = \frac{1}{2} \gamma K_p H^2$  (2.26)

Figure 2.14 shows a frictionless wall retaining cohesive soils. The passive earth pressure acting on the wall at any depth below the ground surface can be expressed as follows:

$$\sigma_p = K_p \sigma_v + 2c' \sqrt{K_p}$$
 (2.27)

The total passive force per unit length of the wall can be found from the area of the total pressure diagram as below:

$$P_p = \frac{1}{2} \gamma K_p H^2 + 2c' H \sqrt{K_p}$$
 (2.28)



**Fig. 2.14.** Rankine's passive earth pressure distribution against a retaining wall with backfill cohesive soils (Das, 2009)

Narain et al. (1969) conducted series of laboratory tests with both the loose and dense sands to study passive earth pressure in sands. The test series involved translation of the wall, rotation of the wall about its bottom, and rotation of the wall about its top. The passive earth pressure coefficient  $K_p$  increased directly with the wall movements till a maximum value was reached. The magnitude of passive pressures and displacement required to cause maximum pressures were maximum in the case of rotation of the wall about its bottom and minimum in the case of rotation of the wall about its top. The distribution of passive pressures along the height of the wall was triangular only in the case of the wall that was translated; whereas the distribution of the passive pressures was parabolic in the case of the wall that was rotated. The location of the resultant force was above one third from bottom in the case of rotation of the wall about its bottom, while it was below one third from bottom in the case of rotation of the wall about its top.

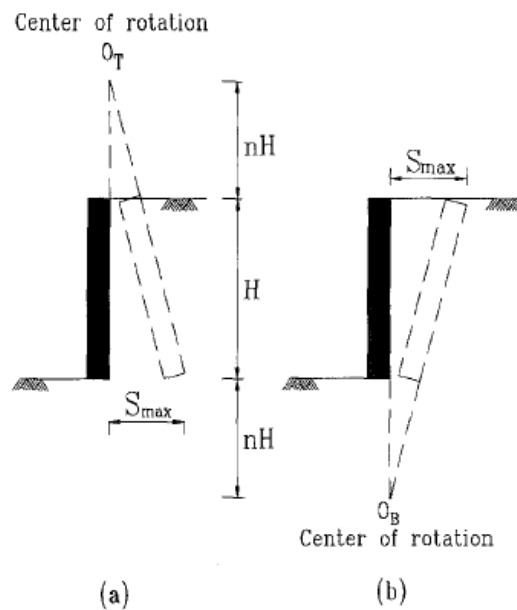
Khoury (1994) investigated a laboratory experiment on the passive earth pressure of overconsolidated homogeneous and layered cohesionless soil acting on a retaining wall. The wall movement used in the tests was horizontal, no any rotation of the wall was allowed. The author proposed the following formula:

$$K_{pm} = b\sqrt{OCR}K_{pr} \quad (2.29)$$

Where:  $K_{pm}$  = coefficient of passive earth pressure of overconsolidated sand;  $K_{pr}$  = coefficient of passive earth pressure by Rankine's method for normally consolidated sand;  $b$  = constant ( $b = 1$  when  $OCR = 1.0$ ;  $b = 2.5$  when  $OCR > 1.0$ ).

Fang et al. (1994) performed laboratory experiments of passive earth pressure acting on vertical rigid wall with horizontal dry sand backfill. The entire retaining wall facility used in this

investigation comprised four components including model retaining wall, soil bin, driving system, and data acquisition system. The test series involved translational wall movement, rotation about a point above the top (RTT), and rotation about a point below the wall base (RBT) (Figure 2.15).



**Fig. 2.15.** Passive wall movement: (a) RTT mode; (b) RBT mode (Fang et al., 1994)

For the translational wall movement, the pressure distribution was fundamentally hydrostatic at every single stage of wall movement. At different depths of soil mass, the passive state was approximately simultaneously reached.

For the RTT wall movement ( $n = 0.00$ ), the pressure distribution obtained was not close to linear. The resultant force  $P_p$  was evidently lower than that estimated using Coulomb and Rankine theories and its location was about  $0.18H$  above the base of the wall.

For the RBT wall movement ( $n = 0.00$ ), the pressure distribution obtained was nonlinear. The total force increased with an increase of wall movement continually, and the location of  $P_p$  was at about  $0.55H$  above the base of the wall.

The authors also reported that the magnitude and location of the passive thrust  $P_p$  were importantly influenced by the wall movement mode when  $n$  value was small; whereas, the passive thrust  $P_p$  was slightly affected by the wall movement mode when  $n$  value was greater than about 2.0.

Fang et al. (1997) investigated experimentally passive earth pressures acting on a vertical rigid wall with inclined backfill dry sand. They reported that  $K_p$  increased with increasing backfill inclination. The experimental earth pressure distributions were approximately linear at every single stage of the wall motion up to failure. The experimental results presented a fairly good agreement with that determined by Coulomb's theory. However, Rankine's theory showed a tendency of underestimation of the passive thrust. They also found that  $K_p$  increased with wall motion before reaching the passive condition.

Kumar and Rao (1997) attempted to determine the coefficient of passive earth pressure of sand by method of slices. A design chart which showed the variation of  $K_p$  with  $\delta/\phi'$  was presented for determining the coefficient of passive earth pressure  $K_p$  for practical uses.

Mazindrani and Ganjali (1997) studied the earth pressure of inclined cohesive backfill. The empirical formula for estimating passive earth pressure coefficient  $K_p$  was proposed as follows:

$$K_p = \frac{1}{\cos^2 \phi'} \left\{ 2 \cos^2 \beta + 2 \left( \frac{c'}{\gamma z} \right) \cos \phi' \sin \phi' + \sqrt{4 \cos^2 \beta (\cos^2 \beta - \cos^2 \phi') + 4 \left( \frac{c'}{\gamma z} \right)^2 \cos^2 \phi' + 8 \left( \frac{c'}{\gamma z} \right) \cos^2 \beta \sin \phi' \cos \phi'} \right\} - 1$$

(2-30)

Where:  $\phi'$  = angle of internal friction of soil,  $\gamma$  = unit weight of soil,  $z$  = depth to any point on the vertical back of the retaining wall from the level ground surface,  $c'$  = soil cohesion,  $\beta$  = inclined backfill angle.

They indicated that  $K_p$  increased with an increase of value of inclined backfill angle ( $\beta$ ).  $K_p$  decreased with increasing value of soil cohesion ( $c'$ ).

Zhu and Qian (2000) proposed a new method for estimation of coefficient of passive earth pressure  $K_p$  using triangular slices based on the limit equilibrium method. The method was just applied for the case of cohesionless backfill without surcharge. They also provided tables and charts for determining  $K_p$  for practical application.

Duncan and Robert (2001) attempted to determine passive earth pressure using Log Spiral earth pressure theory along with special computer program corrected for 3D effects. They found that passive earth pressure for resisting structures movement was controlled by four main factors. These factors consisted of the amount and direction of the wall movement, strength and stiffness of the soil, friction and adhesion on the interface between the soil and structure, and the structure shape.

Fang et al. (2002) conducted experimentally investigation of passive earth pressure acting on a vertical rigid wall with horizontal backfill dry sand. The test series involved wall with backfill loose sand, wall with backfill medium dense sand, and wall with backfill dense sand. The research only focused on the translational wall movement to study the effect of soil density on the passive earth pressure.

For the wall with backfill loose sand, the passive earth pressure increased with increasing wall movement and lastly a limit passive pressure was reached. The passive earth pressure distributions was essentially linear at every single stage of wall movement. The passive thrust was slightly underestimated using Coulomb's theory.

For the wall with backfill dense sand, the passive earth pressure increased with increasing wall movement. After reaching a peak value,  $K_p$  decreased with increasing wall movement until reaching an ultimate value. In this case, the passive earth pressure would be overestimated using Coulomb's theory with the angle of shearing resistance at peak ( $\phi'_{peak}$ ). The authors recommended that the dilation and strength reduction of the backfill dense sand should be considered in estimation of the passive earth pressure.

Hanna and Khoury (2005) investigated a laboratory experiment on the passive earth pressure of overconsolidated cohesionless soil acting on a retaining wall. Tests were conducted on walls retaining homogeneous overconsolidated sand, strong overconsolidated cohesionless backfill overlying weak deposit, and weak overconsolidated cohesionless backfill overlying strong deposit.

For smooth walls ( $\delta=0$ ) retaining homogeneous overconsolidated sand:

$$K_{p(OC)} = b\sqrt{OCR}K_{p(NC)} \quad (2.31)$$

Where:  $OCR = \frac{\sigma_v}{\gamma h}$  ( $\sigma_v$  = vertical pressure measured by the respective transducer at a

given location,  $\gamma h$  = overburden pressure calculated at that location);  $K_{p(NC)}$  = coefficient of passive earth pressure of normally consolidated sand (Rankine's value), and  $b$  = constant.

They found the constant  $b$  depended on the OCR: If the OCR was equal to 1, the constant  $b$  was equal to 1; whereas, if the OCR was greater than 1, the constant  $b$  was continuously



greater than 2. Generally, the constant  $b$  increased moderately with increasing the OCR.

For rough wall ( $1/2 \phi' < \delta < 2/3 \phi'$ ) retaining homogeneous overconsolidated sand:

$$K_{p(OCR)} = K_{p(NC)} \left[ 1.5 - \left( \frac{\delta - 25}{100} \right) \right] OCR^{\sin \delta} \quad (2.32)$$

The authors stated that compaction of cohesionless soil induced additional stresses in the soil mass. These stresses were locked in, causing the backfill soil becoming overconsolidated. Increasing the OCR led to the increase of passive earth pressure acting on the retaining wall. Moreover, they reported that for the case of a strong overconsolidated cohesionless backfill overlying a weak deposit, the passive earth pressures acting against the wall reduced significantly due to extending the failure mechanism to the weak deposit:

$$K_p \left( \frac{\text{backfill}}{\text{deposit}} \right) = \left[ \frac{K_p(\text{backfill}) + K_p(\text{deposit})}{2} \right] \quad (2.33)$$

Where:  $K_p(\text{backfill})$  = the coefficient of passive earth pressure of homogeneous backfill material, and  $K_p(\text{deposit})$  = the coefficient of passive earth pressure of homogeneous lower deposits.

For the case of a weak overconsolidated cohesionless backfill overlying a strong deposit, the passive earth pressure acting against the wall could be estimated based on the condition of the backfill soil because the failure mechanism would remain confined in the backfill layer.

Vrecl – Kojc and Skrabl (2007) proposed a modified 3D failure mechanism for determination of the 3D passive earth pressure coefficient. The authors presented that the total passive earth force was defined by:

$$P = K_{p\gamma} \gamma \frac{h^2}{2} b + K_{pq} qhb + K_{pc} chb \quad (2.34)$$

Where:  $K_{p\gamma}$ ,  $K_{pq}$ ,  $K_{pc}$  = earth pressure coefficient due to soil weight, vertical surcharge loading, and cohesion;  $q$  = surcharge loading;  $H$  = the height of the rigid retaining wall;  $c$  = cohesion;  $\gamma$  = unit weight

Hanna et al. (2011) developed numerical model using finite element method and the Mohr-Coulomb constitutive law to study the passive earth pressure on embedded vertical plate anchors in sand. They attempted to evaluate the stress and strain conditions during installation of anchor plates, and correspondingly evaluate the level of overconsolidation in the surrounding soils. They found that the passive earth pressure acting on anchor plates increased with increasing angle of shearing resistance  $\phi'$  and the overconsolidation ratio of sand; whereas, it decreased with increasing the embedment depth of anchor. Eventually, design theories and design procedures were presented.

Poterasu (2013) attempted to study the influence of soil collapse on the passive earth pressure developed behind the retaining wall. The wall movement was translational. The author introduced that the passive earth pressure coefficient ( $K_p$ ) decreased with increasing collapse potential  $C_p$ . When the backfill collapsible soil was saturated,  $K_p$  decreased with approximately 90% from the values estimated at initial condition.

Weng et al. (2014) studied the development of the passive earth pressure using PFC2D software based on the distinct element method (DEM). They presented that  $K_p$  increased with increasing the angle of shearing resistance ( $\phi'$ ).  $K_p$  also increased with increasing inter-particle stiffness. However,  $K_p$  would decreased with increasing the particle size.

Awn and Zakaria (2014) attempted to study the behavior of gravity retaining wall resting on collapsible soil by a small prototype model. The collapsible soil used in this study was gypsums soil with different gypsum percentages (5%, 20%, 30%, and 50%). The gypsums soil used to simulate the base soil was the fundamental factor controlling the behavior of the retaining wall. The gravity retaining wall model was backfilled with sandy soil. The authors then mixed 2.7% of cement with gypsums soil together as the soil improvement to reduce the vertical and tilting strain of the retaining wall. The authors reported that the collapsible soil layer (gypsums soil layer) experienced collapse when getting water percolation. They also revealed that with their test setup, the movement of the retaining wall was not uniform. The wall might fill toward or backward, settled in the toe faster than heel or the opposite when the collapsible becoming wet. However, after treating the embedded sypseous soil layer (the foundation of the retaining wall), the improvement in rotation settlement and collapse for the retaining wall model reached about 90% in compared to the values before the treatment.

Cai et al. (2016) proposed an analytical method to estimate the passive earth pressure acting on walls retaining cohesive soils subjected to the translation mode. The study was focused on the effects of several factors on passive earth pressure such as internal friction angle of backfill, wall – soil interface friction angle, surcharge pressure, cohesion, unit weight, and inclination angle of the slip surface on the passive earth pressure as well as on the slip-surface angle. The authors presented that the passive earth pressure increased when increasing the soil cohesion, the internal friction angle, and the uniformly distributed load on the surface of the cohesive backfill, respectively. The coefficient of passive earth pressure increased when increasing the internal

friction angle of the backfill. The authors proposed the following formula to estimate the passive earth pressure force on the rigid retaining wall:

$$P = K \left( qH + \frac{\gamma H^2}{2} \right) + (K - 1) \frac{c' H}{\tan \phi'} \quad (2.35)$$

where: K= coefficient of the passive earth pressure on the rigid wall

q = surcharge loading, H = the height of the rigid retaining wall, c' = cohesion of the backfill,  $\phi'$  = angle of shearing resistance,  $\gamma$  = unit weight.

Hanna and Diab (2016) presented a numerical model for a wall retaining cohesionless backfill subjected to passive earth pressure under translation mode. The model applied the finite element technique, the constitutive law of the modified Cam-clay model, and the critical state soil mechanics (CSSM) concept. The model was also capable of incorporating the influence of the soil deformation and the stress history of the sand backfill on the values of passive earth pressure. Finally, the authors proposed the empirical formulae for estimation of the coefficients of passive earth pressure for normally consolidated sand and overconsolidated sand with the given values of the critical state parameters.

## **2.4. Discussion**

Based on the literature review highlighted in this chapter, it can be drawn some main points as follows:

1. Collapsible soils have attracted a number of investigations from researchers/scientists/engineers, but still need more research on them due to their complexity.
2. Numerous research works can be found dealing with at-rest and passive earth pressures theories for normally consolidated or over consolidated cohesionless soils. Few studies were made on cohesive soils. No investigations or attempts were made on the earth pressures of overconsolidated collapsible soils, accordingly no design theories for such type of soils were found in the literature.
3. The theories available in the literature are not applicable for prediction of the earth pressure acting the wall retaining collapsible soils subjected to inundation. The reason is that when inundated, the collapsible soils undergo sudden strength and volume reduction, which further influence the earth pressure of this soil on the wall.

## **CHAPTER 3**

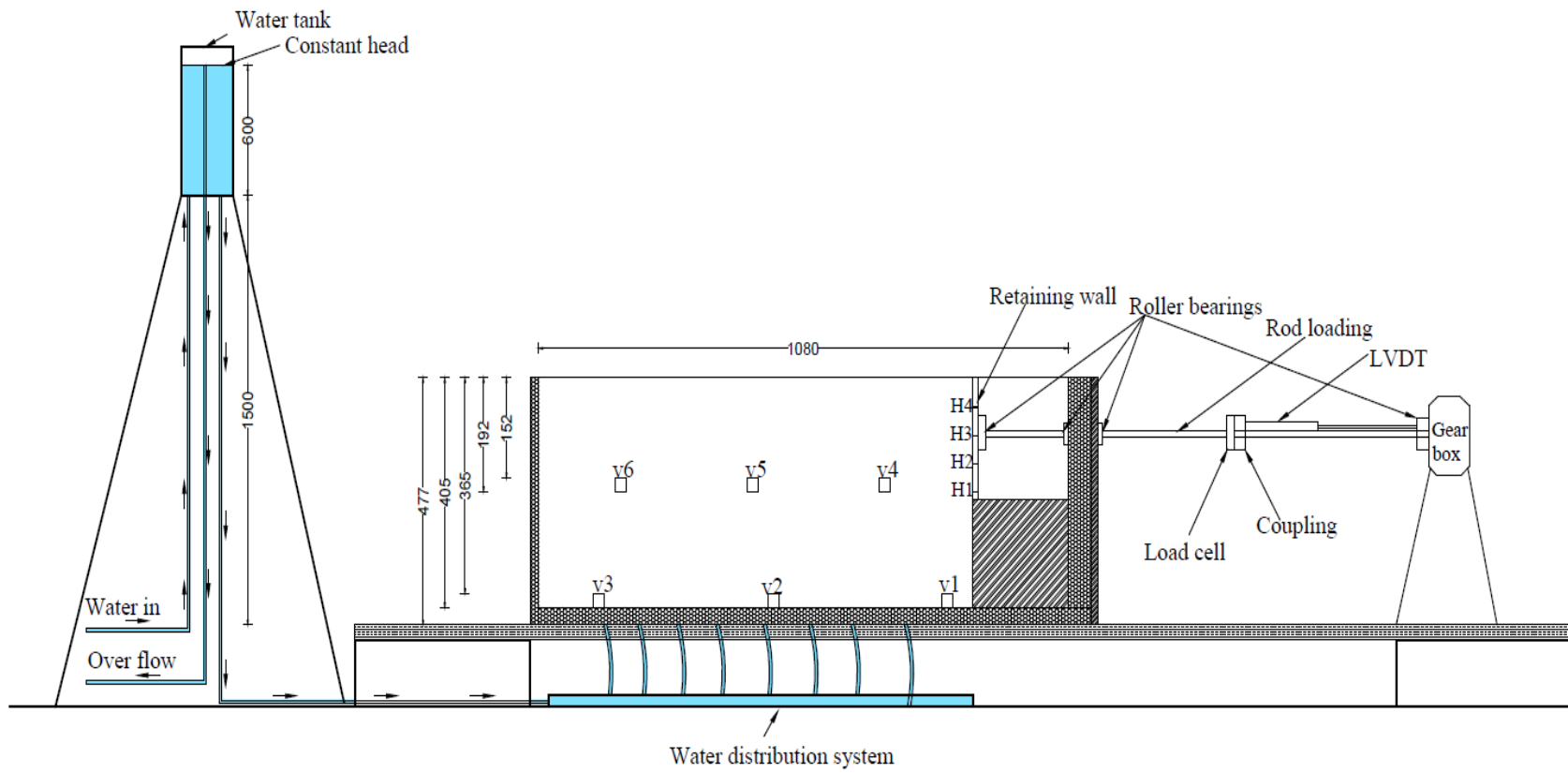
### **EXPERIMENTAL INVESTIGATION**

#### **3.1. General**

The objective of the investigation was to establish the effect of soil collapse potential  $C_p$  and the overconsolidation ratio OCR on the at rest-earth and passive earth pressure of overconsolidated collapsible soil subjected to full inundation due to the rise of groundwater table. An experimental setup was designed and built in the laboratory to simulate the condition of retaining wall with backfill collapsible soil subjected to at-rest and passive earth pressures. Based on the wall movement, the pressure measured on the wall would be determined as at-rest or passive earth pressure. For the case of passive earth pressure, the wall was pushed toward the backfill soil horizontally without any rotation which generated compressive force on the soil mass. For the case of at-rest earth pressure, the wall was static, no horizontal force was applied to the wall.

#### **3.2. Experimental setup**

An experimental setup was developed in the laboratory to examine the earth pressure on the wall retaining dry or saturated collapsible soils. The model consists of testing tank, retaining wall, loading system, and water distribution system. Figure 3.1 presents the layout of the experimental setup used in this investigation. The testing tank was 1080 mm in length, 195 mm in width, and 405 mm in depth. A metal plate was positioned in the upper part of the testing tank to simulate the retaining wall.



**Fig. 3.1.** Layout of the experimental setup

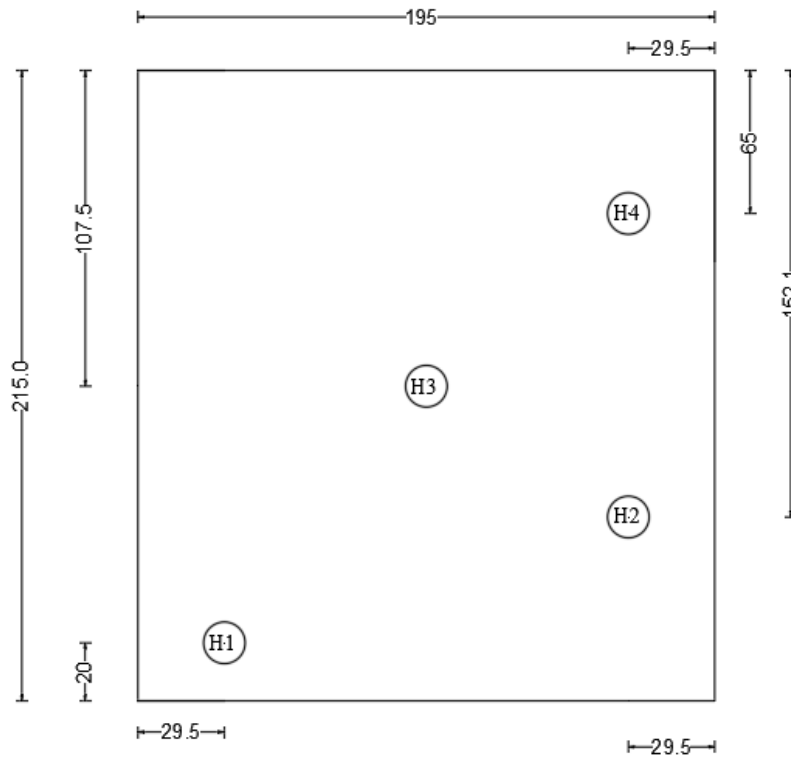
The model wall was 195 mm, 215 mm, and 19mm in width, height, and thickness, respectively. The wall was kept in the vertical position through a rod that was supported by two sets of roller. The rod was connected to a load cell to measure the total earth pressure acting on the retaining wall.

Four pressure transducers (H1, H2, H3, and H4) were installed at strategic points on the wall (Figure 3.2) to measure the earth pressure at these points and accordingly the earth pressure distribution acting on the wall. Six pressure transducers (v1 to v6) were placed in the soil mass at predetermined levels (these transducers were also placed in a staggered scheme in vertical direction to avoid boundary effects), to measure the vertical stresses in the soil mass (Figure 3.1). Three transducers were used for each selected level and they was situated at different location along the testing tank. Their readings could validate for the uniform soil mixture all over the testing tank. Each transducer was accommodated in a metal box (Figure 3.3), connected to the data acquisition system (DAS).

Furthermore, linear variable displacement transducers (LVDTs) were installed on the top of the testing tank to measure the collapse settlement of the soil behind the wall during the inundation. In order to develop the state of passive pressure in the retained collapsible soil mass, the wall was pushed horizontally toward the backfill without any rotation. A linear variable displacement transducer was installed to measure the displacement of the wall.

The pressure transducers used in this investigation were high stiffness, insensitive to temperature variation, simple to install, and water resistant. The transducers were individually calibrated in air and in sand prior to testing.



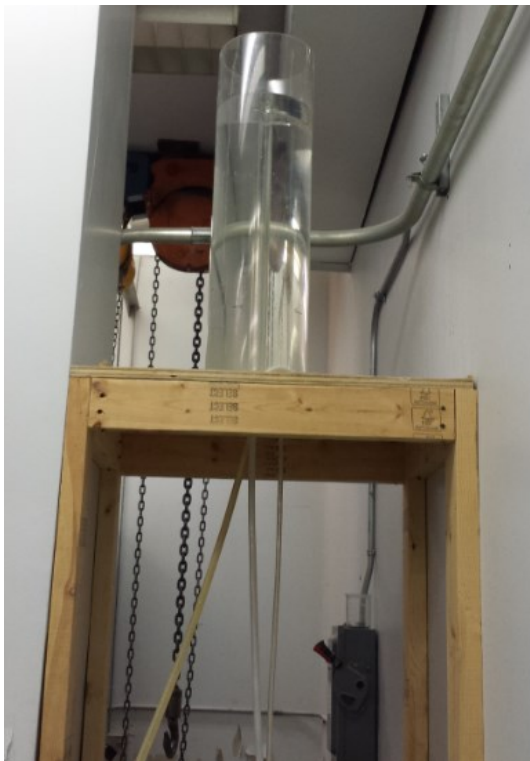


**Fig. 3.2.** Location of transducers on the model retaining wall



**Fig. 3.3.** Pressure transducers and its special box

An elevated water tank made of Plexiglas (Figure 3.4) , positioned on a stable wooden frame, connected to the testing tank through a plastic tube, through which the water was filled to the bottom of the testing tank, simulating the rise of groundwater. The water level in the water tank was remained unchanged during testing to assure laminar flow inside the soil in the testing tank. The water was filled in the water tank from the water source and one outflow pipe was fixed inside the water tank to keep the water at a constant level by draining the excess water. The second outflow pipe was connected to a water distributor (Figure 3.5) which introduced water to the soil tank through ten water inlets. To ensure even and uniform distribution of water throughout the collapsible soil during inundation, a thin layer of coarse silica sand was placed and slightly compacted at the bottom of the testing tank.



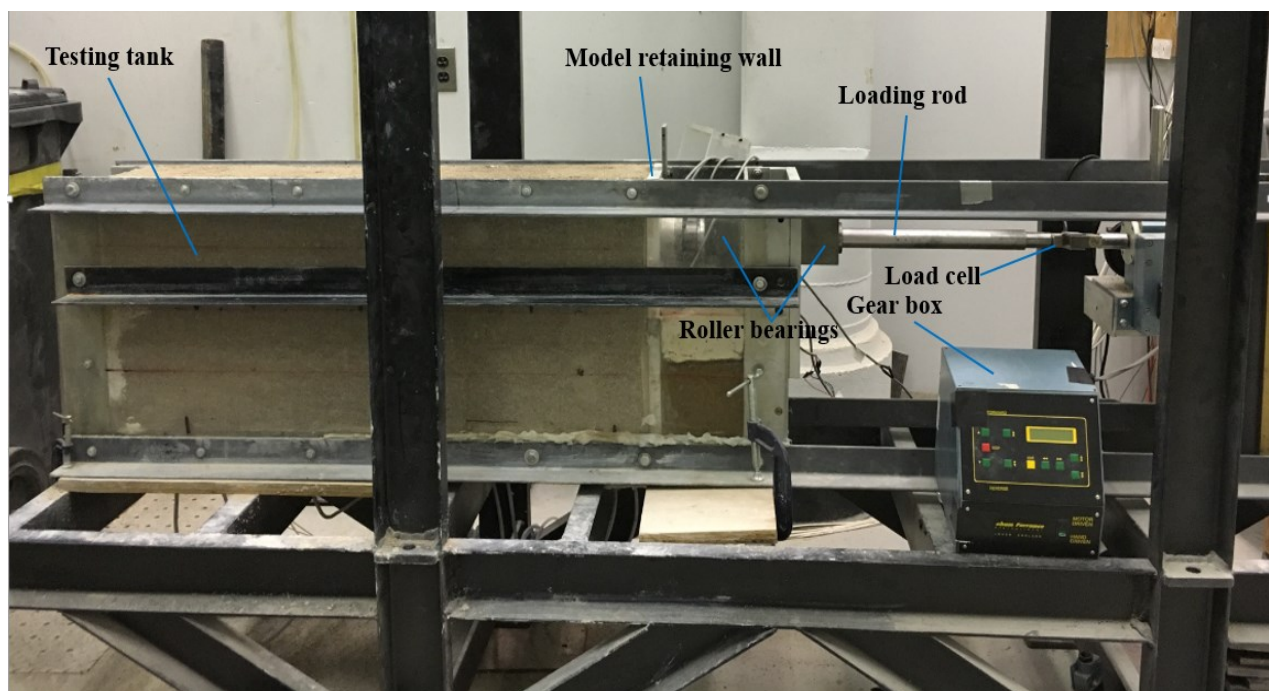
**Fig. 3.4.** Water tank



**Fig. 3.5.** Water distribution system

The loading system was composed of a gear box. The fixed gear box was used to generate horizontal forces which caused horizontal movement of the metal rod that was connected to the retaining wall. The horizontal forces generated by the gear box were measured by the load cell. One side of the load cell was connected to the horizontal retaining wall rod, another side of the load cell was connected to the horizontal rod of the gear box.

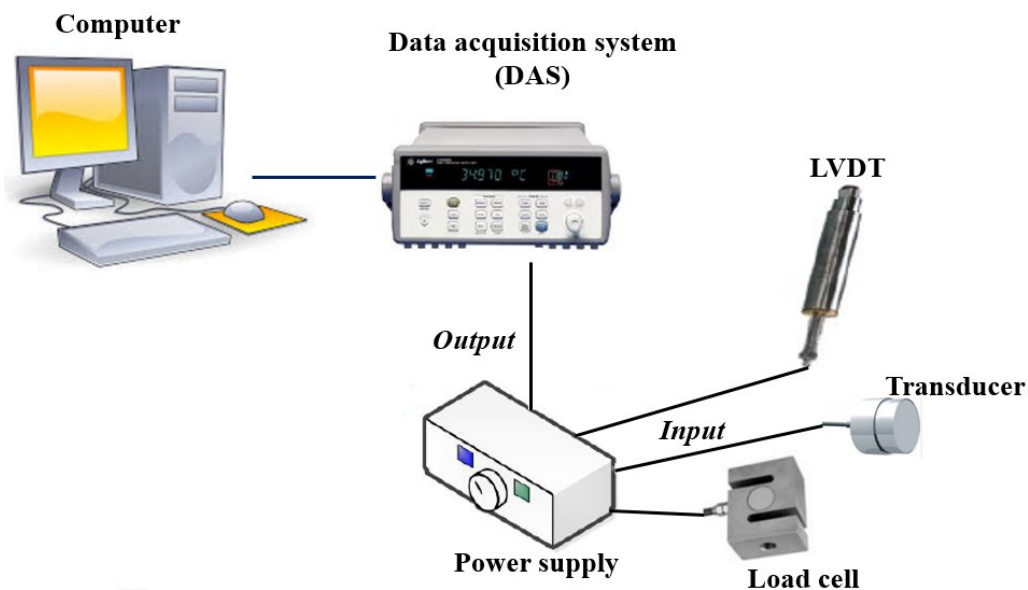
Figure 3.6 presents the photograph of main components of the experimental setup.



**Fig. 3.6.** Photograph of the experimental setup

In this experimental setup, an electric current of a constant voltage was used during the tests in order to supply the excitation voltages to the load cell, transducers, and LVDTs. Output readings from the load cell, transducers, and LVDTs were recorded on a computer in form of excel files through a Data Acquisition System (DAS) manufactured by Agilent Technologies by using

software. The software allowed data recording in the desired time intervals. The DAS allowed the measurement of millivolts from these device. The millivolt outputs from the load cell, transducers, and LVDTs were transformed to Newton (N), kilopascal (kPa), and millimeter (mm), respectively, based on the calibration of these device. Figure 3.7 illustrates schematically the connection between DAS and the other experimental instruments.



**Fig. 3.7.** Data acquisition system and measuring equipment

### 3.3. Collapsible soil preparation

The soil used in this investigation was a laboratory prepared collapsible soils formed by mixing Kaolin clay with fine sand. Generally, the higher the clay content the higher the collapse potential ( $C_p$ ). Miller et al. (1998) reported that the maximum collapse occurs at around 18% of clay content. The clay played a role of cementing agent that bonds the sand particles at low water content. In this study, three different types of Kaolin clay, known commercially as Rogers, Sapphire, and KT-

Cast, were used to produce collapsible soil samples, to be tested under different water content, and the compaction energy to produce the desired collapse potential  $C_p$ , following ASTM D5333 (2003). Chemical analysis and physical properties of the three types of Kaolin clay provided by the manufacture are given in Tables 3.1 and 3.2. After a number of trials, it was concluded that the mixtures formed by Rogers clay produced the highest collapse potential  $C_p$ . Sand mixed with 6%, 8%, 10%, and 14% of Rogers clay at 5% water content will produce the soils having collapse potentials  $C_p$  of 4.2%, 9%, 12.5%, and 18%, respectively.

Collapse potential  $C_p$  is the key parameter used to define collapsible soil. Oedometer tests were conducted on these soil mixtures to determine the collapse potential of these mixtures, following ASTM D5333 (2003). Firstly, soil specimen (within the oedometer ring) is placed in the oedometer apparatus. Secondly, a predetermined vertical stress is gradually applied to the soil specimen until reaching a maximum load of 200 kPa. Thirdly, the soil specimen is inundated while the load is still maintained on the soil specimen. The collapse potential ( $C_p$ ) is equal to the deformation of soil due to the addition of water, divided by the initial height of the specimen. In other words, the collapse potential ( $C_p$ ) is the collapse strain due to inundation of the specimen under 200 kPa pressure in oedometer apparatus, as shown in equation (2.4): 
$$C_p = \frac{\Delta h}{h_0} \cdot 100\%$$

Table 3.3 presents summary of the response to wetting Oedometer tests on these mixtures used in this investigation. Table 3.4 presents a summary of the laboratory test results performed on these mixtures. The range of the collapse potential of 4.2% to 18% are defined as moderate to severe, which is widely encountered in the field, as in Table 3.5. Figure 3.8 presents the grain-size distribution for these mixtures.

**Table 3.1.** Chemical analysis of the Kaoline clay (“Digitalfire.com reference library”)

Component elements properties	Rogers	Sapphire	KT-Cast
CaO (%)	0.28	0.2	0.09
MgO (%)	0.26	0.15	0.09
K <sub>2</sub> O (%)	0.26	0.19	0.1
Na <sub>2</sub> O (%)	0.12	0.1	0.09
TiO <sub>2</sub> (%)	1.3	1.4	1.64
Al <sub>2</sub> O <sub>3</sub> (%)	37.5	38.15	38.8
SiO <sub>2</sub> (%)	46.5	46.25	45.1
Fe <sub>2</sub> O <sub>3</sub> (%)	1.0	0.65	0.5

**Table 3.2.** Physical properties of the Kaoline clay (“Digitalfire.com reference library”)

Physical properties	Rogers	Sapphire	KT-Cast
pH	4.8	4.8	5.5
Dry Modulus of rupture (psi)	400*	400*	225
Surface area (m <sup>2</sup> /g)	16.5	16.5	12

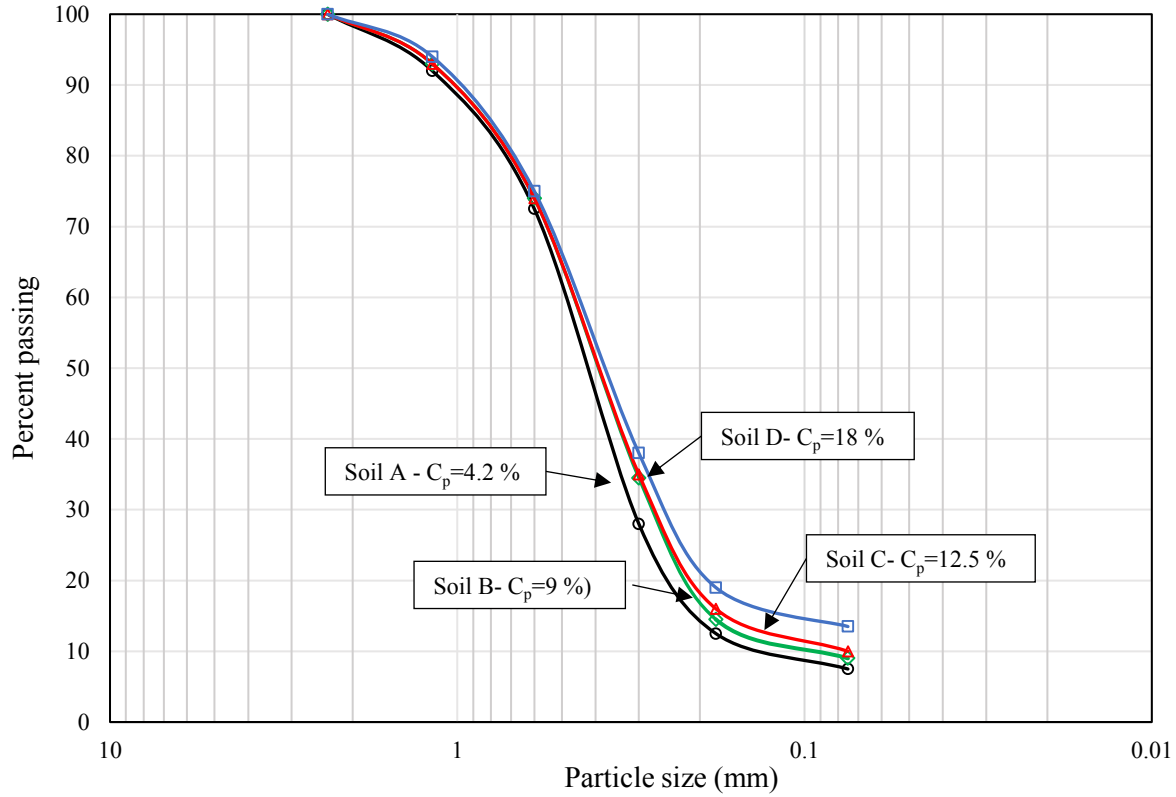
Note: \* Extruded 50% ball clay, 50%flint

**Table 3.3.** Summary of the response to wetting Oedometer tests on these mixtures used in this investigation

Mixtures	Rogers clay content (%)	Initial water content (%)	Compaction used for the soil mixtures placed the consolidation ring in Oedometer tests			C <sub>p</sub> (%)
			Compaction weight (gm)	Number of drops	Height of drops (mm)	
A	6	5	100	8	150	4.2
B	8	5	100	8	150	9.0
C	10	5	100	8	150	12.5
D	14	5	100	6	150	18

**Table 3.4.** Physical properties of collapsible soil mixtures

<b>Soil properties</b>	<b>Soil – A</b> ( $C_p = 4.2\%$ )	<b>Soil – B</b> ( $C_p = 9.0\%$ )	<b>Soil – C</b> ( $C_p = 12.5\%$ )	<b>Soil – D</b> ( $C_p = 18\%$ )
Clay content in %	6	8	10	14
Water content in %	5	5	5	5
Unit weight ( $\gamma$ ) in $\text{kN/m}^3$	16.28	16.25	16.2	15.3
Void ratio (e)	0.67	0.69	0.70	0.80
Specific gravity (Gs)	2.66	2.67	2.67	2.68
Cohesion ( $c'$ ) in kPa	9	12.5	15.5	18
Angle of shearing resistance ( $\phi'$ ) in degree	40	38.5	35	31
Liquid limit (LL)	-	-	15.9	24.7
Plastic limit (PL)	-	-	13.35	17.3
Plasticity index (PI)	-	-	2.55	7.4
Coefficient of uniformity ( $C_u$ )	4	5.4	21.9	30
Coefficient of curvature ( $C_c$ )	1.27	1.65	6.47	8.53



**Fig. 3.8.** Particle size distribution for the collapsible soil mixtures

**Table 3.5.** Mixtures classification based on collapse potential  $C_p$

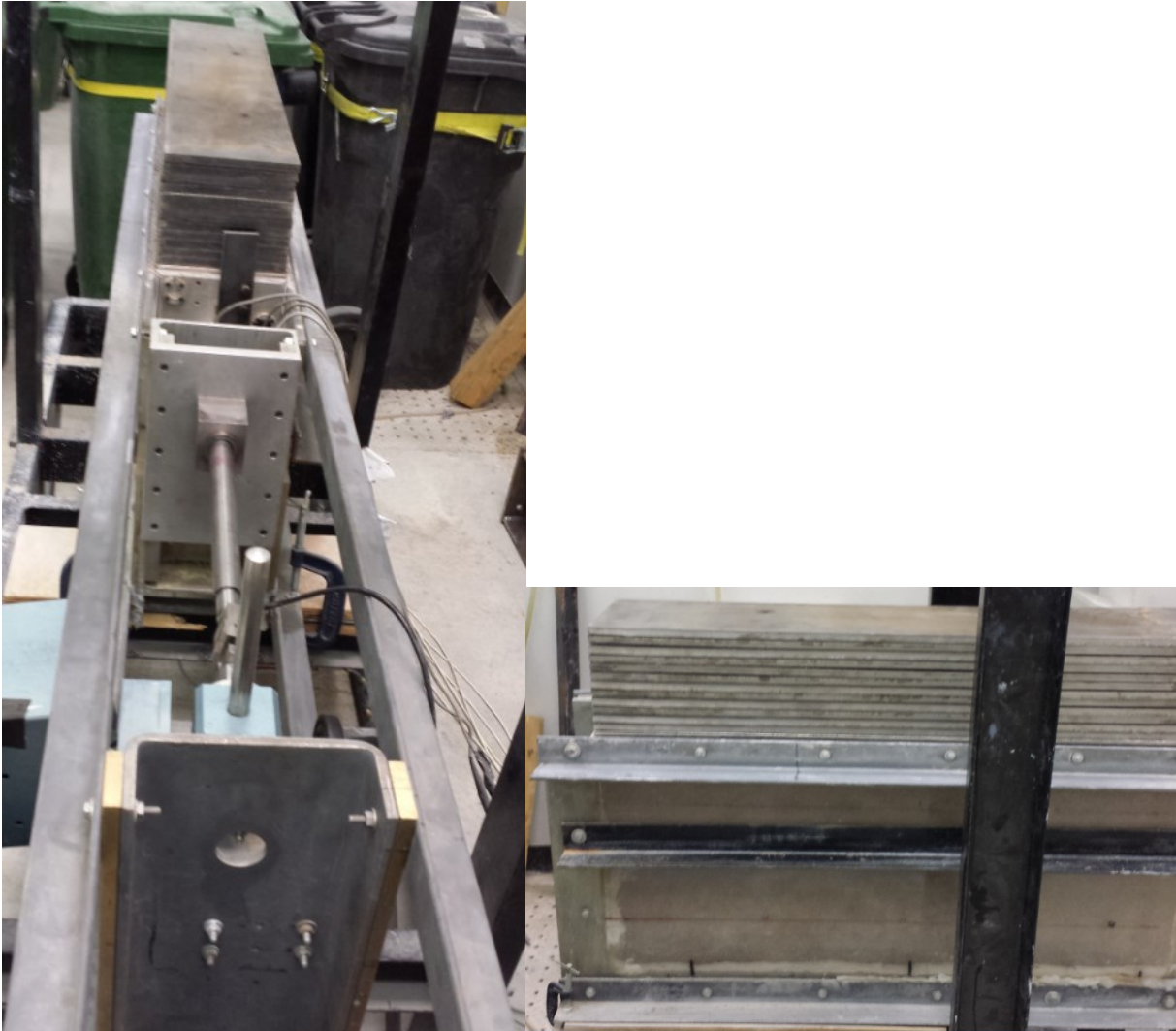
Classification	Collapsible soil mixtures			
	Soil – A ( $C_p = 4.2\%$ )	Soil – B ( $C_p = 9.0\%$ )	Soil – C ( $C_p = 12.5\%$ )	Soil – D ( $C_p = 18\%$ )
In terms of severity of foundation problem (Jennings and Knight, 1975)	Moderate trouble	Trouble	Severe trouble	Severe trouble
In terms of collapse potential (ASTM D5333, 2003)	Moderate	Moderately severe	Severe	Severe



### **3.4. Test procedure**

- a. Test was commenced by placing a thin layer of coarse silica sand at the bottom of the testing tank to ensure a uniform distribution of the water throughout the collapsible soil during inundation.
- b. The prepared collapsible soil was then placed on the top of the coarse silica sand layer in four sublayers, and compacted by means of falling weight. The compaction was made of a 4.5 kg hammer that drops freely from a height of 45.7 cm on an aluminum plate placed on the top of the soil layer. The number of drops was predetermined for each layer to obtain uniform soil mixture in the testing tank.
- c. When reaching the desired depth of the soil in the testing tank, tests, all measuring devices were connected and readings were recorded at regular interval by the DAS using a computer.
- d. For the tests performed on the saturated soils, water was introduced to the soil from the bottom of the testing tank, at a slow rate, to simulate the rise of the groundwater table. This process continued until the full inundation of the soil in the testing tank was noted.
- e. In order to produce overconsolidated soil in the testing tank, a rigid plate made of metal was placed on the top of the soil surface. A uniform static load – surcharge loading  $P_s$  (Figure 3.10) was subsequently applied on the plate, allowing loading and unloading. The process continued

until no further increase in the vertical displacement, which was measured by the LVDTs mounted on the top of the plat.



**Fig. 3.9.** Photograph of the application of the surcharge loading  $P_s$

- f. In order to generate the passive earth pressure condition, the wall was pushed toward the backfill soil horizontally without any rotation by the gear box according to the displacement

steps of approximately 0.5mm/minute. Concurrently, all measuring devices were connected and readings were recorded at regular interval by the DAS.

### 3.5. Test program

The present investigation mainly focused on examining the role of the soil collapse potential  $C_p$  and stress history (overconsolidation ratio - OCR) as well as the effect of inundation on at-rest and passive earth pressures. In this investigation, tests were performed on both dry and saturated soils. Test program for at-rest case: the main test program is listed in Table 3.6, and a series of tests for the purpose of validation is shown in Table 3.7. Test program for passive case is presented in Table 3.8. Moreover, the list of the repeated tests to ensure the repeatability of the test results is presented in Table 3.9.

**Table 3.6.** Details of testing program for at-rest earth pressure (for the purpose of developing empirical formulae)

Test no.	Soil type	Surcharge $P_s$ (kPa)	Testing condition
1	A ( $C_p = 4.2\%$ )	0	Dry soil
2	C ( $C_p = 12.5\%$ )	0	Dry soil
3	D ( $C_p = 18\%$ )	0	Dry soil
4	A ( $C_p = 4.2\%$ )	0	Saturated soil
5	C ( $C_p = 12.5\%$ )	0	Saturated soil
6	D ( $C_p = 18\%$ )	0	Saturated soil
7	A ( $C_p = 4.2\%$ )	8	Dry soil
8	C ( $C_p = 12.5\%$ )	8	Dry soil
9	D ( $C_p = 18\%$ )	8	Dry soil
10	A ( $C_p = 4.2\%$ )	8	Saturated soil
11	C ( $C_p = 12.5\%$ )	8	Saturated soil
12	D ( $C_p = 18\%$ )	8	Saturated soil

**Table 3.7.** Test program for at-rest earth pressure (for the purpose of validating the proposed empirical formulae)

Test no.	Soil type	Surcharge Ps (kPa)	Testing condition
13	A ( $C_p = 4.2\%$ )	16	Dry soil
14	C ( $C_p = 12.5\%$ )	16	Dry soil
15	D ( $C_p = 18\%$ )	16	Dry soil
16	A ( $C_p = 4.2\%$ )	16	Saturated soil
17	C ( $C_p = 12.5\%$ )	16	Saturated soil
18	D ( $C_p = 18\%$ )	16	Saturated soil
19	B ( $C_p = 9.0\%$ )	0	Dry soil
20	B ( $C_p = 9.0\%$ )	8	Dry soil
21	B ( $C_p = 9.0\%$ )	16	Dry soil
22	B ( $C_p = 9.0\%$ )	0	Saturated soil
23	B ( $C_p = 9.0\%$ )	8	Saturated soil
24	B ( $C_p = 9.0\%$ )	16	Saturated soil

**Table 3.8.** Details of testing program for passive earth pressure

Test No.	Soil type	Test series	Surcharge Ps (kPa)	Testing condition
1	A ( $C_p = 4.2\%$ )	Se1	0	Dry soil
2	B ( $C_p = 9.0\%$ )		0	Dry soil
3	C ( $C_p = 12.5\%$ )		0	Dry soil
4	D ( $C_p = 18\%$ )		0	Dry soil
5	A ( $C_p = 4.2\%$ )	Se2	8	Dry soil
6	B ( $C_p = 9.0\%$ )		8	Dry soil
7	C ( $C_p = 12.5\%$ )		8	Dry soil
8	D ( $C_p = 18\%$ )		8	Dry soil
9	A ( $C_p = 4.2\%$ )	Se3	16	Dry soil
10	B ( $C_p = 9.0\%$ )		16	Dry soil
11	C ( $C_p = 12.5\%$ )		16	Dry soil
12	D ( $C_p = 18\%$ )		16	Dry soil
13	A ( $C_p = 4.2\%$ )	Se1'	0	Saturated soil
14	B ( $C_p = 9.0\%$ )		0	Saturated soil
15	C ( $C_p = 12.5\%$ )		0	Saturated soil
16	D ( $C_p = 18\%$ )		0	Saturated soil
17	A ( $C_p = 4.2\%$ )	Se2'	8	Saturated soil
18	B ( $C_p = 9.0\%$ )		8	Saturated soil
19	C ( $C_p = 12.5\%$ )		8	Saturated soil
20	D ( $C_p = 18\%$ )		8	Saturated soil
21	A ( $C_p = 4.2\%$ )	Se3'	16	Saturated soil
22	B ( $C_p = 9.0\%$ )		16	Saturated soil
23	C ( $C_p = 12.5\%$ )		16	Saturated soil
24	D ( $C_p = 18\%$ )		16	Saturated soil

**Table 3.9.** Test program for studying the repeatability of the test results

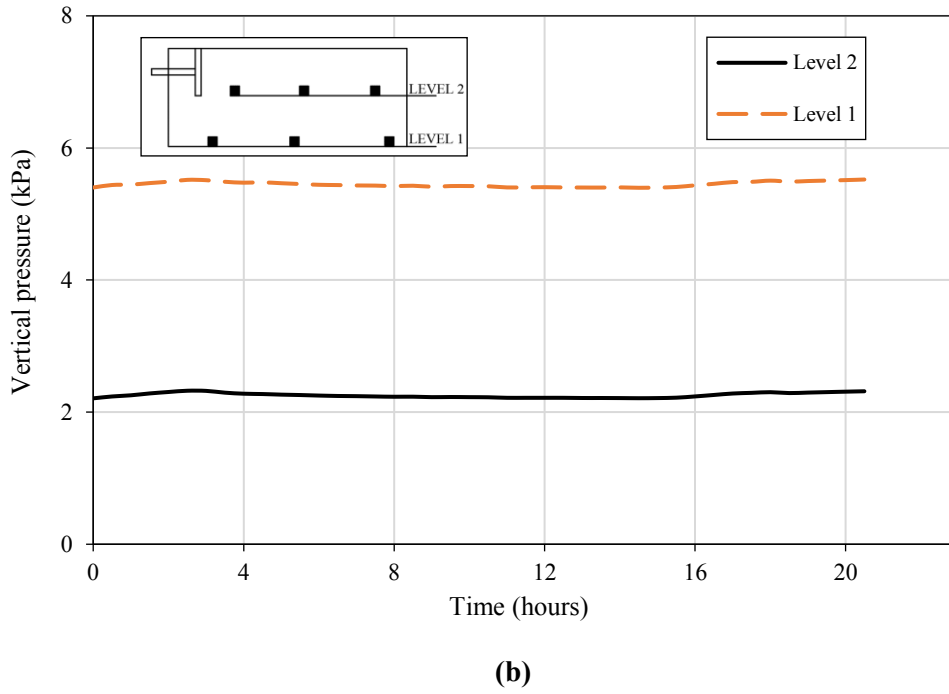
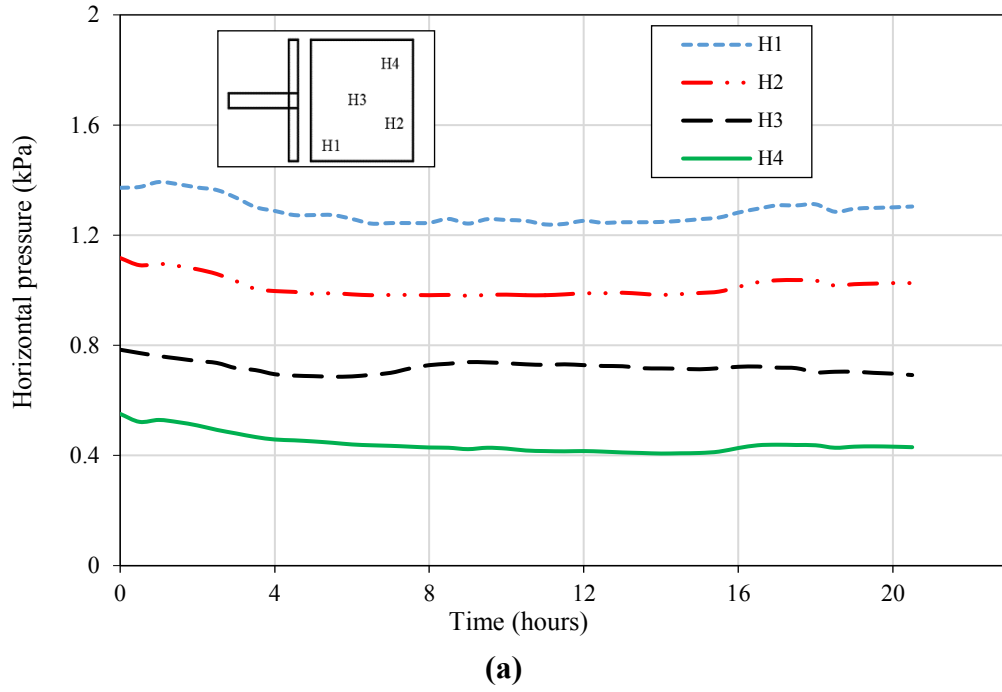
Test no.	Test type	Surcharge $P_s$ (kPa)	Soil type	Testing condition
3*	At-rest	0	D ( $C_p = 18\%$ )	Dry soil
6*				Saturated soil
4*	Passive	0	D ( $C_p = 18\%$ )	Dry soil
16*				Saturated soil

### 3.6. Validation of the experimental setup

Firstly, to consider the friction force in the bearings and the friction force between the wall and the testing tank during pushing the wall to simulate the passive earth pressure condition, Preliminary tests were conducted on the empty tank. After that, an average friction force was estimated and it was used to subtract the readings of all the passive tests.

Secondly, to ensure the suitability of the experimental setup, tests were conducted on homogenous loose sand and its coefficient of passive earth pressure ( $K_p$ ) was calculated and validated by using Rankine's theory; while the coefficient of at-rest earth pressure ( $K_0$ ) was computed and validated by using Jaky's equation.

\*At-rest earth pressure test with homogenous loose sand: the wall was static, no horizontal force was applied to the wall. The horizontal earth pressure acting on the wall was measured by the transducers installed on the wall, and vertical pressure in the sand mass measured by the transducers placed in the sand mass. The test result is presented graphically in Figure 3.10.

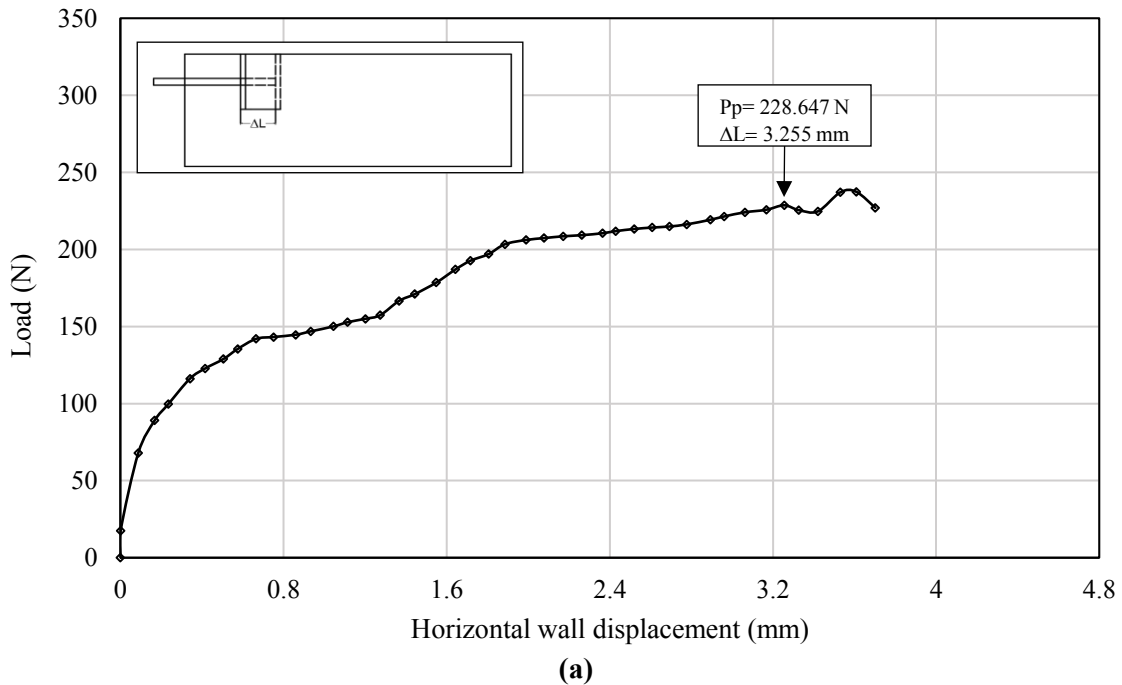


**Fig. 3.10.** Test results: (a) horizontal pressure measured by wall transducers versus time; (b) vertical pressure in the sand mass measured by transducers versus time

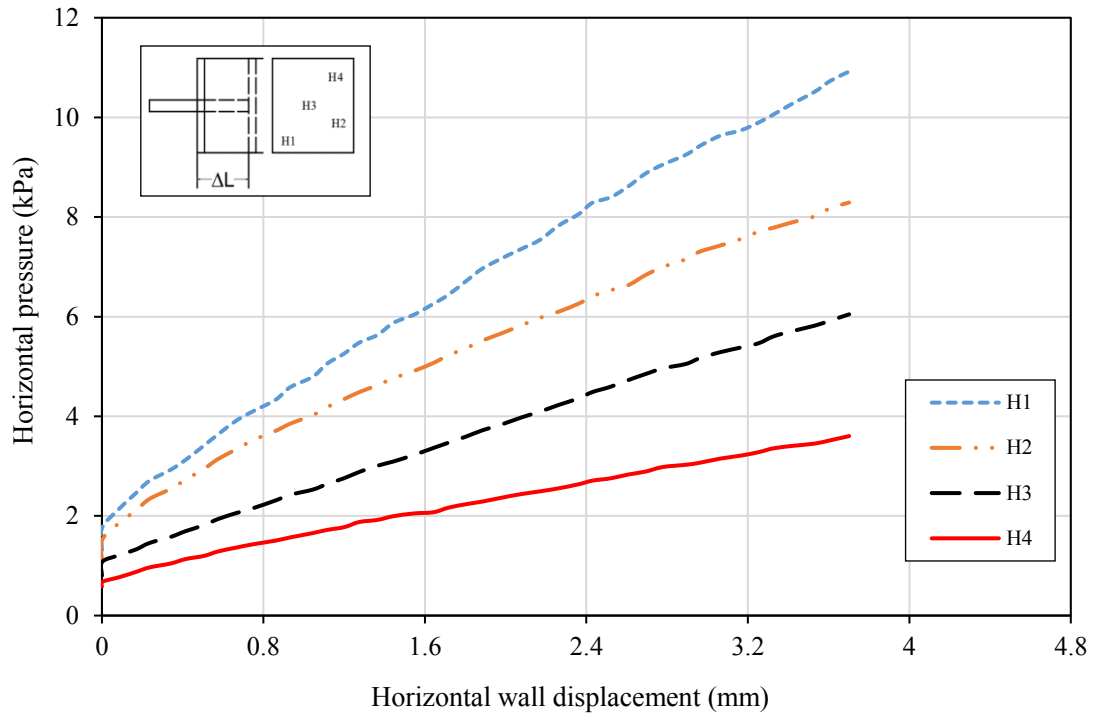
The experimental value of  $K_0$  was estimated using the equation:  $K_0 = \frac{\sigma'_h}{\sigma'_v}$ ; where  $\sigma'_h$  = at-rest earth pressure in kPa measured on the wall by the respective transducer;  $\sigma'_v$  = the corresponding vertical pressure at the same element. The experimental result of  $K_0$  is 0.45.

In order to validate the result given by the experimental setup,  $K_0$  was also calculated using Jaky's theory as the equation:  $K_0 = 1 - \sin \phi' = 1 - \sin 33.26^\circ = 0.452$  ( $\phi' = 33.26^\circ$  is angle of shearing resistance of the tested sand). It can be seen that the result given by the experimental setup agreed well with the result given by Jaky's theory.

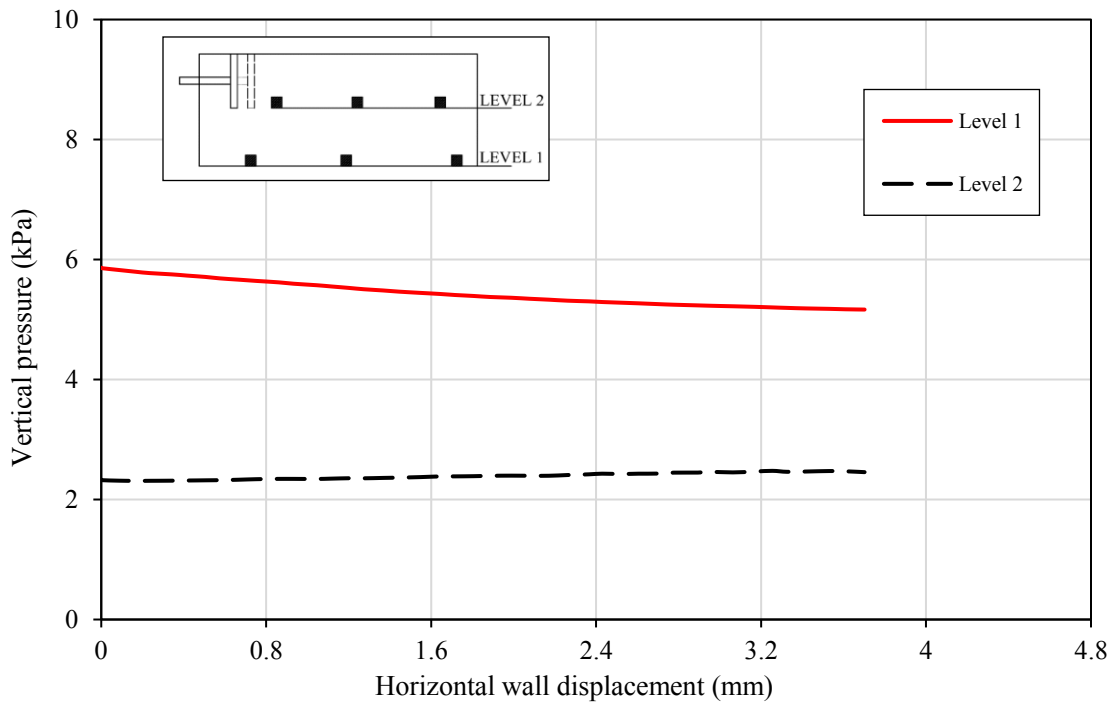
\* Passive earth pressure test with homogenous loose sand: the wall was pushed toward the retaining sand horizontally without any rotation by the gear box according to the displacement steps of approximately 0.5mm/minute in order to generate the passive earth pressure condition. The test result is presented diagrammatically in Figure 3.11.







(b)



(c)

**Fig. 3.11.** Test results: (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the sand mass versus the wall displacement

The experimental value of  $K_p$  was estimated as followings:  $K_p = \frac{P_p}{0.5\gamma H^2 b} = 3.42$

Where: P = passive earth force at failure (P = 228.647 N), measured by the load cell.

H = the height of the retaining wall (H= 215mm)

b = the width of the retaining wall (b=195mm)

$\gamma$  = the unit weight of the sand ( $\gamma = 14.82 \text{ kN/m}^3$ )

In order to validate the result given by the experimental setup,  $K_p$  was also calculated using Rankine's theory as followings:  $K_p = \tan^2\left(45 + \frac{\phi'}{2}\right) = \tan^2\left(45 + \frac{33.26}{2}\right) = 3.43$

It can be seen that the result given by the experimental setup was in good agreement with the result given by the Rankine's theory.

## **CHAPTER 4**

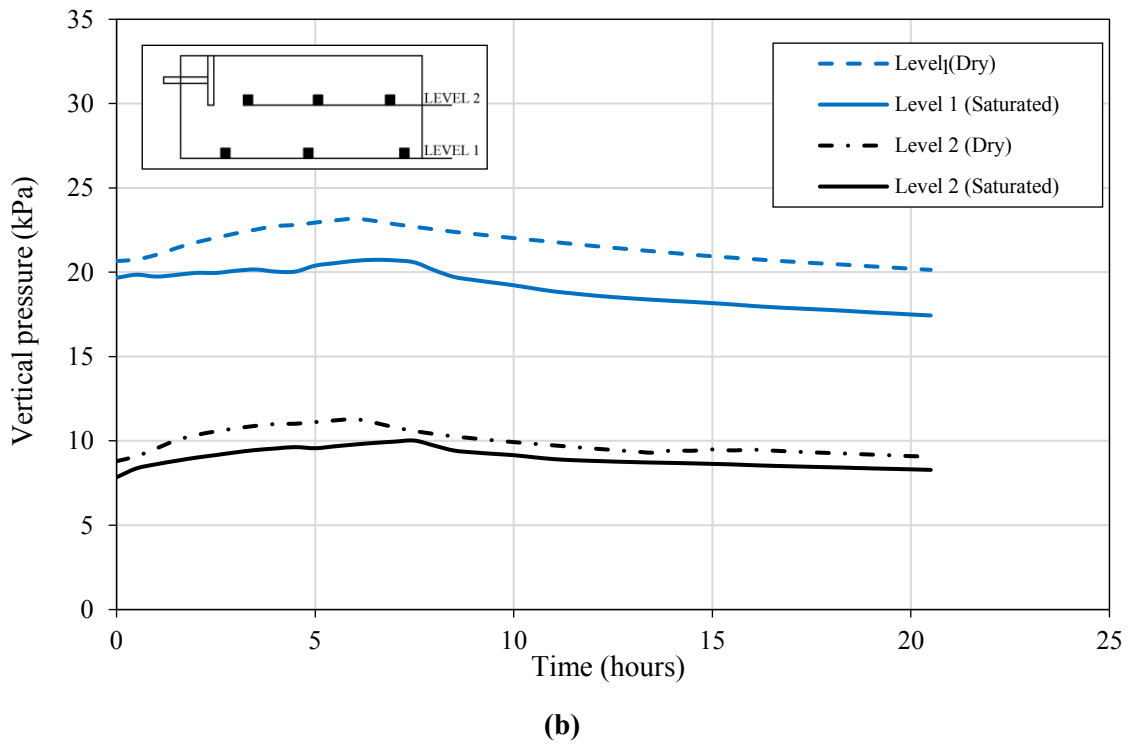
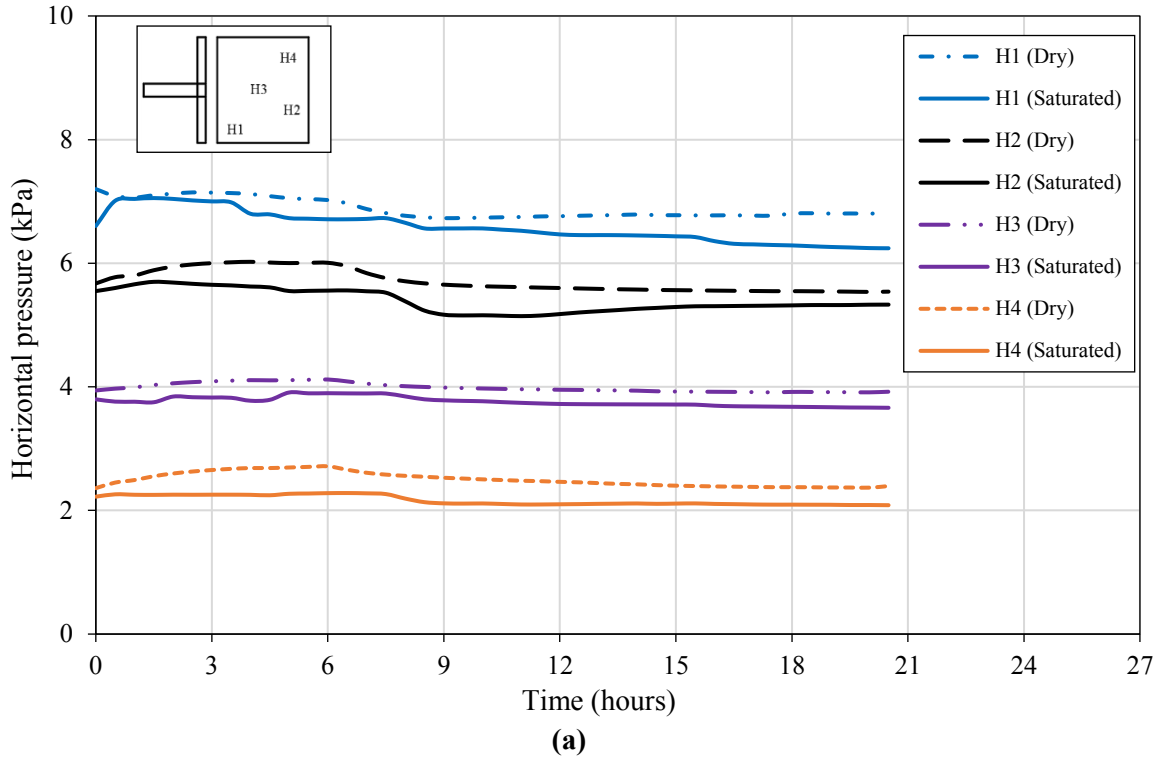
### **TEST RESULTS AND ANALYSIS FOR THE CASE OF AT-REST PRESSURE**

#### **4.1. General**

In this chapter, the results obtained from the present experimental investigation are presented in form of figures, following by discussions about these results. A series of comparing figures have been developed to indicate the difference of soil behavior under the same or different conditions. Finally, empirical formulae and design charts are proposed based on the analysis of these results.

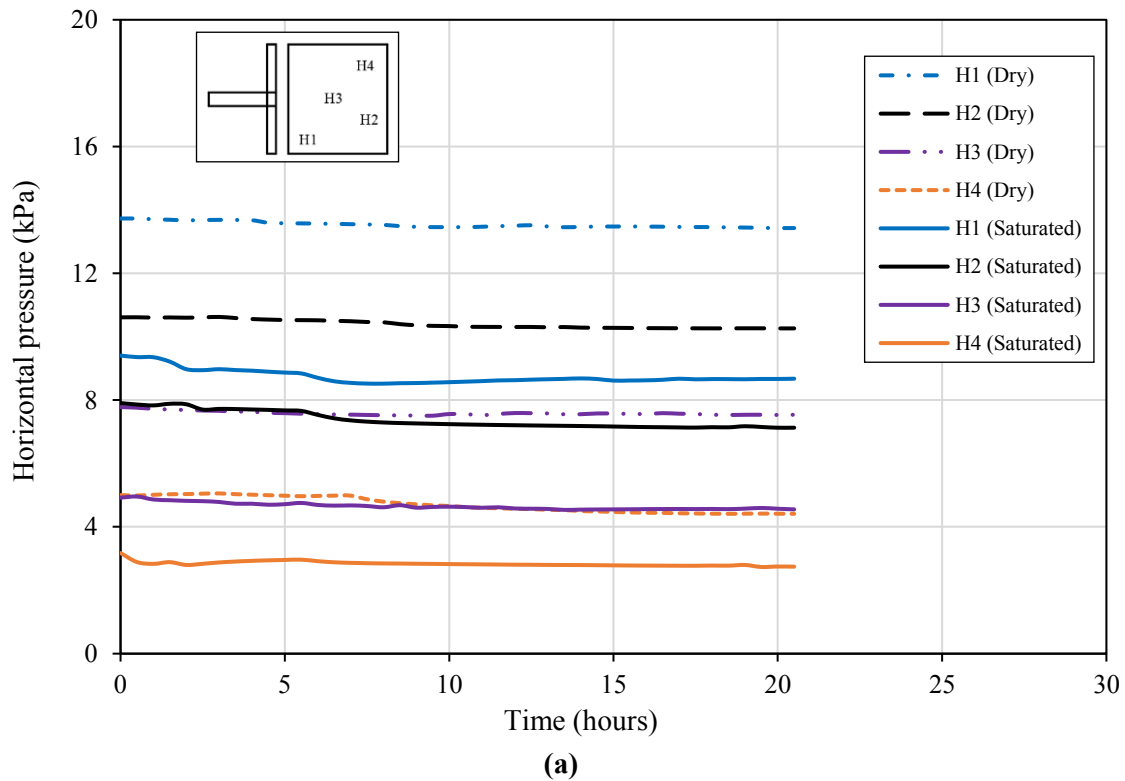
#### **4.2. Results and analysis**

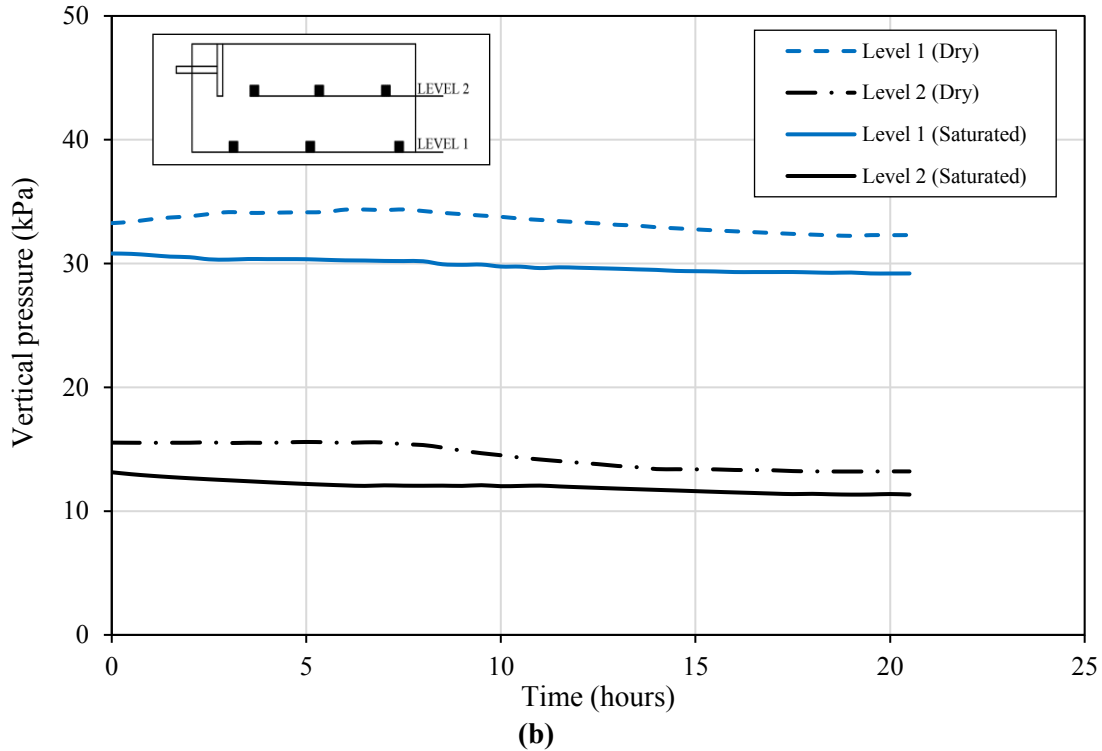
*Test no. 1 & 4* were performed on the dry and saturated soils, respectively, with 6% kaoline clay content, having collapse potential  $C_p$  of 4.2%. The test results are presented diagrammatically in Figure 4.1. From the Fig. 4.1a, it can be seen that at-rest earth pressure acting on the wall measured by transducers slightly changes with time for both dry and saturated soils. The decrease trend is more obvious in case of saturated soils. It can be seen from this figure that the at-rest earth pressure decreases slightly when the soil gets saturated. From the Fig. 4.1b, it can be seen that the vertical pressure in the soil mass slightly changes with time for both dry and saturated soils. The vertical pressure in the soil mass decreases when the soil is saturated.



**Fig. 4.1.** Results from test no. 1 & 4 for soil – A ( $C_p=4.2\%$ ): (a) Horizontal pressure measured by wall transducers versus time; (b) Vertical pressure in the soil mass measured by transducers versus time

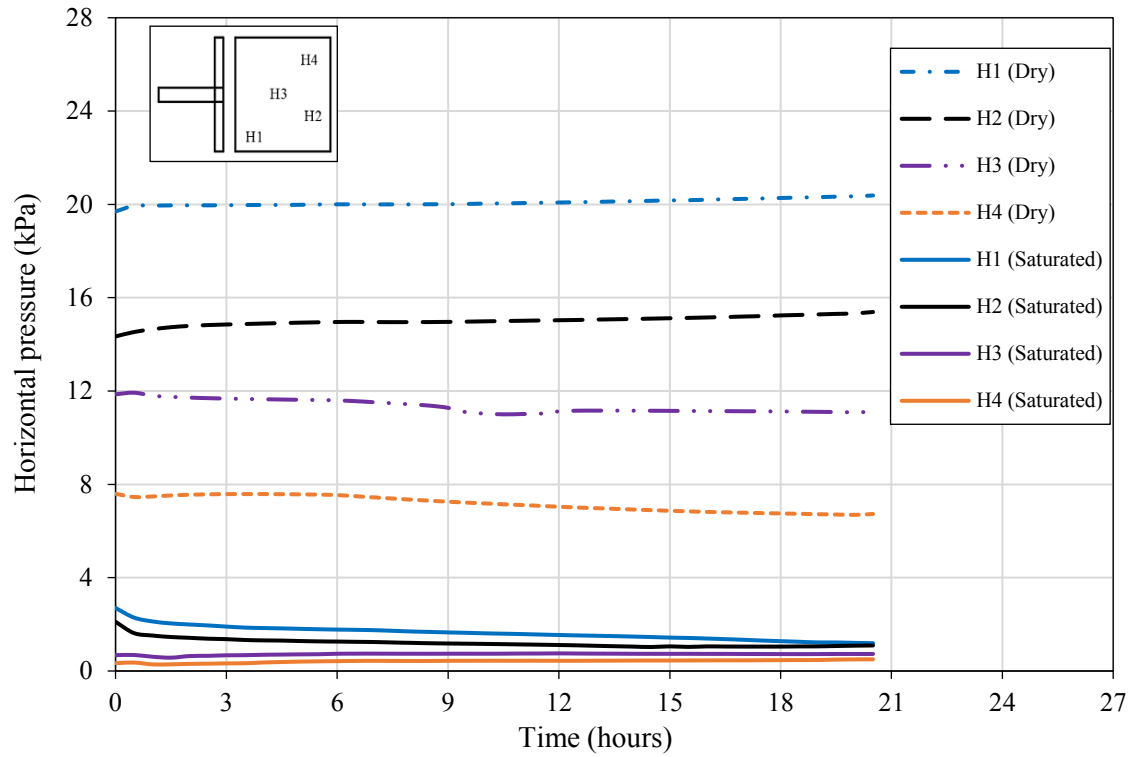
Test no. 2 & 5 were performed on the dry and saturated soils, respectively, with 10% kaoline clay content, having collapse potential  $C_p$  of 12.5%. The test results are presented diagrammatically in Figure 4.2. From the Fig. 4.2a, it can be seen that at-rest earth pressure measured on wall by transducers slightly changes with time for both dry and saturated soils. It can be seen from this figure that the at-rest earth pressure acting on the wall decreases considerably when the soil gets saturated. The at-rest earth pressure acting on the wall decreases with approximately 35% from dry soil to saturated soil. This decrease is strongly larger than that of the soil –  $C_p=4.2\%$ . From the Fig. 4.2b, it can be seen that the vertical pressure in the soil mass slightly decreases with time for both dry and saturated soils. The vertical pressure in the soil mass undergoes a decrease when the soil is saturated.



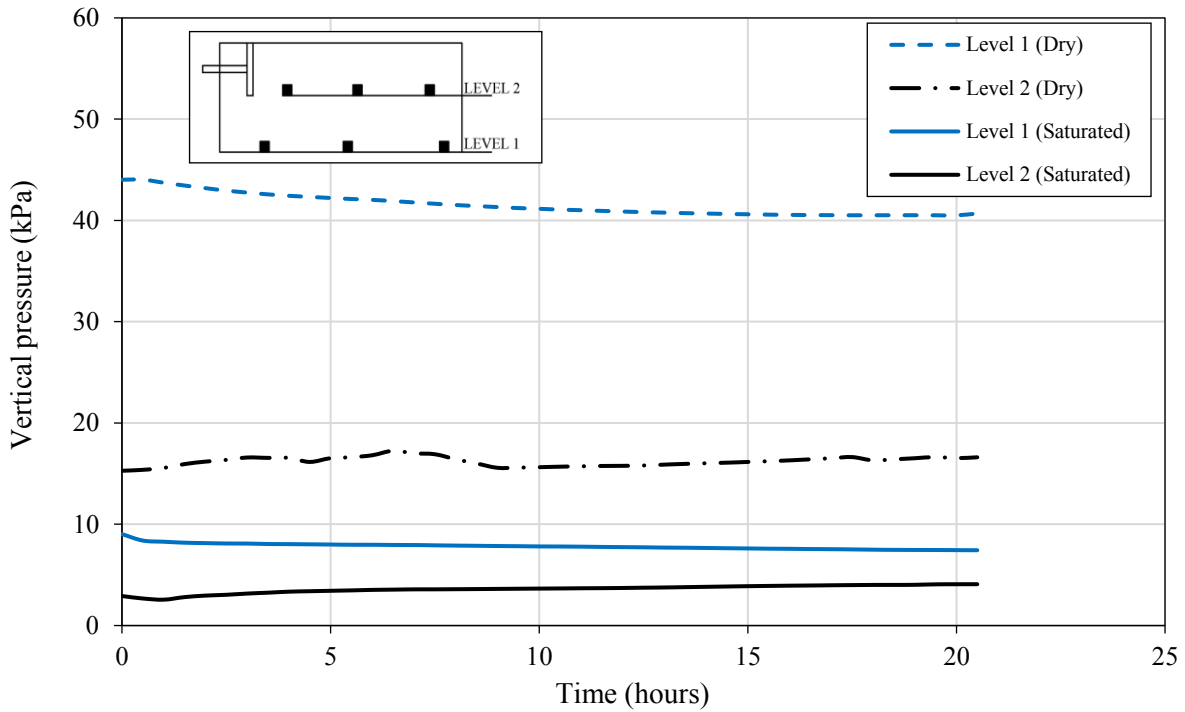


**Fig. 4.2.** Results from test no. 2 & 5 for soil – C ( $C_p=12.5\%$ ): (a) Horizontal pressure measured by wall transducers versus time; (b) Vertical pressure in the soil mass measured by transducers versus time

*Test no. 3 & 6* were performed on the dry and saturated soils, respectively, with 14% kaoline clay content, having collapse potential of 18%. The test results are presented diagrammatically in Figure 4.3. From the Fig. 4.3a, it can be seen from this figure that the at-rest earth pressure acting on the wall decreases significantly when the soil behind the wall gets full inundated. The at-rest earth pressure acting on the wall decreases with roughly 80% from dry soil to saturated soil. From the Fig. 4.3b, it can be seen that the vertical pressure in the soil mass undergoes a significant decrease for the case of saturated soils. The vertical pressure in the soil mass decreases with about 80% from dry soil to saturated soil.



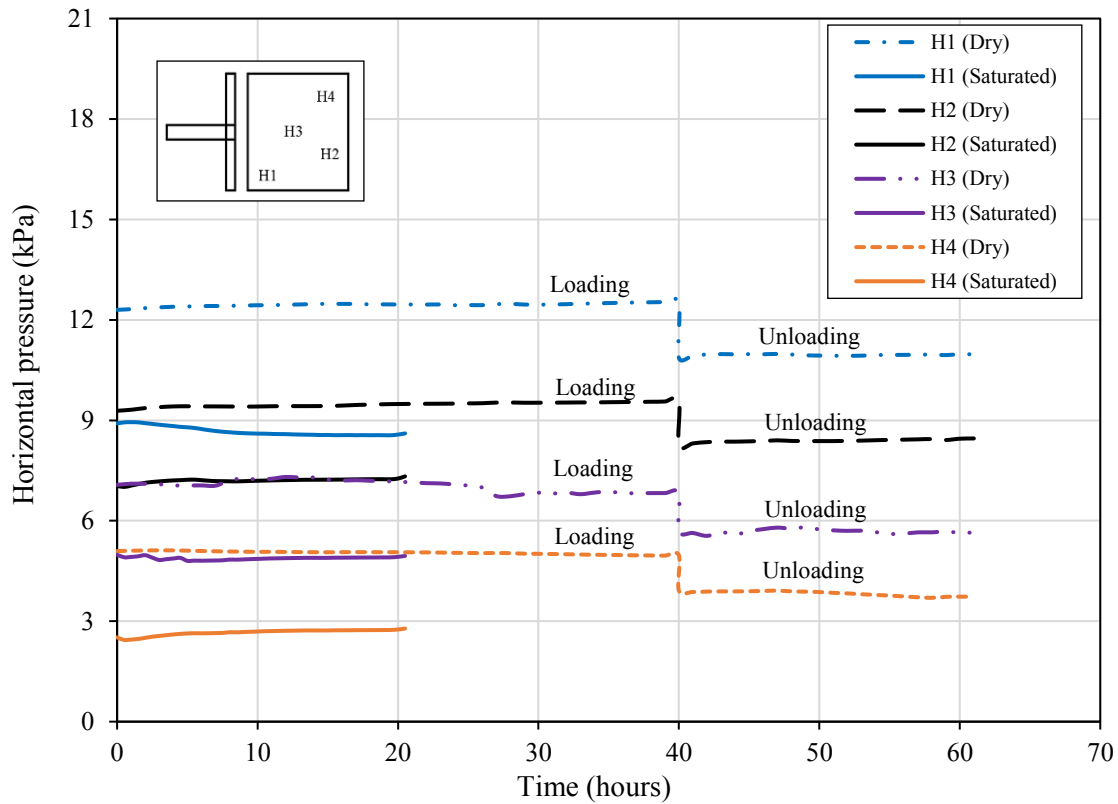
(a)



(b)

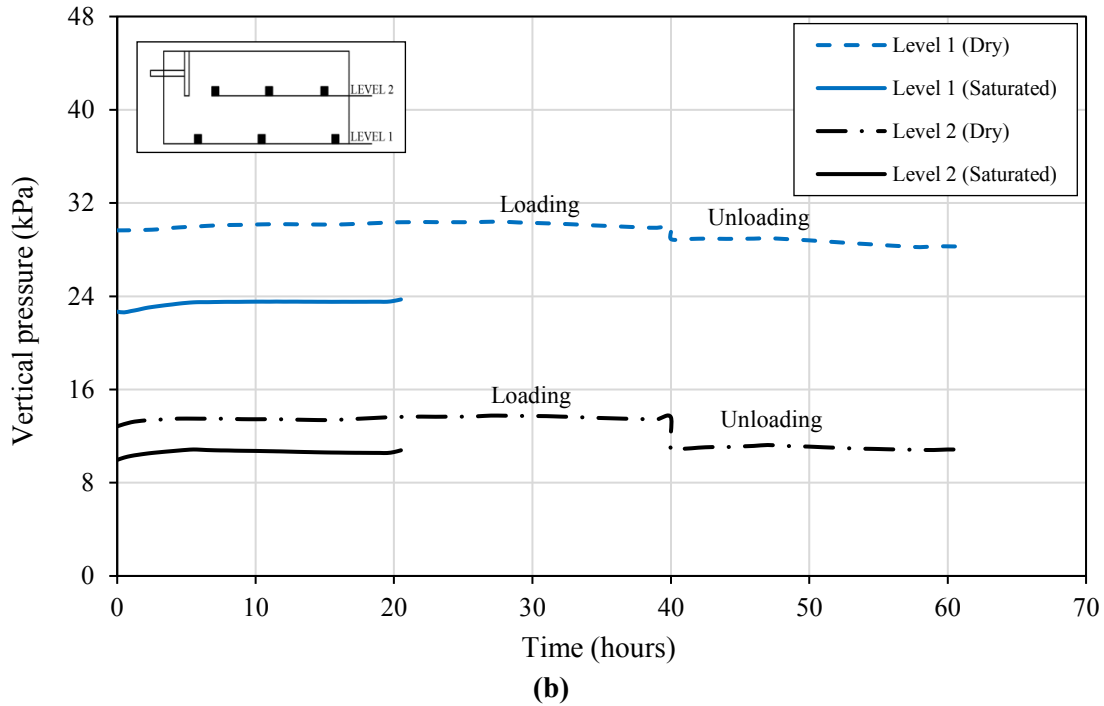
**Fig. 4.3.** Results from test no. 3 & 6 for soil – D ( $C_p=18\%$ ): (a) Horizontal pressure measured by wall transducers versus time; (b) Vertical pressure in the soil mass measured by transducers versus time

Test no. 7 & 10 were performed on the dry and saturated soils, respectively, with 6% kaoline clay content, having collapse potential  $C_p$  of 4.2%, applying surcharge  $P_s = 8$  kPa. The test results are presented diagrammatically in Figure 4.4. It is obvious that the at-rest earth pressure increases considerably when increasing the stress level in the soil mass (OCR) by applying  $P_s$ . From the Fig. 4.4a, it can be seen that at-rest earth pressure acting on the wall varies from loading to unloading state. It should be noted that the at-rest earth pressure acting on the wall decreases slightly when the soil gets saturated. From the Fig. 4.4b, it can be seen that the vertical pressure in the soil mass slightly changes with time for both dry and saturated soils. The vertical pressure in the soil mass decreases when the soil is saturated.



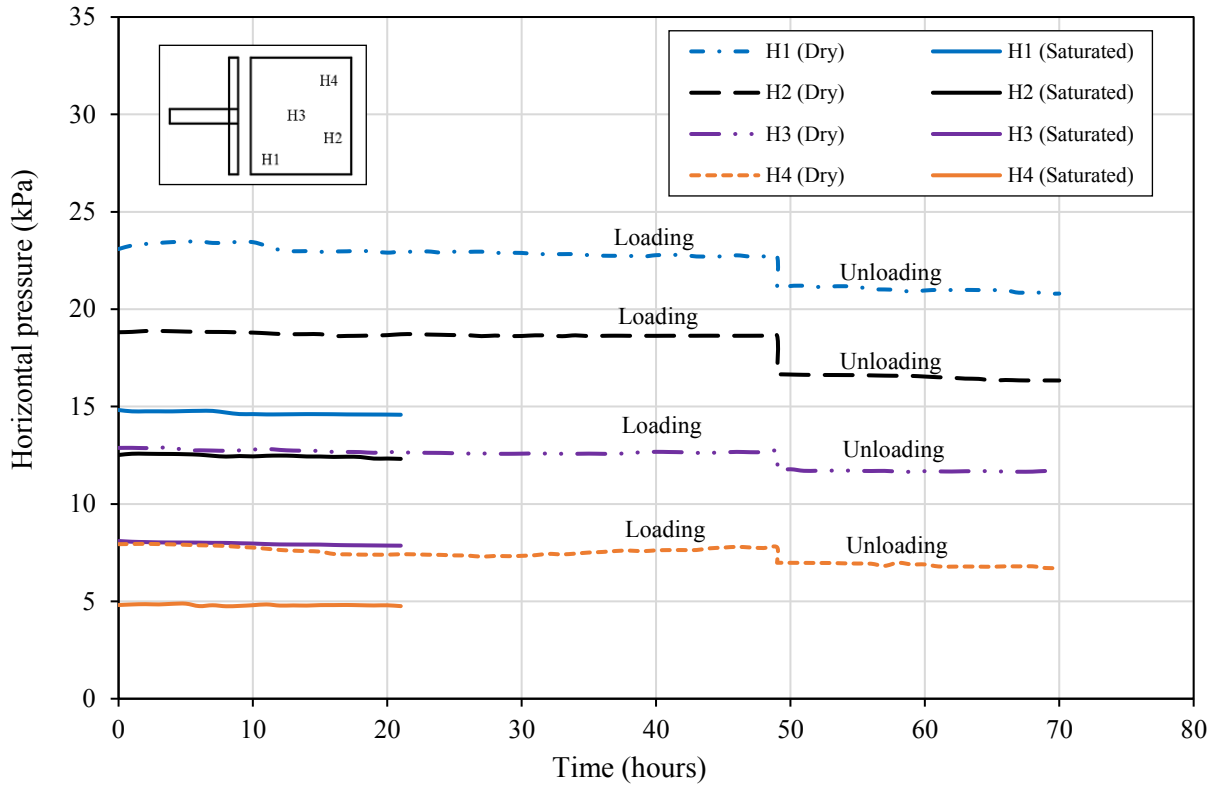
(a)



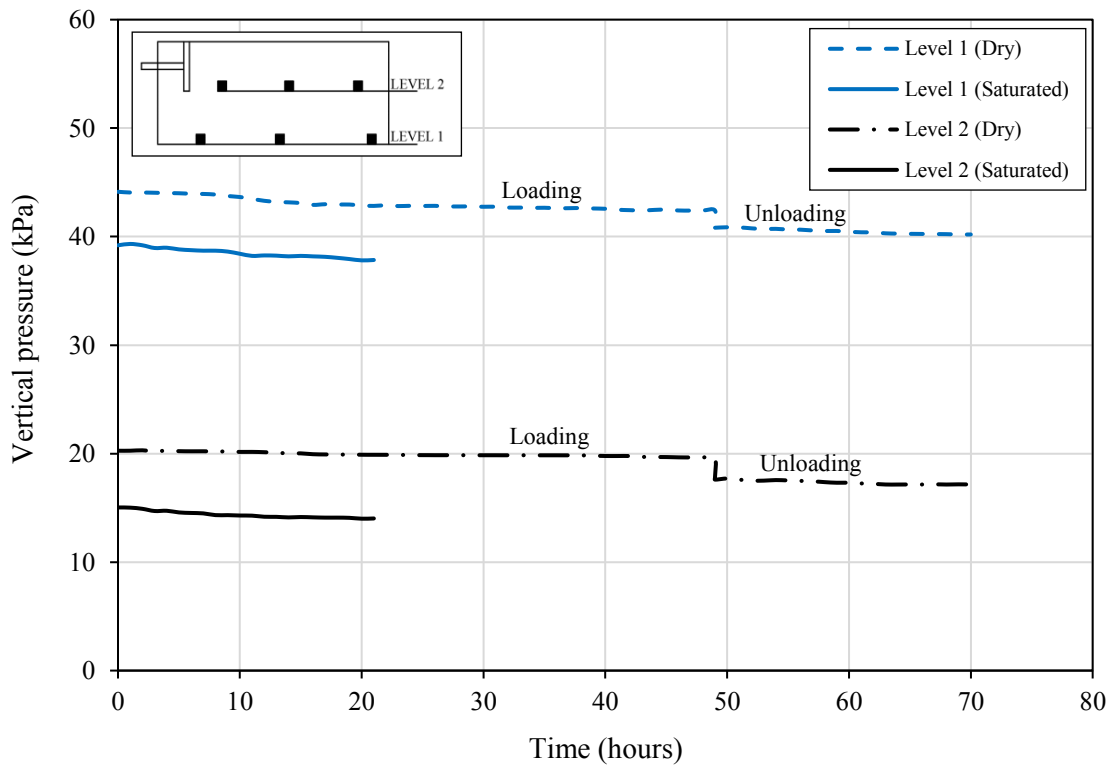


**Fig. 4.4.** Results from test no. 7 & 10 for soil – A ( $C_p=4.2\%$ ): (a) Horizontal pressure measured by wall transducers versus time; (b) Vertical pressure in the soil mass measured by transducers versus time

*Test no. 8 & 11* were performed on the dry and saturated soils, respectively, with 10% kaoline clay content, having collapse potential  $C_p$  of 12.5%, applying surcharge  $P_s = 8$  kPa. The test results are presented graphically in Figure 4.5. The at-rest earth pressure increases considerably when increasing the stress level in the soil mass (OCR) by applying  $P_s$ . From the Fig. 4.5a, it can be seen that at-rest earth pressure acting on the wall varies from loading to unloading condition. The difference of the values of at-rest earth pressure between loading and unloading also depends on the depth. It can be noted from this figure that the at-rest earth pressure acting on the wall decreases considerably when the soil behind the wall gets saturated. From the Fig. 4.5b, it can be seen that the vertical pressure in the soil mass slightly changes with time for both dry and saturated soils. The vertical pressure in the soil mass decreases slightly due to soil's saturation.



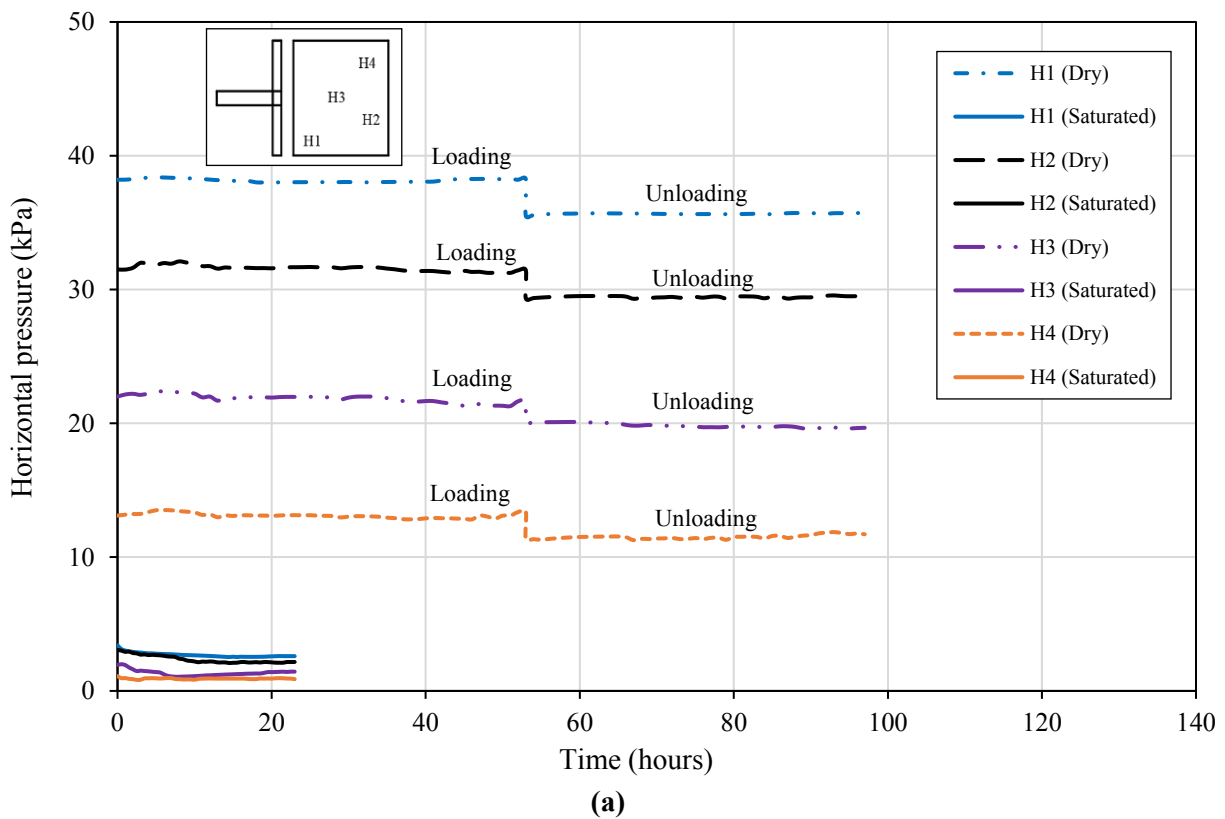
(a)

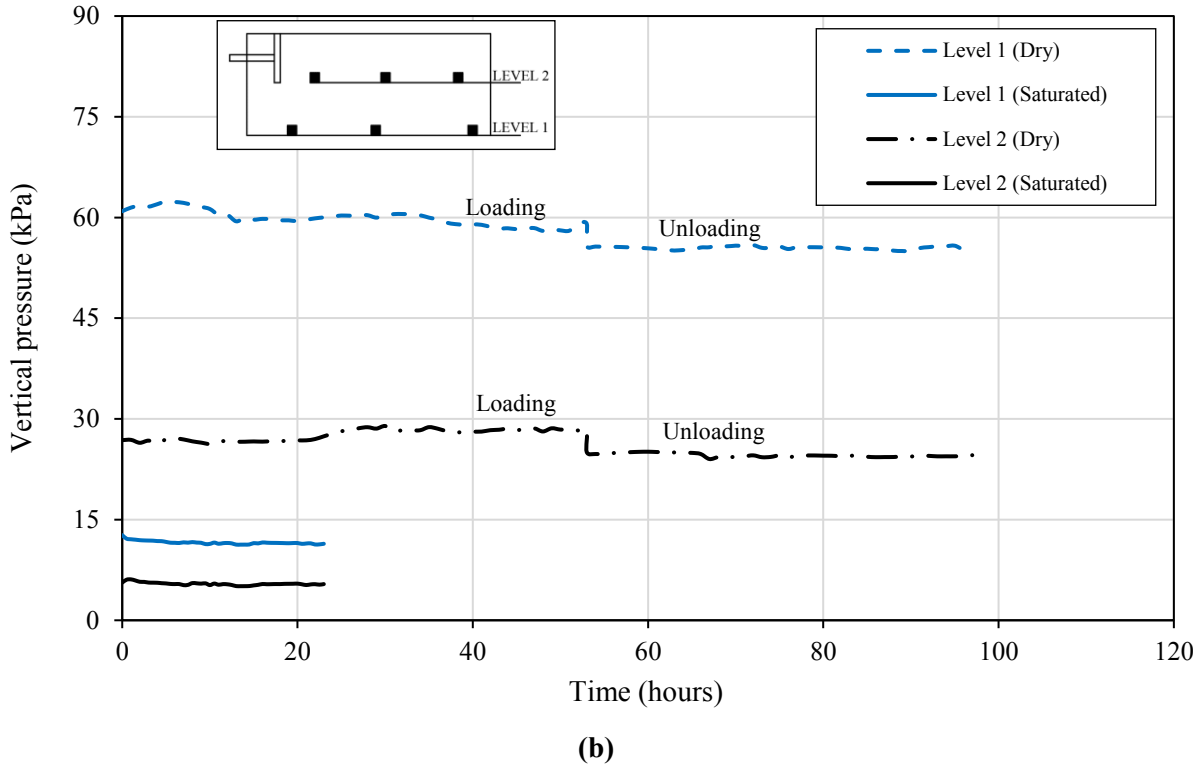


(b)

**Fig. 4.5.** Results from test no. 8 & 11 for soil – C ( $C_p=12.5\%$ ): (a) Horizontal pressure measured by wall transducers versus time; (b) Vertical pressure in the soil mass measured by transducers versus time

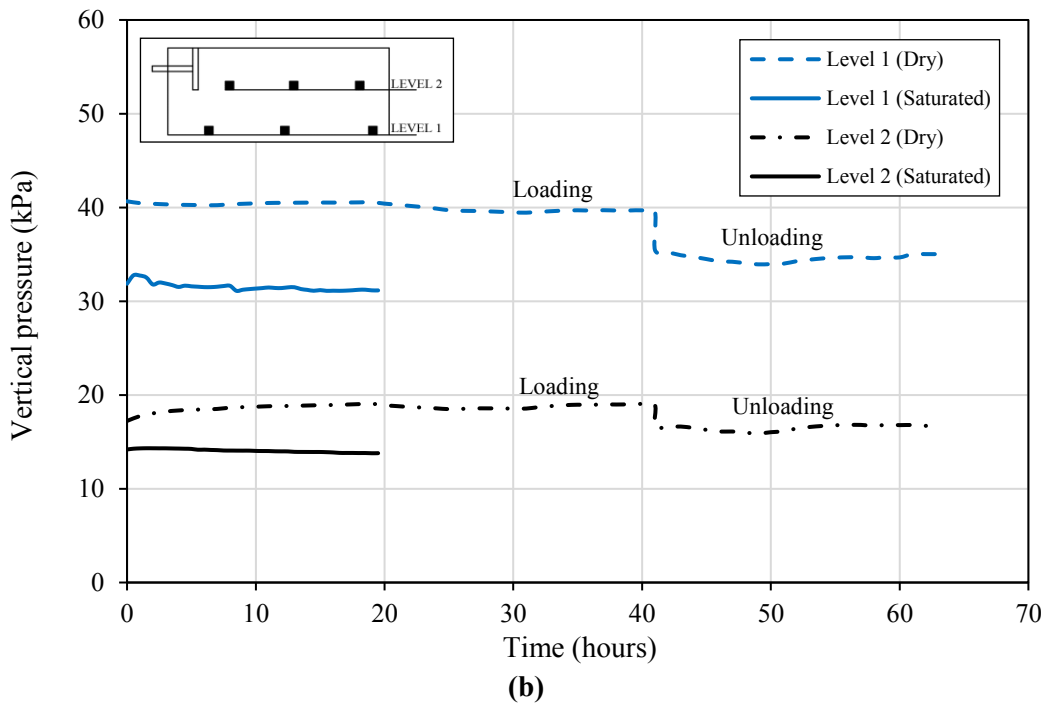
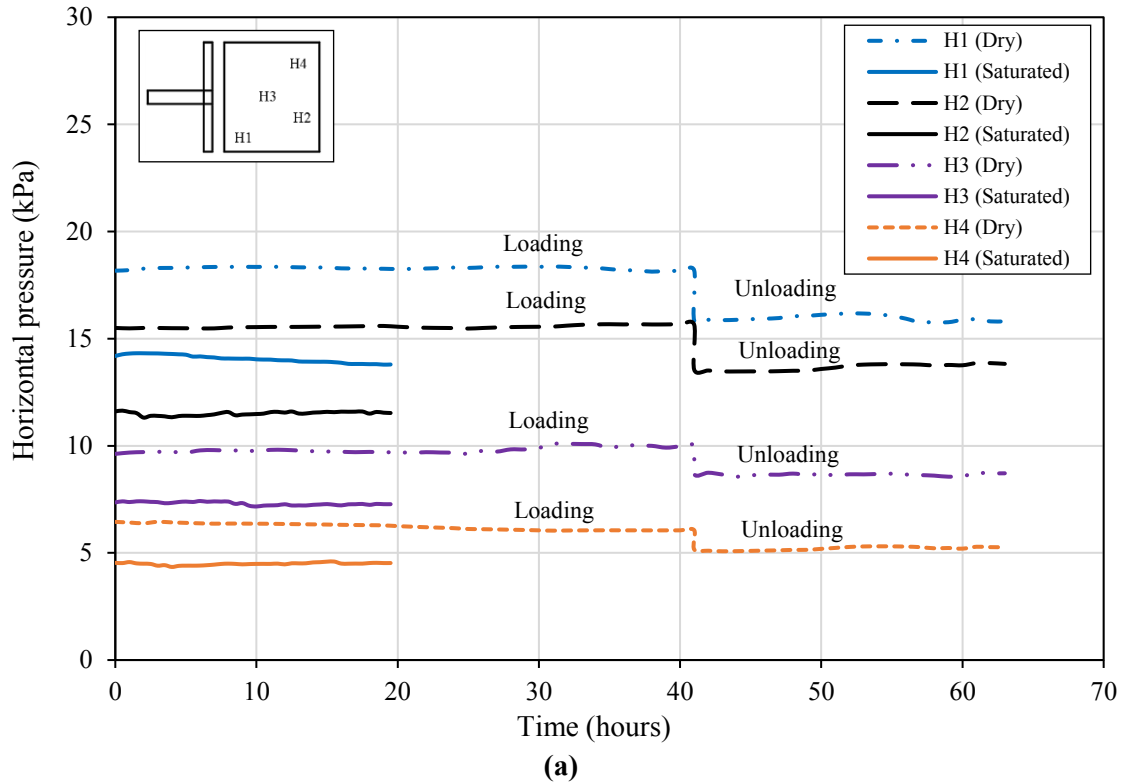
Test no. 9 & 12 were performed on the dry and saturated soils, respectively, with 14% kaoline clay content, having collapse potential  $C_p$  of 18%, applying surcharge  $P_s = 8$  kPa. The test results is presented graphically in Figure 4.6. From the Fig. 4.6a, it can be seen that at-rest earth pressure acting on the wall varies from loading to unloading state. It is very clear from this figure that the at-rest earth pressure acting on the wall decreases significantly when the soil behind the wall gets full inundated. The at-rest earth pressure decreases approximately 90% due to the soil's saturation. From the Fig. 4.6b, it can be seen that the vertical pressure in the soil mass slightly changes with time for both dry and saturated soils. The vertical pressure in the soil mass decreases remarkably upon saturated.





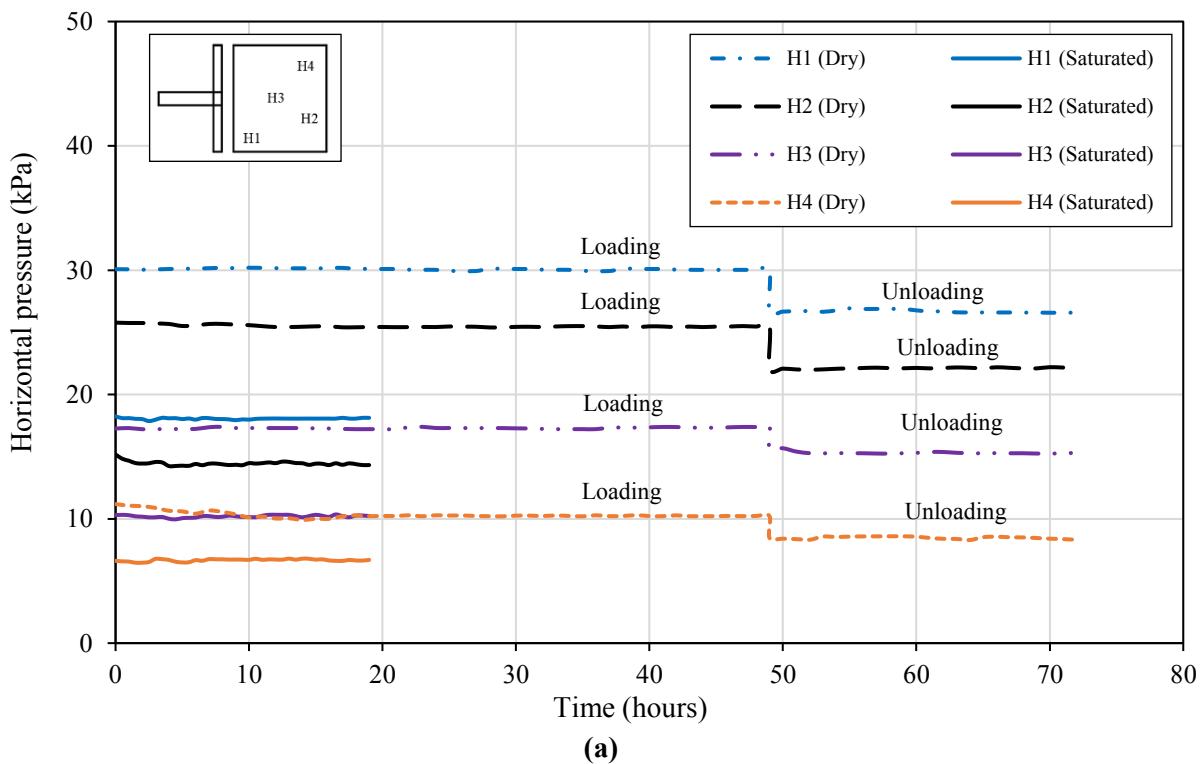
**Fig. 4.6.** Results from test no. 9 & 12 for soil – D ( $C_p=18\%$ ): (a) Horizontal pressure measured by wall transducers versus time; (b) Vertical pressure in the soil mass measured by transducers versus time

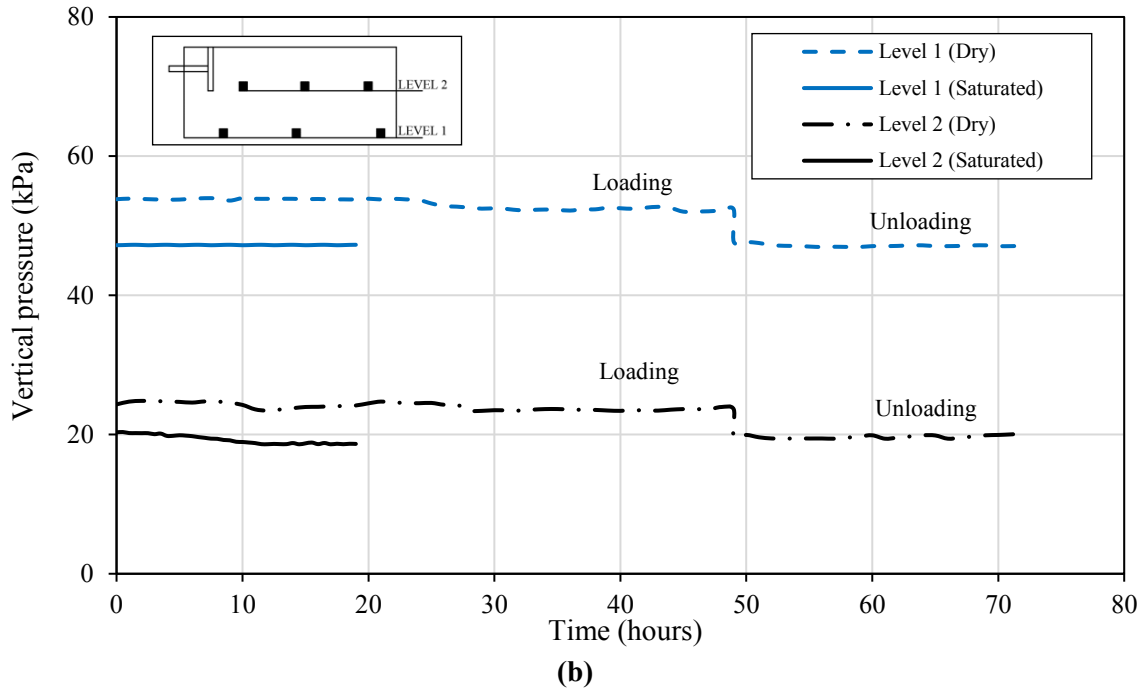
*Test no. 13 & 16* were performed on the dry and saturated soils, respectively, with 6% kaoline clay content, having collapse potential  $C_p$  of 4.2%, applying surcharge  $P_s = 16$  kPa. The test results are presented diagrammatically in Figure 4.7. It is obvious that the at-rest earth pressure increases considerably when increasing the stress level in the soil mass (OCR) by increasing value of  $P_s$ . From the Fig. 4.7a, it can be seen that at-rest earth pressure acting on the wall varies from loading to unloading state. It should be noted that the at-rest earth pressure acting on the wall decreases slightly when the soil is saturated. From the Fig. 4.7b, it can be seen that the vertical pressure in the soil mass slightly changes with time for both dry and saturated soils. The vertical pressure in the soil mass decreases when the soil gets saturated.



**Fig. 4.7.** Results from test no. 13 & 16 for soil – A ( $C_p=4.2\%$ ): (a) Horizontal pressure measured by wall transducers versus time; (b) Vertical pressure in the soil mass measured by transducers versus time

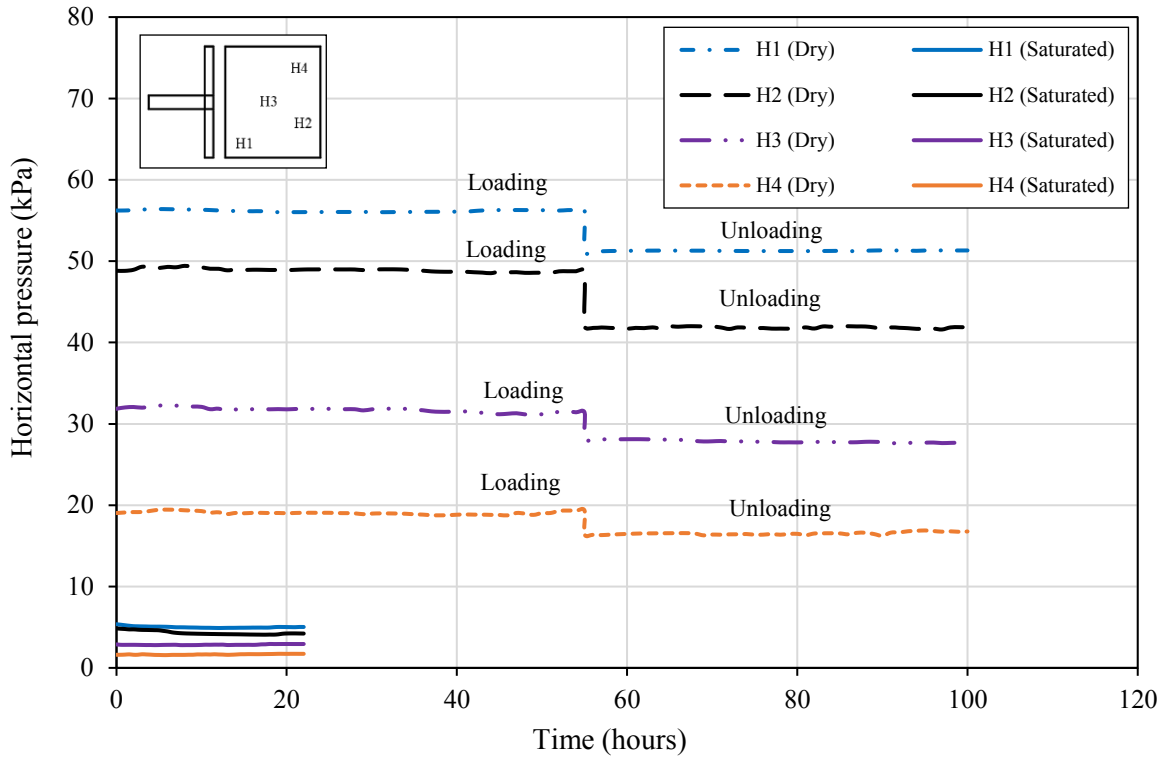
Test no. 14 & 17 were performed on the dry and saturated soils, respectively, with 10% kaoline clay content, having collapse potential  $C_p$  of 12.5%, applying  $P_s = 16$  kPa. The test results are presented diagrammatically in Figure 4.8. It is obvious that the at-rest earth pressure increases considerably when increasing OCR by increasing  $P_s$ . From the Fig. 4.8a, it can be seen that at-rest earth pressure acting on the wall varies from loading to unloading state. It should be noted that the at-rest earth pressure acting on the wall decreases considerably due to the soil's saturation. It is of interest to note from the Fig.4.8a that the at-rest earth pressure decreases approximately 32% due to the soil's saturation. From the Fig. 4.8b, it can be seen that the vertical pressure on the soil mass slightly changes with time for both dry and saturated soils. The vertical pressure in the soil mass decreases due to the saturation of the soil.



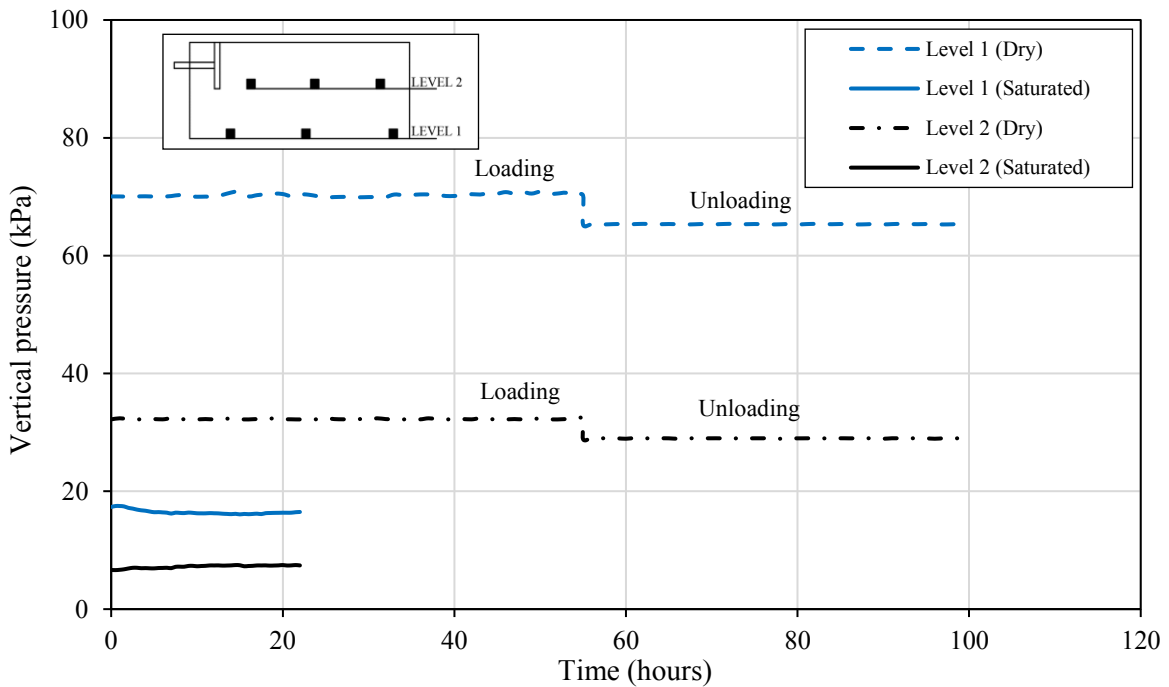


**Fig. 4.8.** Results from test no. 14 & 17 for soil – C ( $C_p=12.5\%$ ): (a) Horizontal pressure measured by wall transducers versus time; (b) Vertical pressure in the soil mass measured by transducers versus time

*Test no. 15 & 18* were performed on the dry and saturated soils, respectively, with 14% kaoline clay content, having collapse potential  $C_p$  of 18%, applying  $P_s = 16$  kPa. The test results are presented diagrammatically in Figure 4.9. Firstly, it is noted that the at-rest earth pressure increases considerably when increasing OCR by increasing  $P_s$ . From the Fig. 4.9a, it can be seen that at-rest earth pressure acting on the wall changes from loading to unloading conditions. It should be noted that the at-rest earth pressure acting on the wall decreases significantly due to the soil's full inundation. The at-rest earth pressure decreases approximately 90% from dry condition to saturated condition. From the Fig. 4.9b, it can be seen that the vertical pressure in the soil mass decreases outstandingly due to the saturation of the soil. The vertical pressure in the soil mass decreases around 75% due to the soil's saturation.



(a)

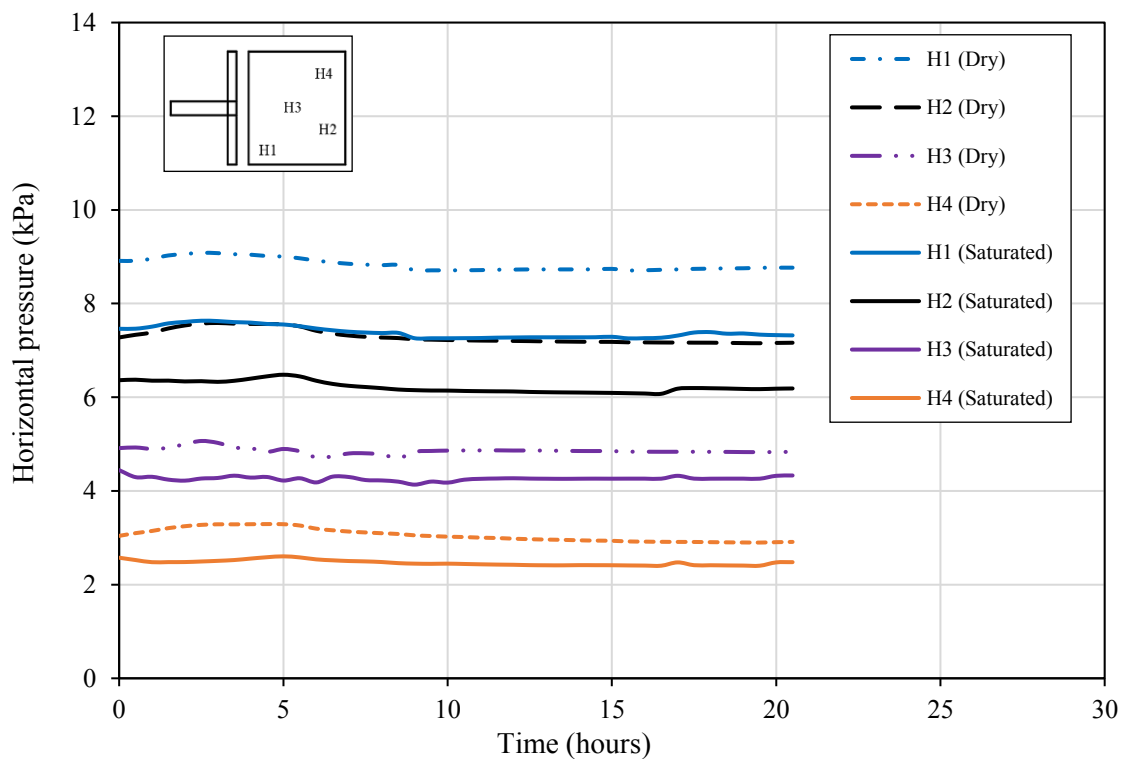


(b)

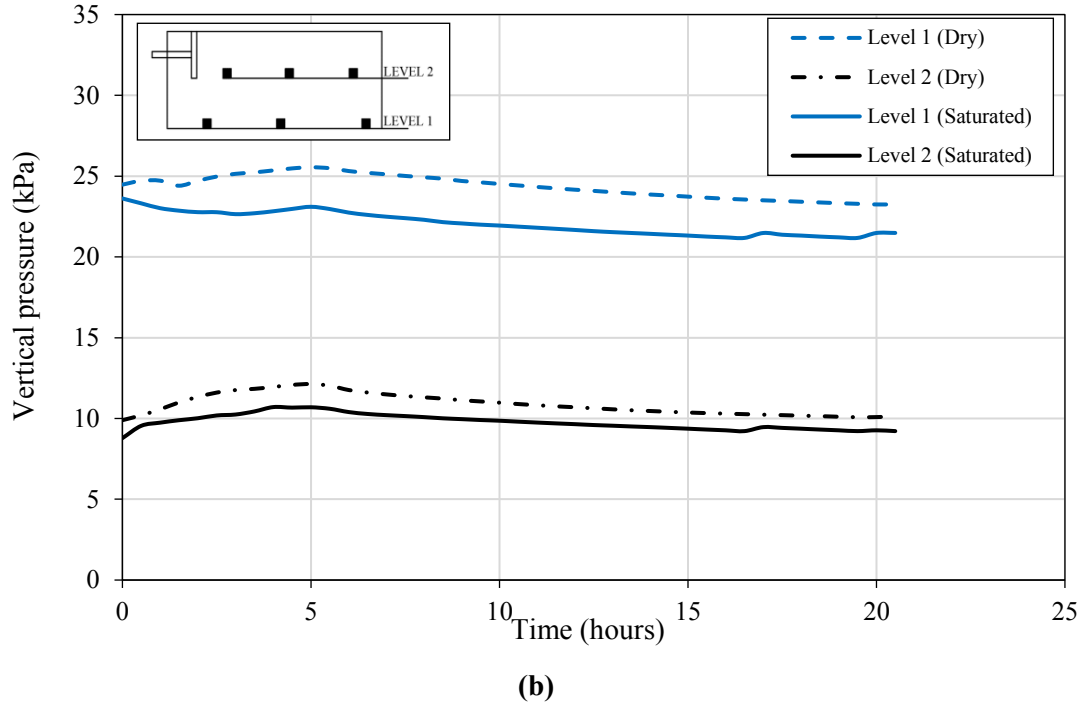
**Fig. 4.9.** Results from test no. 15 & 18 for soil – D ( $C_p=18\%$ ): (a) Horizontal pressure measured by wall transducers versus time; (b) Vertical pressure in the soil mass measured by transducers versus time



Test no. 19 & 22 were performed on the dry and saturated soils, respectively, with 8% kaoline clay content, having collapse potential  $C_p$  of 9.0%. The test results are presented diagrammatically in Figure 4.10. From the Fig. 4.10a, it can be seen that at-rest earth pressure measured on wall by transducers slightly changes with time for both dry and saturated soils. It can be seen from this figure that the at-rest earth pressure on the wall decreases moderately when the soil gets saturated. From the Fig. 4.10b, it can be seen that the vertical pressure in the soil mass slightly decreases with time for both dry and saturated soils. The vertical pressure in the soil mass experiences a decrease when the soil gets saturated.

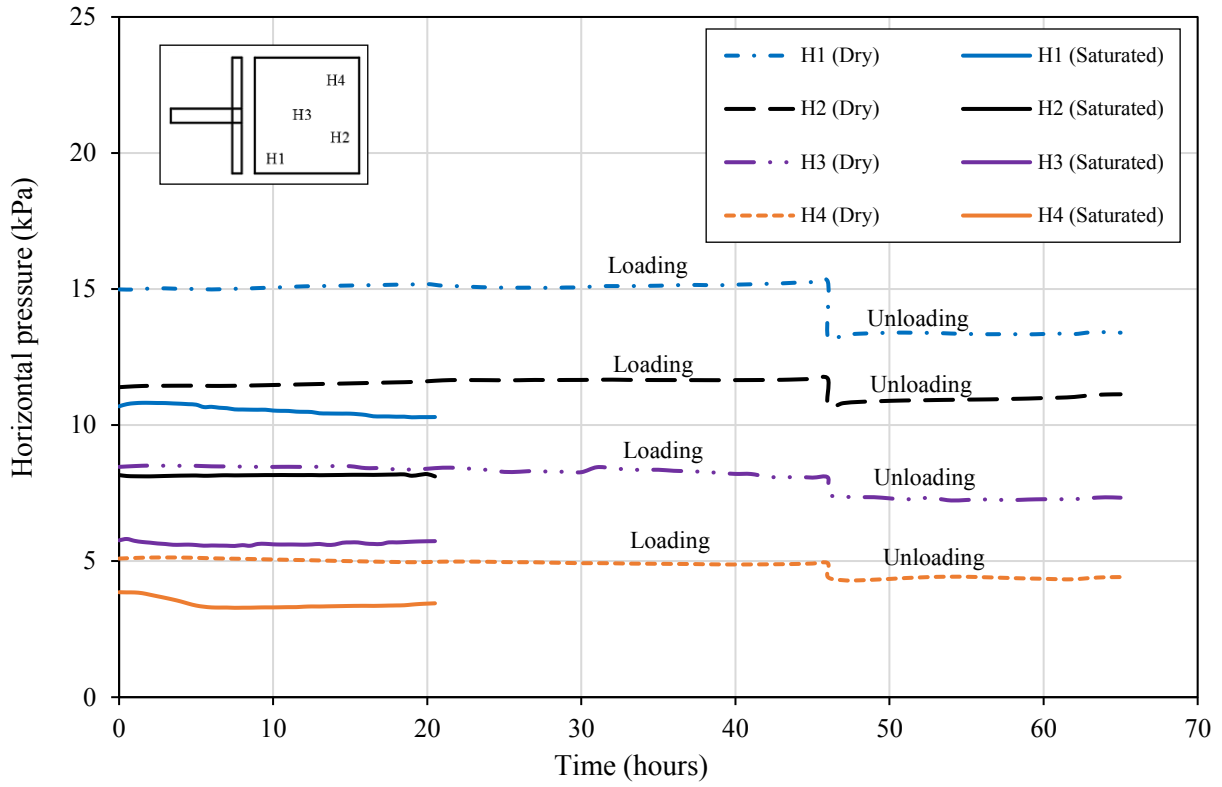


(a)

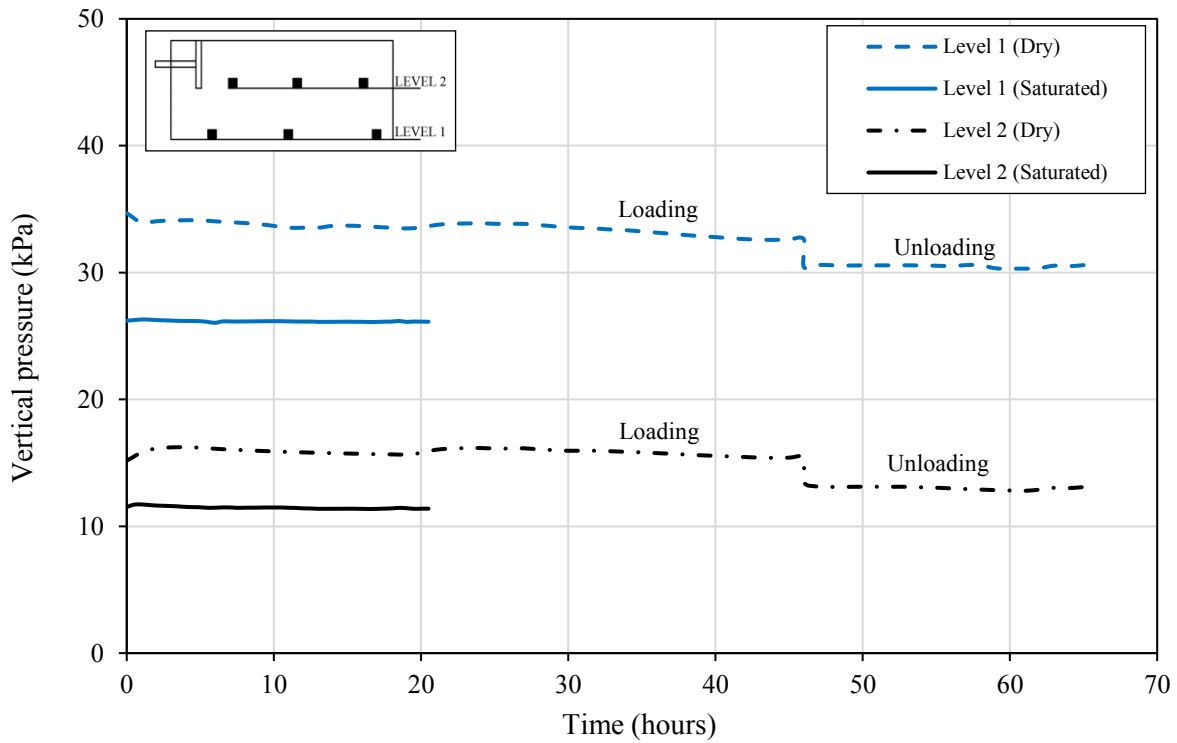


**Fig. 4.10.** Results from test no. 19 & 22 for soil – B ( $C_p=9.0\%$ ): (a) Horizontal pressure measured by wall transducers versus time; (b) Vertical pressure in the soil mass measured by transducers versus time

Test no. 20 & 23 were performed on the dry and saturated soils, respectively, with 8% kaoline clay content, having collapse potential of 9.0%, applying  $P_s = 8$  kPa. The test results are presented graphically in Figure 4.11. It is clear that the at-rest earth pressure increases considerably when increasing the stress level in the soil mass (OCR) by applying  $P_s$ . From the Fig. 4.11a, it can be seen that at-rest earth pressure acting on the wall varies from loading to unloading state. It can be noted from this figure that the at-rest earth pressure acting on the wall decreases moderately due to saturation of the soils. From the Fig. 4.11b, it can be seen that the vertical pressure in the soil mass slightly changes with time for both dry and saturated soils. The vertical pressure in the soil mass decreases due to soil's saturation.



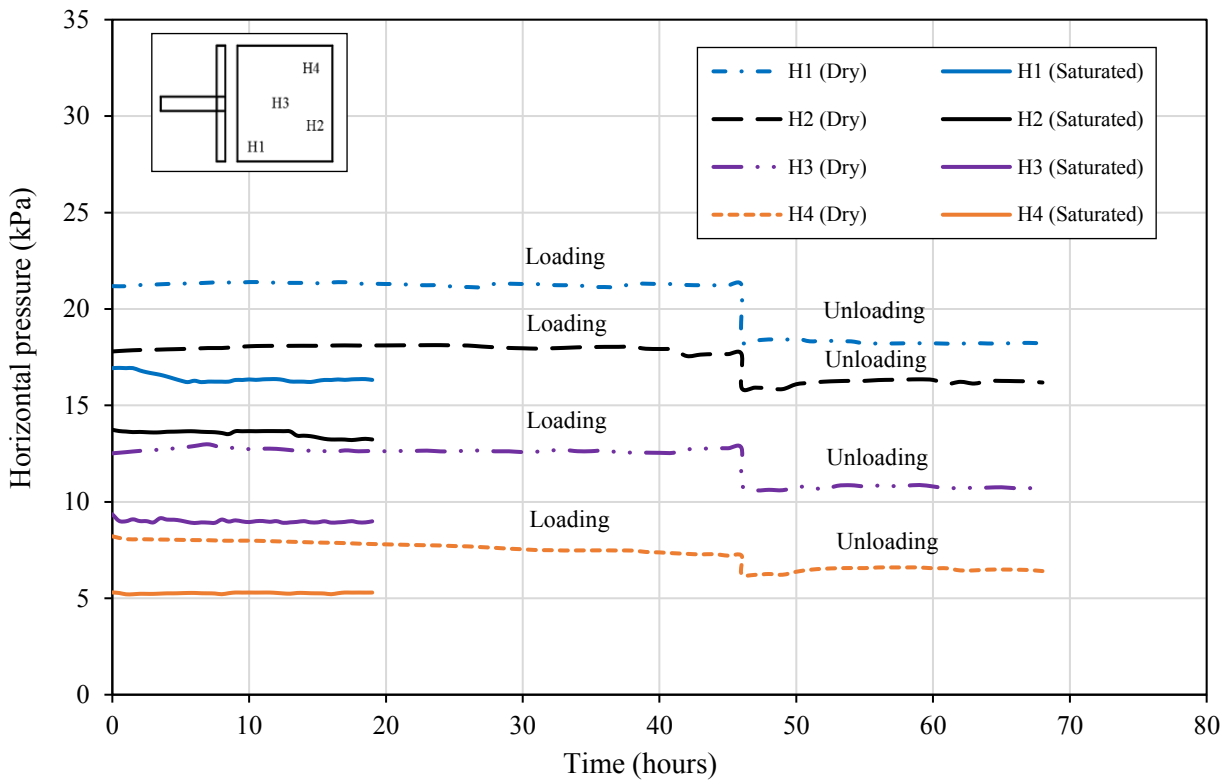
(a)



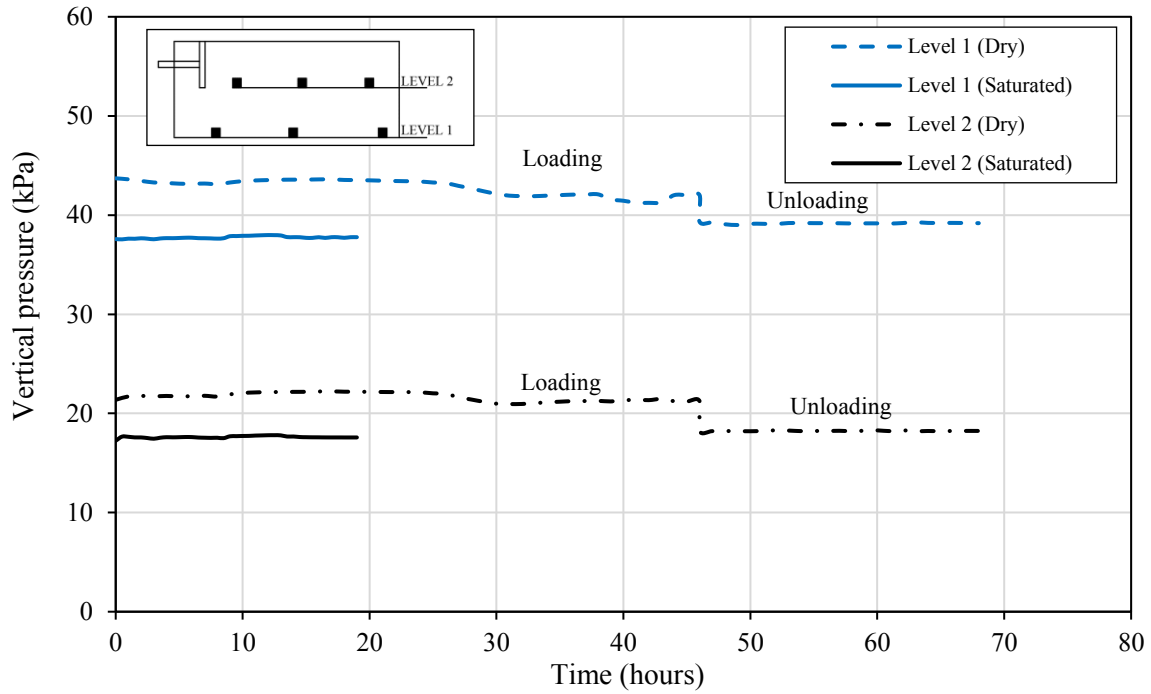
(b)

**Fig. 4.11.** Results from test no. 20 & 23 for soil – B ( $C_p=9.0\%$ ): (a) Horizontal pressure measured by wall transducers versus time; (b) Vertical pressure in the soil mass measured by transducers versus time

Test no. 21 & 24 were performed on the dry and saturated soils, respectively, with 8% kaoline clay content, having collapse potential  $C_p$  of 9.0%, applying  $P_s = 16$  kPa. The test results are presented diagrammatically in Figure 4.12. It is obvious that the at-rest earth pressure increases considerably when increasing OCR by increasing  $P_s$ . From the Fig. 4.12a, it can be seen that at-rest earth pressure acting on the wall varies from loading to unloading state. It should be noted from this figure that the at-rest earth pressure acting on the wall decreases moderately due to the soil's full inundation. From the Fig. 4.12b, it can be seen that the vertical pressure on the soil mass slightly varies with time for both dry and saturated soils. The vertical pressure in the soil mass decreases upon saturated.



(a)



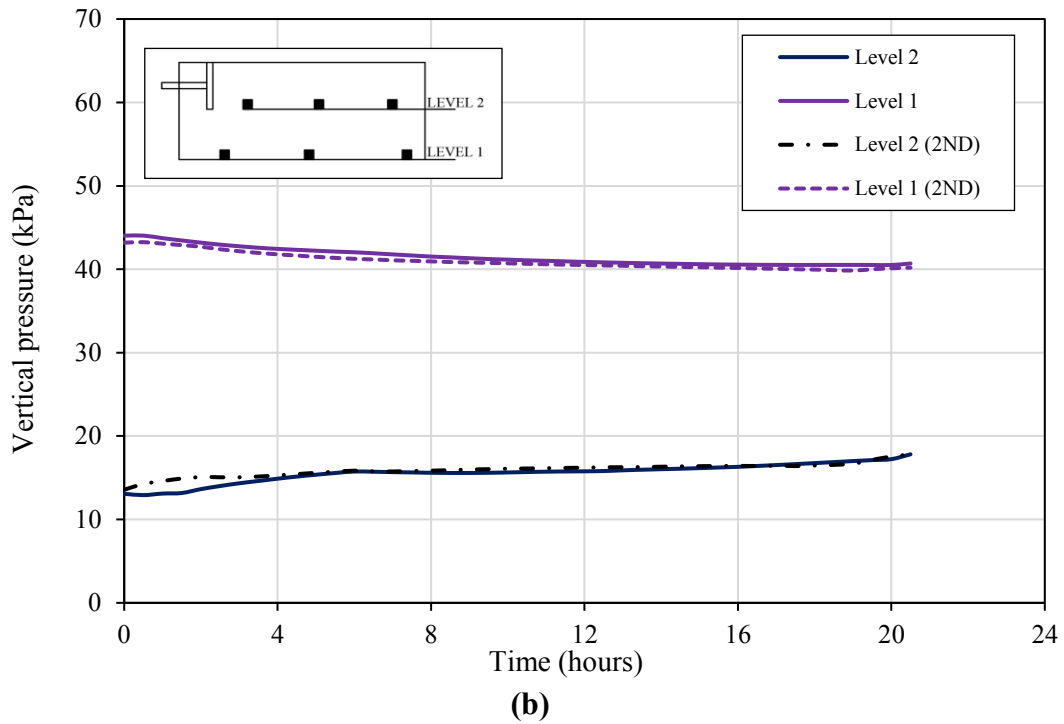
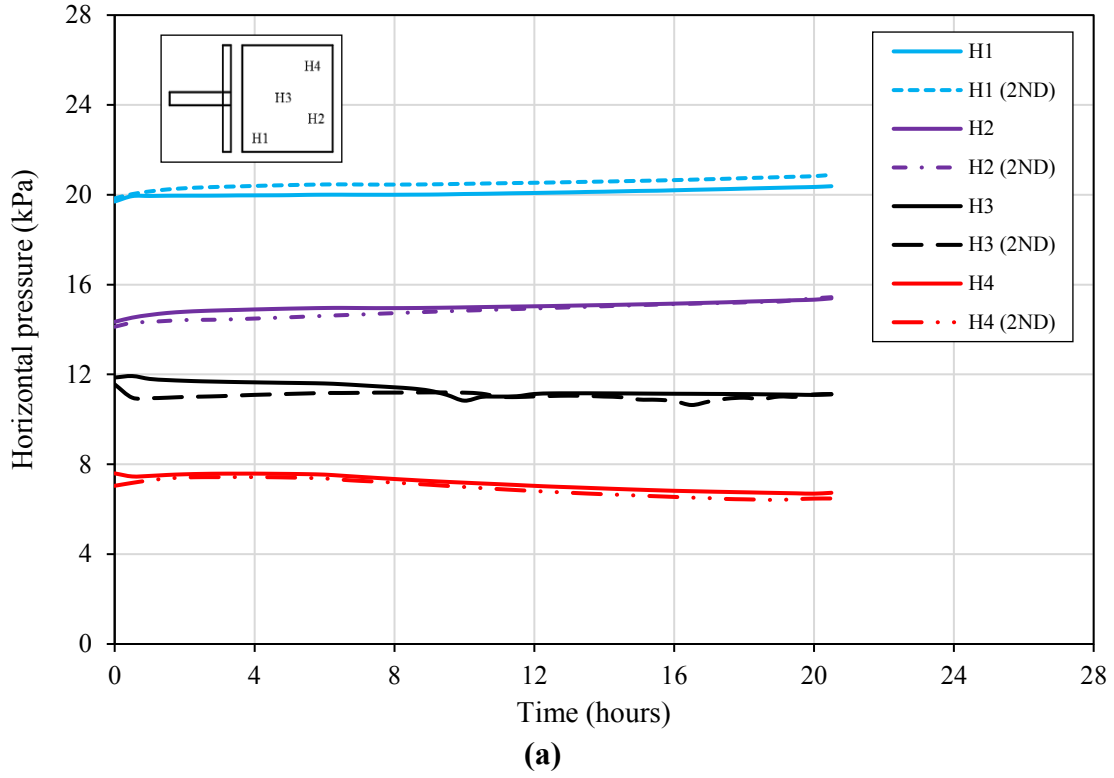
(b)

**Fig. 4.12.** Results from test no. 21 & 24 for soil – B ( $C_p=9.0\%$ ): (a) Horizontal pressure measured by wall transducers versus time; (b) Vertical pressure in the soil mass measured by transducers versus time

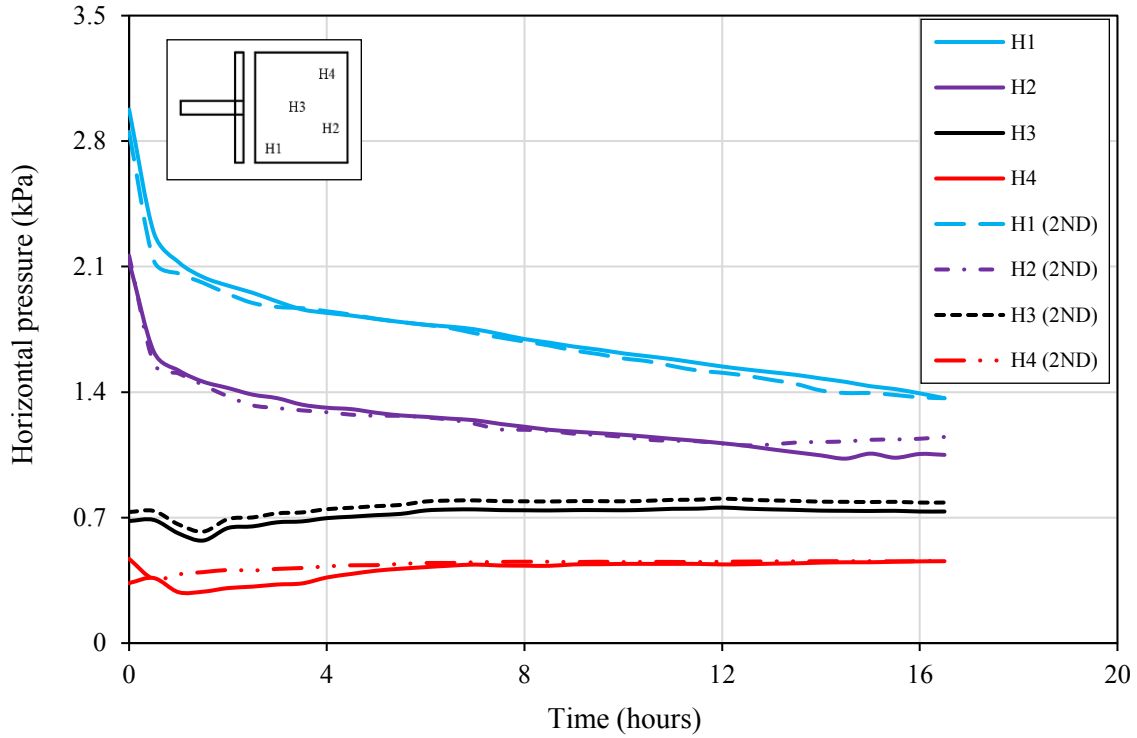
*Results from repeatability tests (for studying the repeatability test results):* These results are presented graphically in Figures 4.13 to 4.14.

It is essential to check the repeatability of the test results by conducting some tests under the same conditions to check the variation in the results between these tests.

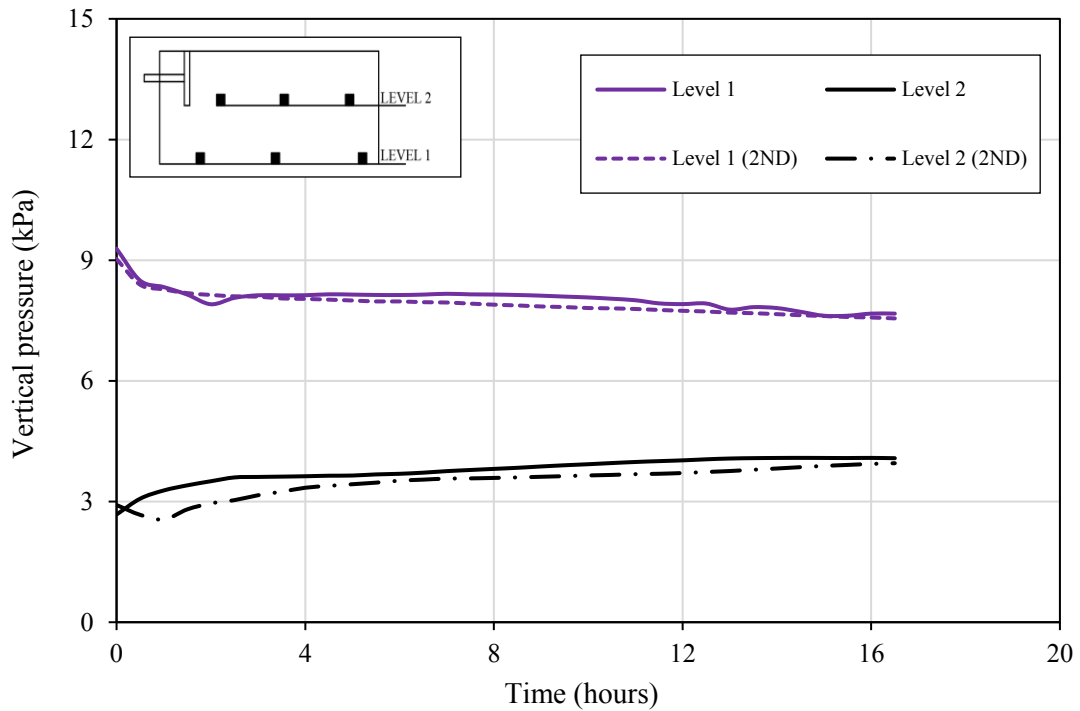
The first set of repeated tests were performed on the dry soil. The second set of the repeated tests were conducted on the saturated soil. The results of two tests (original and repeated) performed under the same conditions with the same soil were compared together. Figure 4.13 and 4.14 presents the original and repeated at-rest earth pressure test results for the dry and saturated soil D ( $C_p=18\%$ ), respectively. It can be generally seen from these figures that the results obtained from two tests performed on the same soil under the same conditions are almost identical.



**Fig. 4.13.** Results from test no. 3 and test no. 3\* (2<sup>ND</sup>) performed on dry soil - D ( $C_p=18\%$ ): (a) Horizontal pressure measured by wall transducers versus time; (b) Vertical pressure in the soil mass measured by transducers versus time



(a)

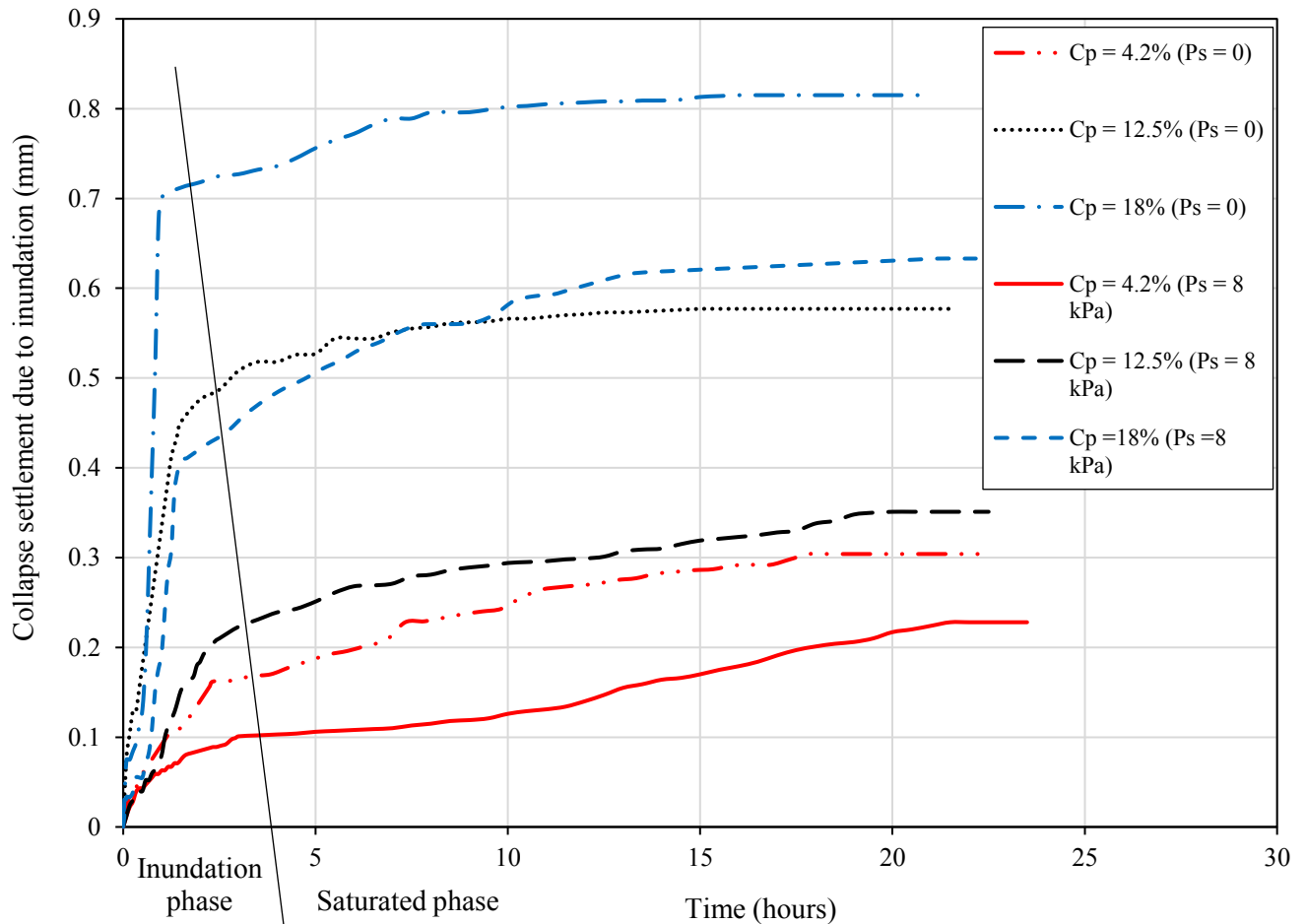


(b)

**Fig. 4.14.** Results from test no.6 and test no. 6\* (2<sup>ND</sup>) performed on the saturated soil - D ( $C_p=18\%$ ): (a) Horizontal pressure measured by wall transducers versus time; (b) Vertical pressure in the soil mass measured by transducers versus time

### *Collapse settlement due to inundation*

In this experimental investigation, the correlation of the collapse settlement due to inundation and time during inundation process is examined and presented graphically in Figure 4.15. It can be noted that the collapse settlement increases during the inundation process and reaches maximum when reaching 100% saturation.



**Fig. 4.15.** Test result: collapse settlement due to inundation versus time

Furthermore, the collapse settlement increases rapidly up to a degree of saturation of about 80% then increases gradually until it reaches 100% saturation. It should be reported herein that for a given value of the OCR, the collapse settlement increases with the increase of the collapse potential  $C_p$ . It is of interest to note that collapse settlement takes place in relatively short period for higher



values of the  $C_p$ , while it takes longer period for higher OCR. It is of interest to report herein that the collapse settlement does not end at 100% saturation but rather continues at a lower rate after, up to 20-40% of the collapse settlement at 100% saturation.

### 4.3. Developing empirical formulae

In this investigation, the overconsolidation ratio OCR values for the case of dry and saturated collapsible soils were determined from Equations (4.1) and (4.2), respectively:

$$OCR_{(dry)} = \frac{\sigma'_v}{\gamma' h} \quad (4.1)$$

$$OCR_{(saturated)} = \frac{\sigma_v - u_w}{\gamma' h} \quad (4.2)$$

The coefficients of at-rest earth pressure  $K_0$  for the dry and saturated soil conditions were determined as follows:

$$K_{0(dry)} = \frac{\sigma'_h}{\sigma'_v} \quad (4.3)$$

$$K_{0(saturated)} = \frac{\sigma'_h}{\sigma'_v} = \frac{\sigma_h - u_w}{\sigma_v - u_w} \quad (4.4)$$

Where

$\sigma_v$  = total vertical pressure at a given level measured by the transducer;

$\sigma_h$  = total horizontal pressure measured on the wall by the respective transducer;

$\sigma'_v = \sigma_v - u_w$  = effective vertical pressure at a given level;

$\sigma'_h = \sigma_h - u_w$  = effective horizontal pressure at a given level;

$u_w = \gamma_w h$  = pore water pressure;

$\gamma' h$  = theoretical effective overburden pressure at that level.

It should be reported herein that for dry soils, the OCR increases gradually due to the increase of the clay content ( $C_p$ ) in the soil. The OCR also increases when increasing the surcharge loading on the soil mass. It can be also seen from Table 4.1 that the OCR decreases considerably due to the saturation of the soil. Upon saturated, the soil with greater  $C_p$  experiences larger decrease in the OCR.

**Table 4.1.** Summary of all test results for at-rest earth pressure

Test No.	Soil type	$C_p$ (%)	Surcharge $P_s$ (kPa)	Condition	OCR	$K_0$
1	A	4.2	0	Dry	3.3	0.613
2	C	12.5	0	Dry	5.5	0.864
3	D	18	0	Dry	7.2	1.049
4	A	4.2	0	Saturated	2.8	0.602
5	C	12.5	0	Saturated	4.7	0.615
6	D	18	0	Saturated	1.1	0.215
7	A	4.2	8	Dry	4.5	0.709
8	C	12.5	8	Dry	6.9	0.979
9	D	18	8	Dry	9.9	1.275
10	A	4.2	8	Saturated	3.7	0.689
11	C	12.5	8	Saturated	5.9	0.732
12	D	18	8	Saturated	1.8	0.275
13	A	4.2	16	Dry	6.0	0.831
14	C	12.5	16	Dry	8.2	1.082
15	D	18	16	Dry	11.7	1.449
16	A	4.2	16	Saturated	4.9	0.802
17	C	12.5	16	Saturated	7.0	0.812
18	D	18	16	Saturated	2.7	0.340
19	B	9	0	Dry	3.9	0.722
20	B	9	8	Dry	5.1	0.821
21	B	9	16	Dry	6.6	0.923
22	B	9	0	Saturated	3.3	0.605
23	B	9	8	Saturated	4.1	0.696
24	B	9	16	Saturated	5.7	0.811

Furthermore, it can be noted from this table that the coefficient of at-rest earth pressure of dry soils increases moderately by increasing the  $C_p$ , which can be modelled in a linear relation, while it also increases due to the increase of stress level (OCR) in the soil. It can be noted that the coefficient of at-rest earth pressure dropped considerably when the soil was fully inundated.

This can be explained that collapsible soil due to its unstable soil fabric, the bond strength between particles is easy to be weakened and/or broken due to the presence of water, causing collapse, which is associated with loss of strength and significant settlement. It is of interest to note that the at-rest earth pressure for saturated soil decreases with the increase of the collapse potential  $C_p$  of the soil and increases with the increase of the “OCR”.

#### **4.3.1. Proposed formula for estimating coefficient of at-rest earth pressure $K_0$ for collapsible soils at dry state**

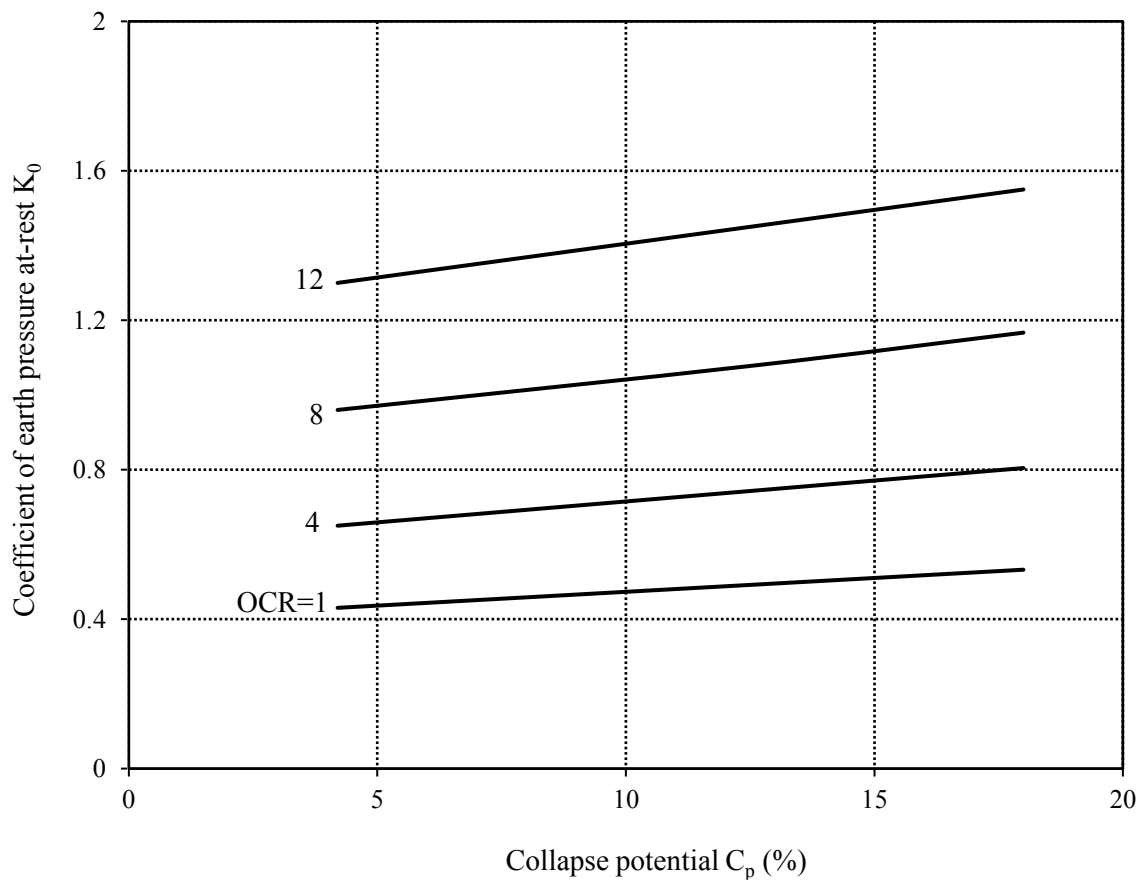
Test results of the present experimental investigation were used to develop the following empirical formula, which predict the coefficient of earth pressure at-rest for given OCR and  $C_p$  values for collapsible soils in the dry state:

$$K_0 = (0.007C_p + 0.4)(0.18OCR + 0.8) \quad (4.5)$$

Table 4.2 presents the values of the coefficient of at-rest earth pressure  $K_0$  estimated by Eq. (4.5) for dry soils for given OCR and  $C_p$  values, together with the theoretical values of Meyerhof (1976), Massarsch (1979), Mayne & Kulhawy (1982), and Hanna and Al-Romhein (2008), where good agreement can be noted.

Furthermore, the values predicted by Eq. (4.5) for normally consolidated soils were also compared with the values given by Jaky (1944). These values are given in Table 4.3, and reasonable agreement can be seen.

For convenience, the theoretical values evaluated using Eq. (4.5) were presented in graphical form in Figure 4.16.



**Fig. 4.16.** The theoretical values of  $K_0$  obtained from the proposed formula [Eq. (4.5)] for the dry state

**Table 4.2.** Comparison between Experimental and Theoretical values of  $K_0$  for overconsolidated collapsible soil at dry state

Experimental data							Theoretical results $K_0$				
Test no.	Soil properties				OCR	$K_0$	Meyerhof 1976	Massarch 1979	Mayne & Kulhawy 1982	Hanna & Al-Romhein 2008	Proposed formula [Eq.(4.5)]
	Soil type	$C_p$ (%)	$\phi'$ (degree)	PI (%)							
1	A	4.2	40	-	3.3	0.613	0.649	-	0.768	0.621	0.600
2	C	12.5	35	2.55	5.5	0.864	0.999	1.057	1.134	0.834	0.878
3	D	18	31	7.4	7.2	1.049	1.301	1.264	1.340	0.940	1.112
7	A	4.2	40	-	4.5	0.709	0.757	-	0.939	0.717	0.700
8	C	12.5	35	2.55	6.9	0.979	1.119	1.184	1.291	0.912	1.001
9	D	18	31	7.4	9.9	1.275	1.526	1.482	1.579	1.045	1.368
13	A	4.2	40	-	6.0	0.831	0.875	-	1.130	0.819	0.810
14	C	12.5	35	2.55	8.2	1.082	1.221	1.291	1.426	0.976	1.114
15	D	18	31	7.4	11.7	1.449	1.659	1.611	1.721	1.206	1.538
19	B	9	38.5	-	3.9	0.722	0.745	-	0.881	0.689	0.700
20	B	9	38.5	-	5.1	0.821	0.852	-	1.041	0.776	0.800
21	B	9	38.5	-	6.6	0.923	0.970	-	1.222	0.870	0.923

**Table 4.3.** Comparison between the predicted values of  $K_0$  obtained from Eq. (4.5) and the values given by Jaky (1944) for the case of normally consolidated soils at dry state

Soil properties			Coefficient of at-rest earth pressure, $K_0$	
$C_p$ (%)	$\phi'$ (degree)	OCR	Jaky (1944)	Proposed formula [Eq. (4.5)]
4.2	40	1	0.357	0.430
9.0	38.5	1	0.377	0.460
12.5	35	1	0.426	0.492
18	31	1	0.485	0.530

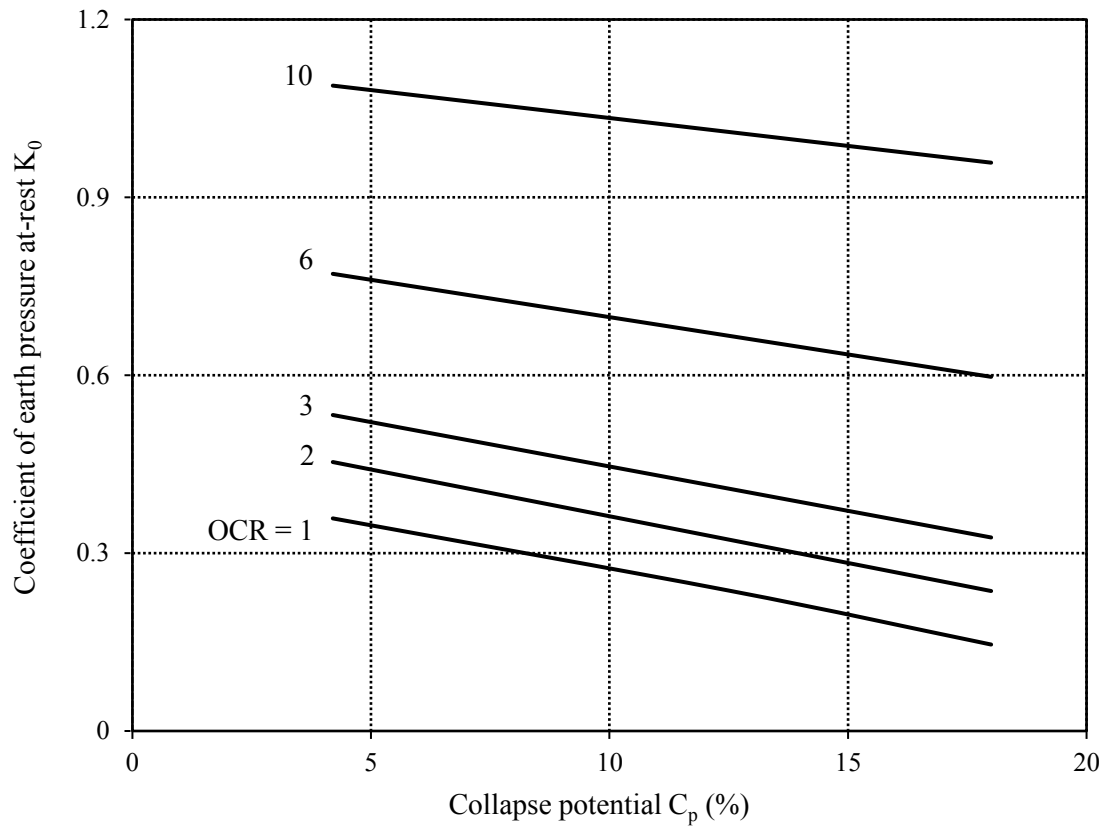
#### 4.3.2. Proposed formula for estimating coefficient of at-rest earth pressure $K_0$ for collapsible soils for the case after full inundation

Test results of the present experimental investigation after full inundation were used to develop the following empirical formula, which predict the coefficient of earth pressure at-rest for given OCR and  $C_p$  values for collapsible soils in the saturated state:

$$K_0 = (0.41 - 0.014C_p) (0.39OCR + 0.64) \quad (4.6)$$

For practical use, the theoretical values evaluated using Eq. (4.6) were plotted in Figure 4.17.

Considering that no experimental results available in the literature to validate the results predicted by the proposed Eq. (4.6) for the case of saturated collapsible soil, additional tests were conducted only for the purpose to validate the proposed empirical formula. The results of these comparisons are presented in Tables 4.4 and 4.5, where good agreement can be noted.



**Fig. 4.17.** The theoretical values of  $K_0$  obtained from the proposed formula [Eq. (4.6)] for the case after full inundation

**Table 4.4.** Comparison between Experimental and Theoretical values of  $K_0$  for the overconsolidated collapsible soil after full inundation

Test No.	Experimental results				Theoretical results of $K_0$
	Soil type	$C_p$ (%)	OCR	$K_0$	Proposed formula [Eq.(4.6)]
4	A	4.2	2.8	0.602	0.608
5	C	12.5	4.7	0.615	0.581
6	D	18	1.1	0.215	0.170
10	A	4.2	3.7	0.689	0.730
11	C	12.5	5.9	0.732	0.693
12	D	18	1.8	0.275	0.212

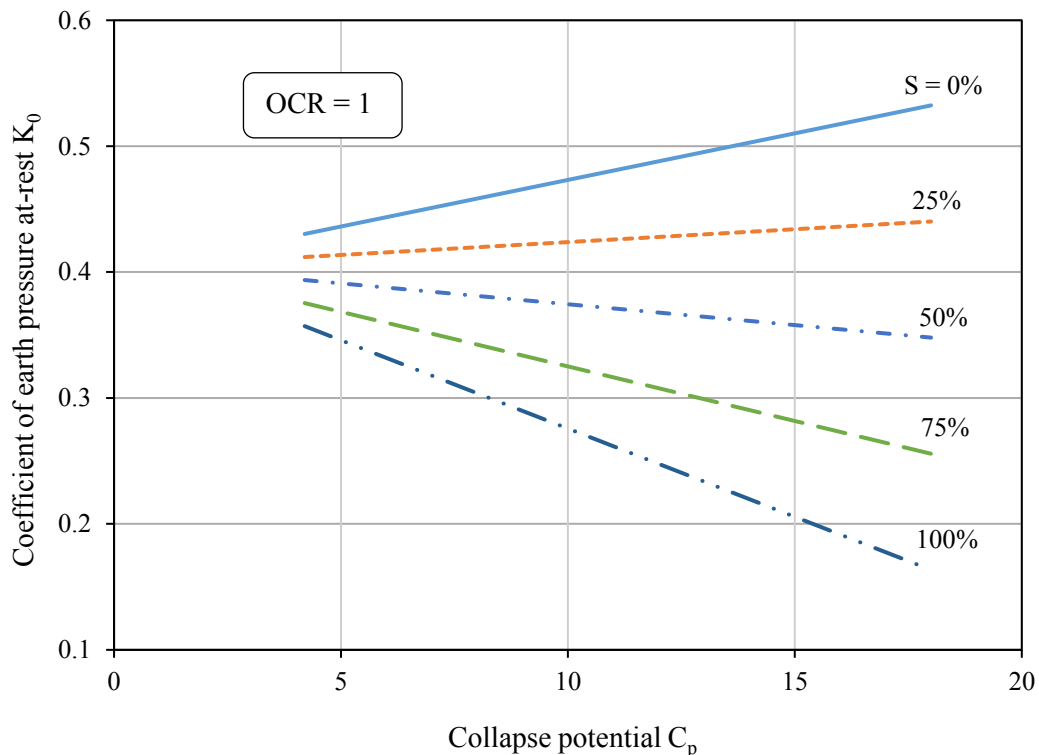
**Table 4.5.** Comparison between Theoretical and Experimental results from the additional Tests of  $K_0$  for the overconsolidated collapsible soil after full inundation.

Test No.	Experimental results				Theoretical results of $K_0$
	Soil type	$C_p$ (%)	OCR	$K_0$	Proposed formula [Eq. (4.6)]
16	A	4.2	4.9	0.802	0.890
17	C	12.5	7.0	0.812	0.792
18	D	18	2.7	0.340	0.277
22	B	9	3.3	0.605	0.560
23	B	9	4.1	0.696	0.660
24	B	9	5.7	0.811	0.813

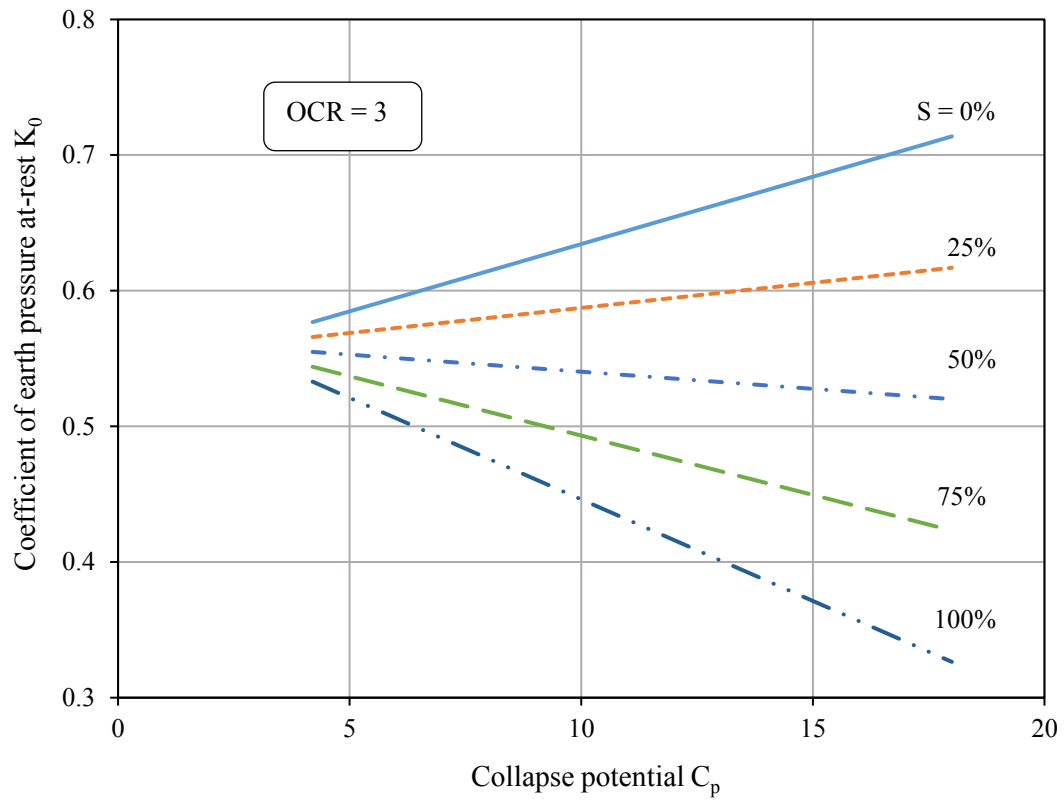


### 4.3.3. Proposed design charts for predicting coefficient of at-rest earth pressure $K_0$ for unsaturated collapsible soils at different degree of saturation

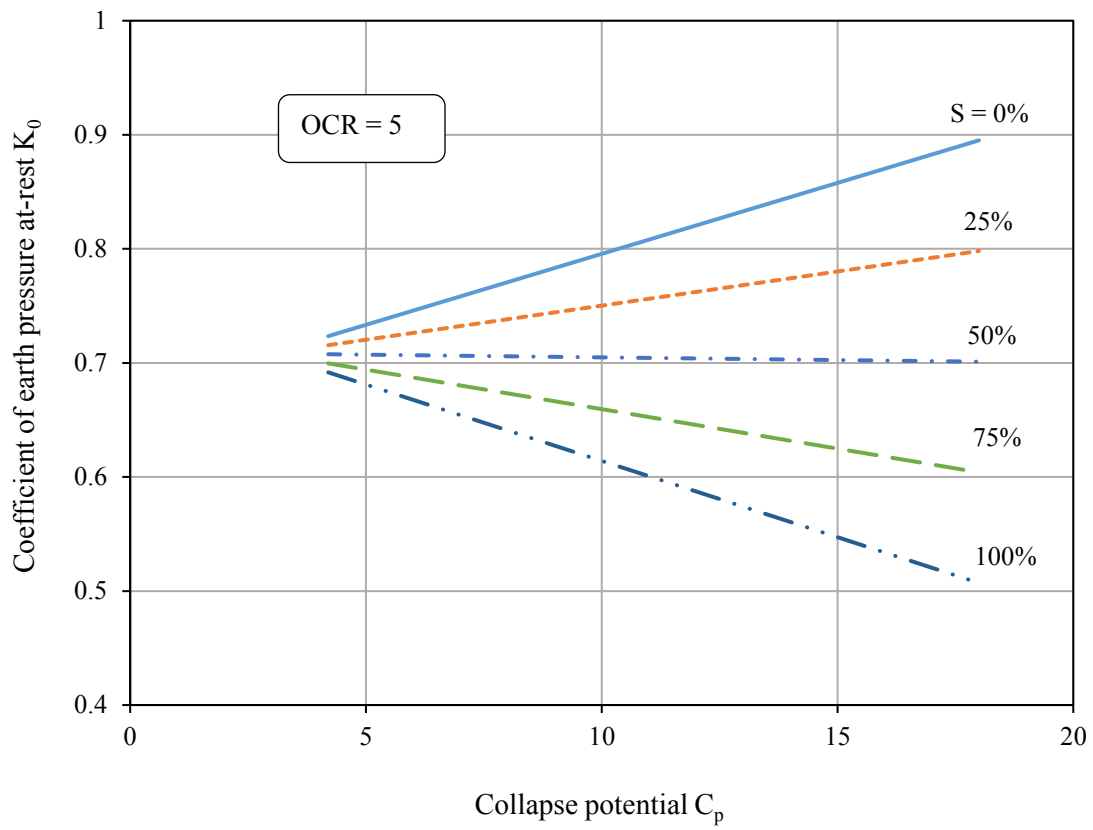
After validating the proposed empirical formulae given in Eqs. (4.5) and (4.6) for the dry and saturated states, respectively, these formulae were used to develop design charts given in Figure 4.18 to predict the values of coefficient of at-rest earth pressure of overconsolidated collapsible soil  $K_0$  as the function of the collapse potential ( $C_p$ ), the overconsolidation ratio (OCR) and the degree of saturation (S) of the soil. In these charts, it was assumed that the coefficient of at-rest earth pressure  $K_0$  decreases linearly with the increase of the degree of saturation (S).



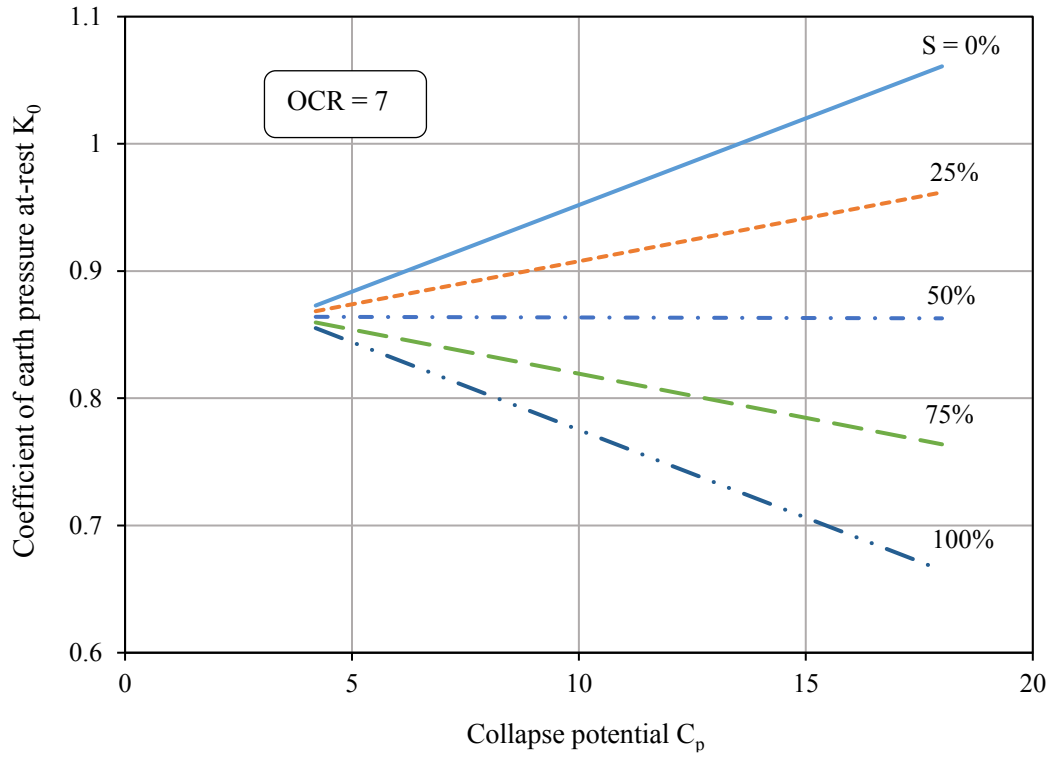
(a)



(b)



(c)



(d)

**Fig. 4.18.** Design charts: The theoretical values of  $K_0$  for unsaturated collapsible soils: (a) OCR = 1; (b) OCR = 3; (c) OCR = 5; (d) OCR = 7.

## **CHAPTER 5**

### **TEST RESULTS AND ANALYSIS FOR THE CASE OF PASSIVE PRESSURE**

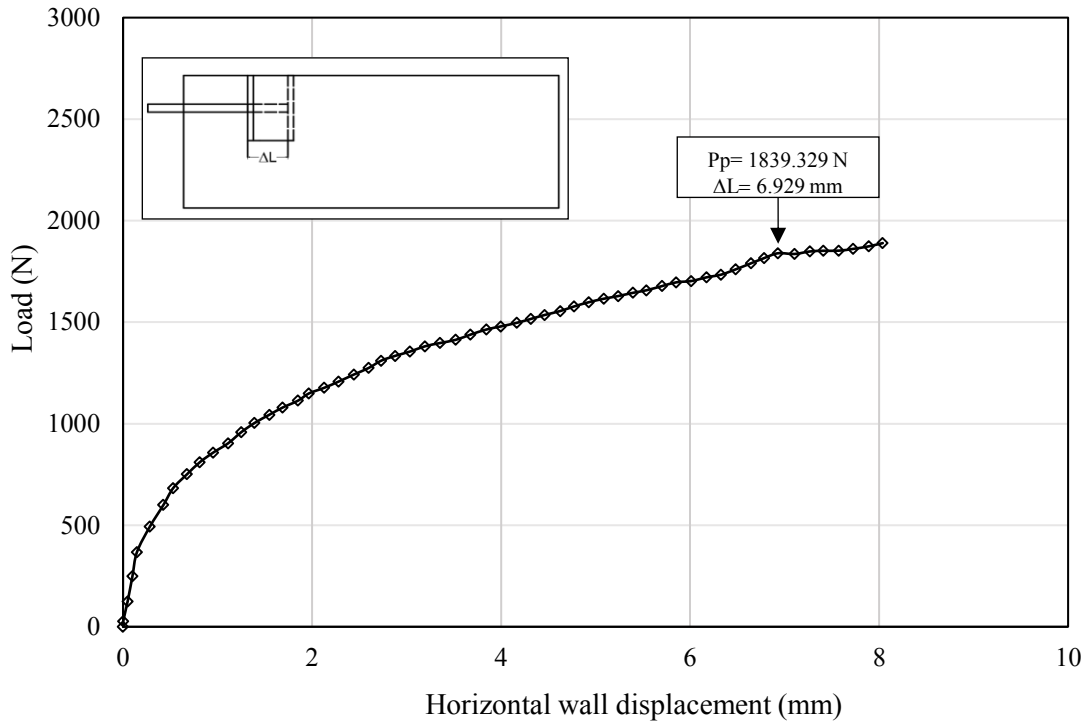
#### **5.1. General**

In this chapter, the results obtained from the present experimental investigation are presented in form of figures. A series of comparing figures have been developed to indicate the difference of soil behavior under the same or different conditions. An analytical model was developed to predict the passive earth force  $P_p$  and the coefficient of passive earth pressure  $K_p$  of collapsible soils. The analytical model developed in this investigation was used to develop results for a wide range of parameters. These results are presented as design chart, which will assist designers to estimate the coefficient of passive earth pressure  $K_p$  for given OCR and  $C_p$  values for dry collapsible soils. Moreover, the reduction factor was introduced to take into account the effect of the soil collapse on the coefficient of passive earth pressure  $K_p$  for the case of inundated collapsible soils.

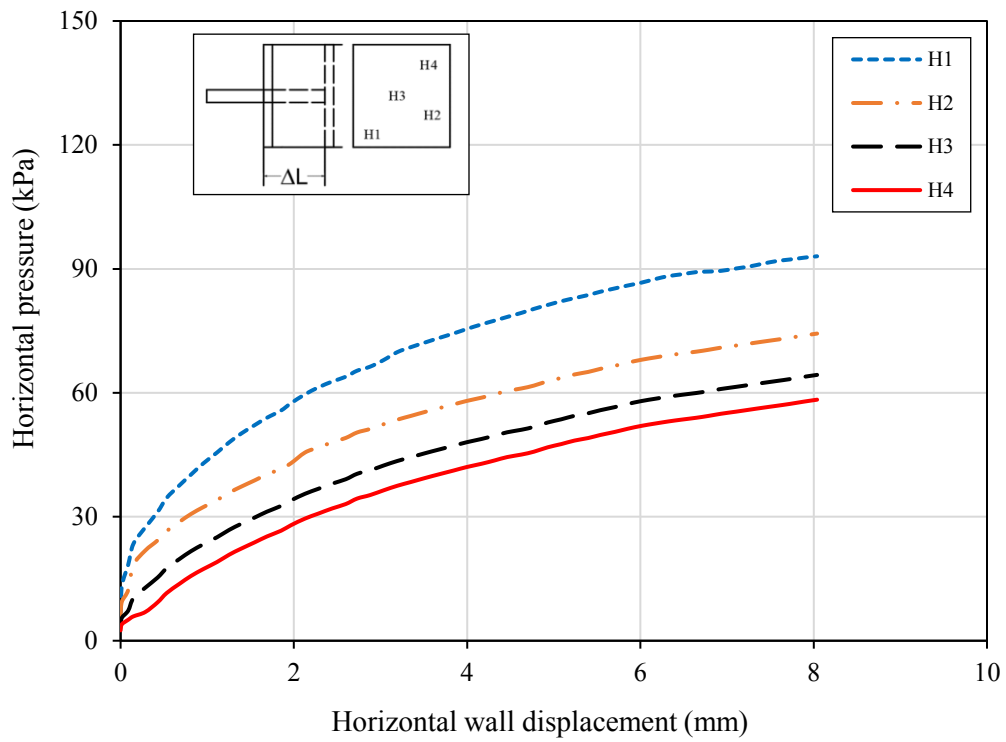
#### **5.2. Results and analysis**

The experimental results are presented in graphical form in Figures 5.1 to 5.24. It can be noted that the passive earth force ( $P_p$ ) and the horizontal wall displacement ( $\Delta L$ ) recorded at the failure point of the soil are also highlighted and indicated.

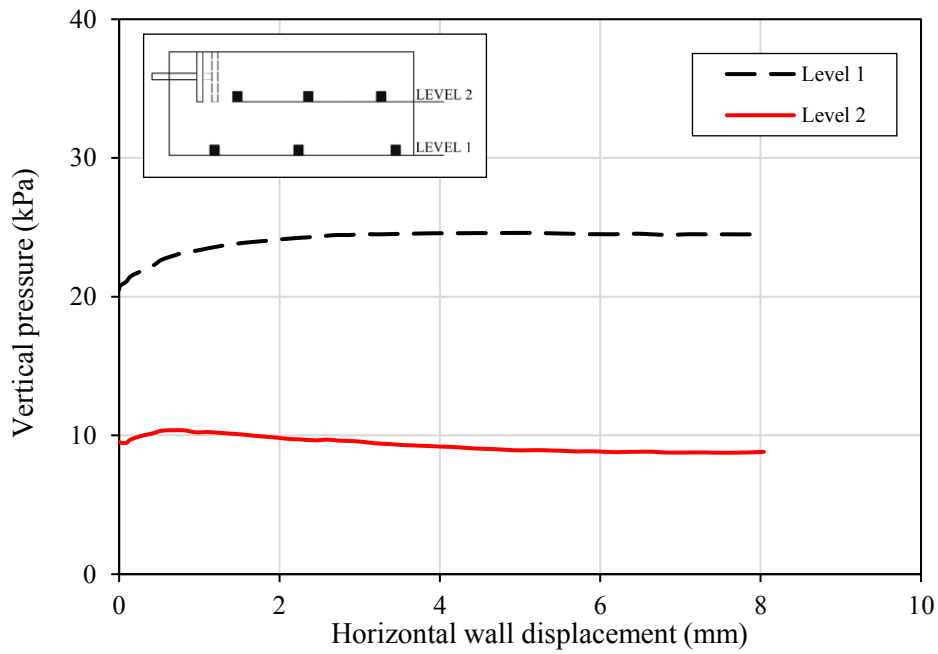
### Results for test no. 1



(a)



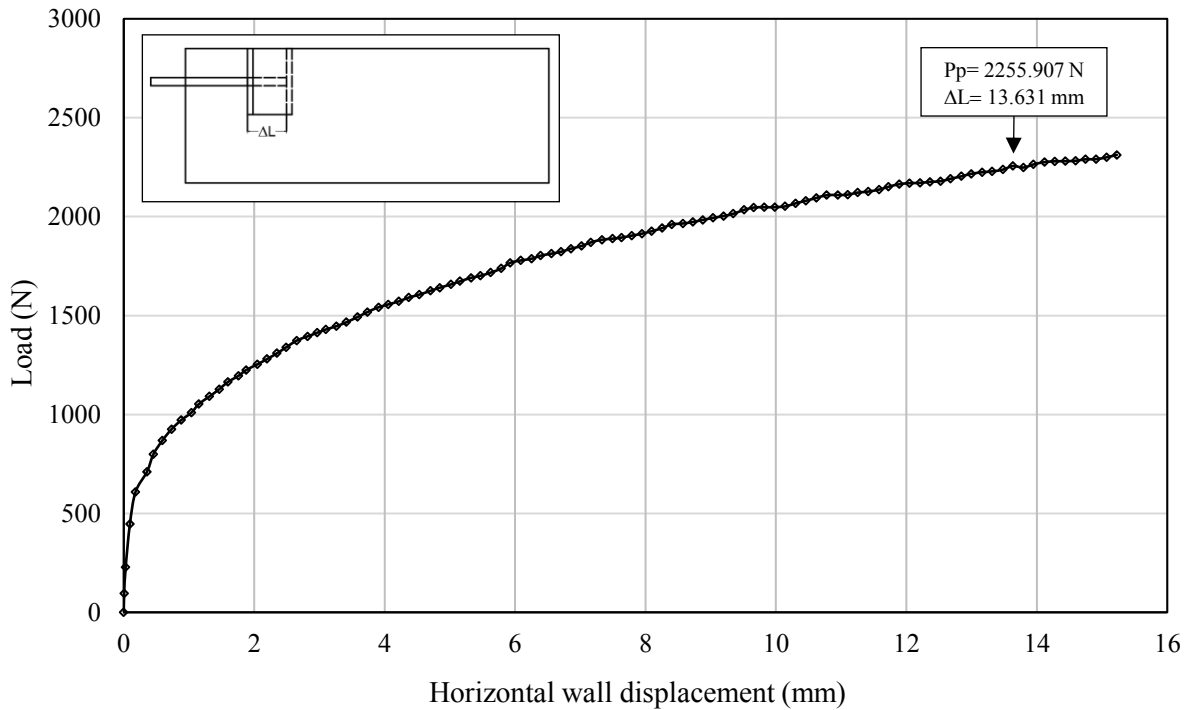
(b)



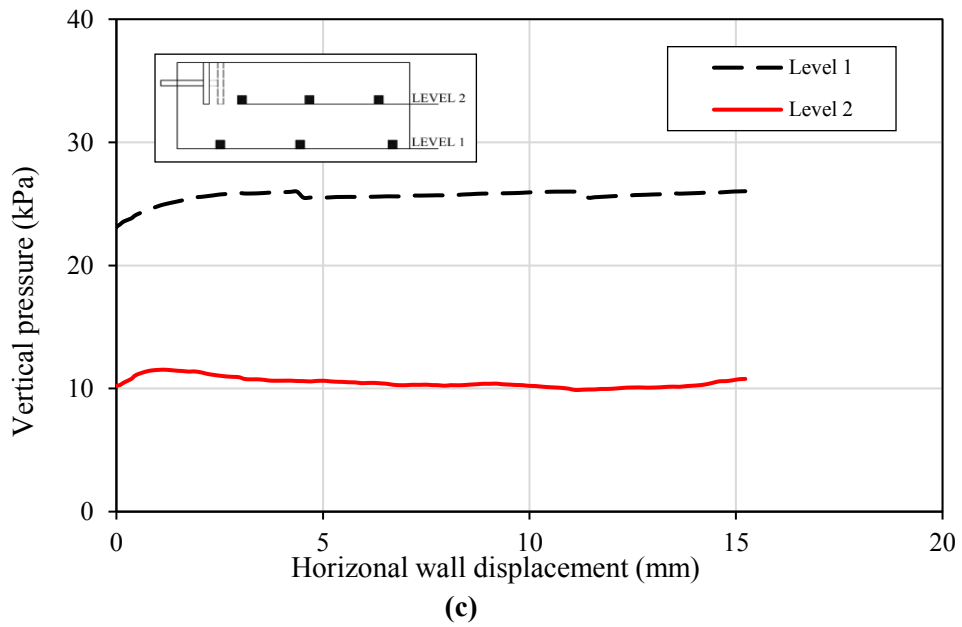
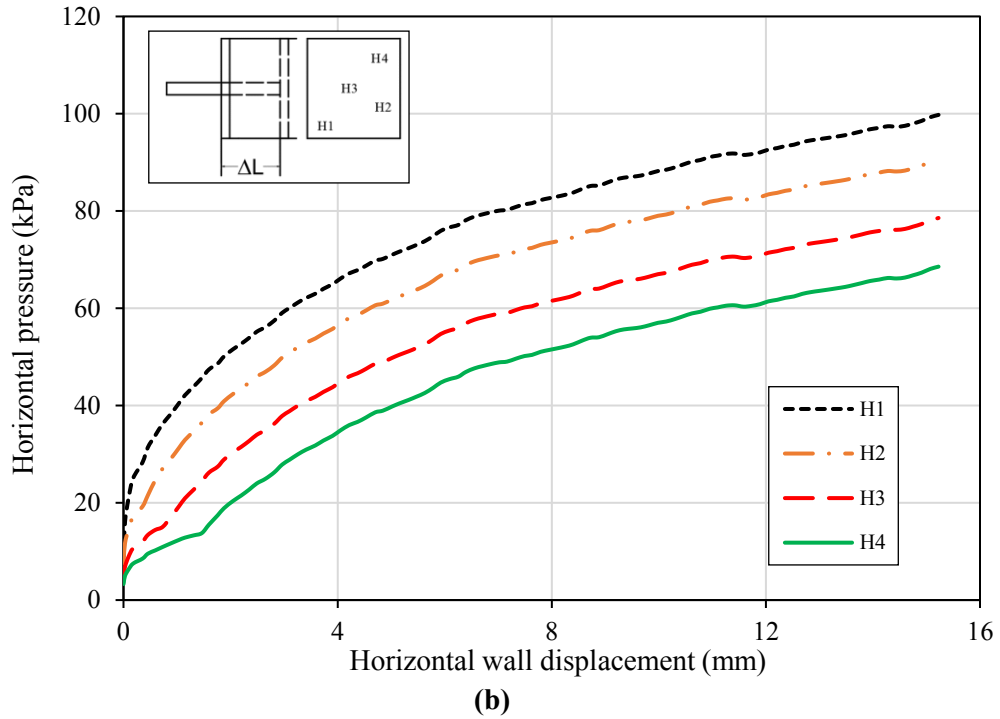
(c)

**Fig. 5.1.** Results for test no.1 (Se1) for soil A –  $C_p = 4.2\%$ : (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the soil mass versus the wall displacement

**Results for test no. 2**



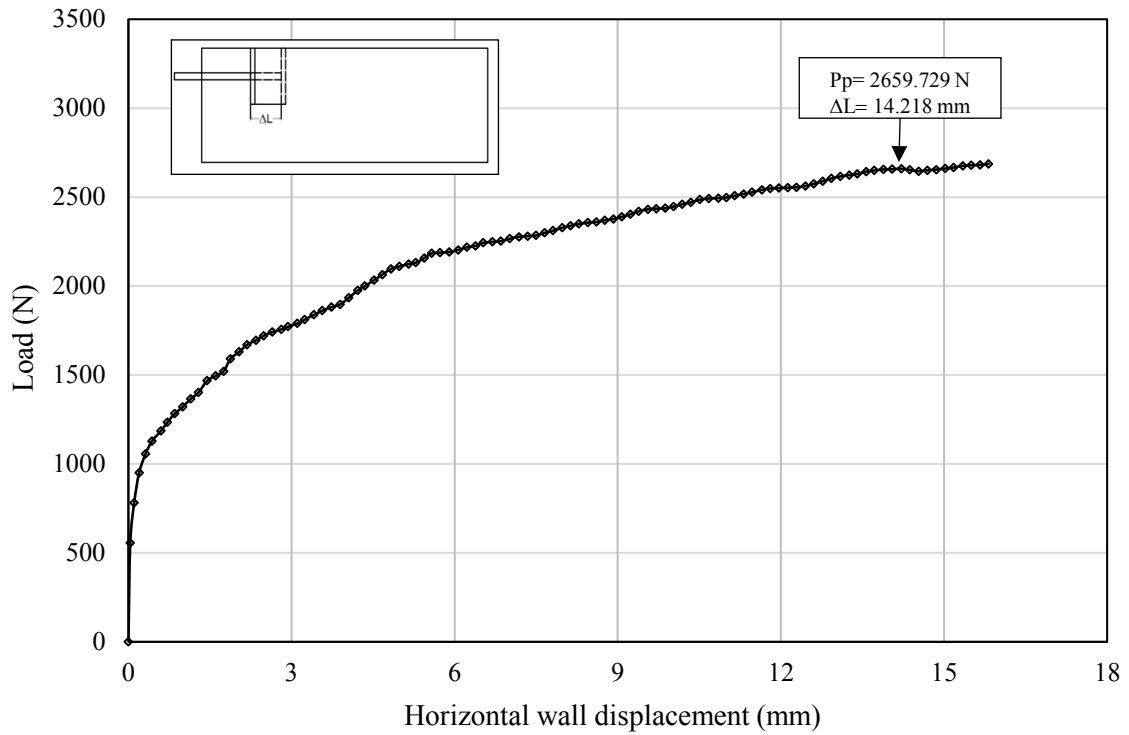
(a)



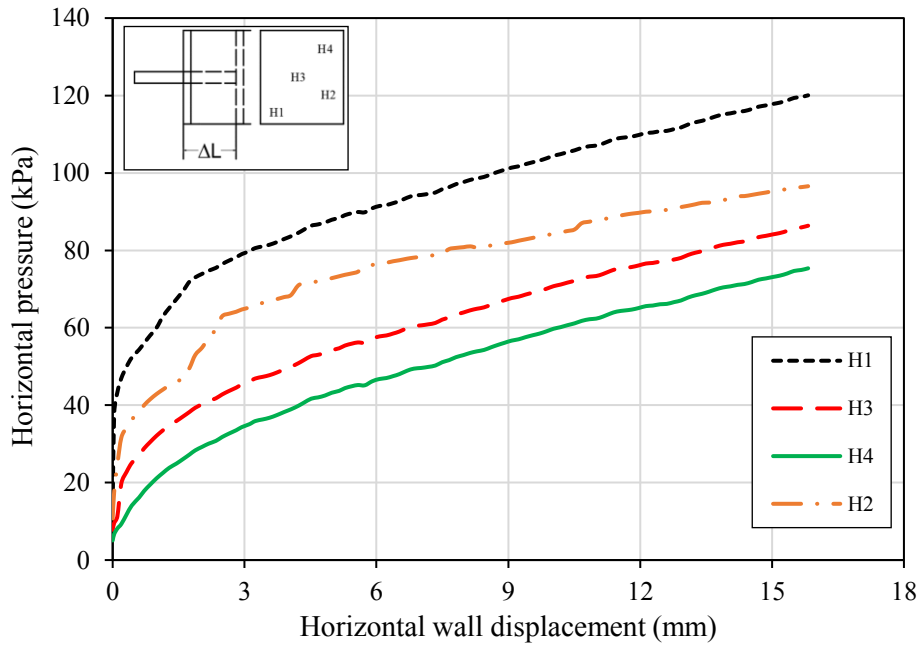
**Fig. 5.2.** Results for test no.2 (Se1) for soil B –  $C_p = 9.0\%$ : (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the soil mass versus the wall displacement



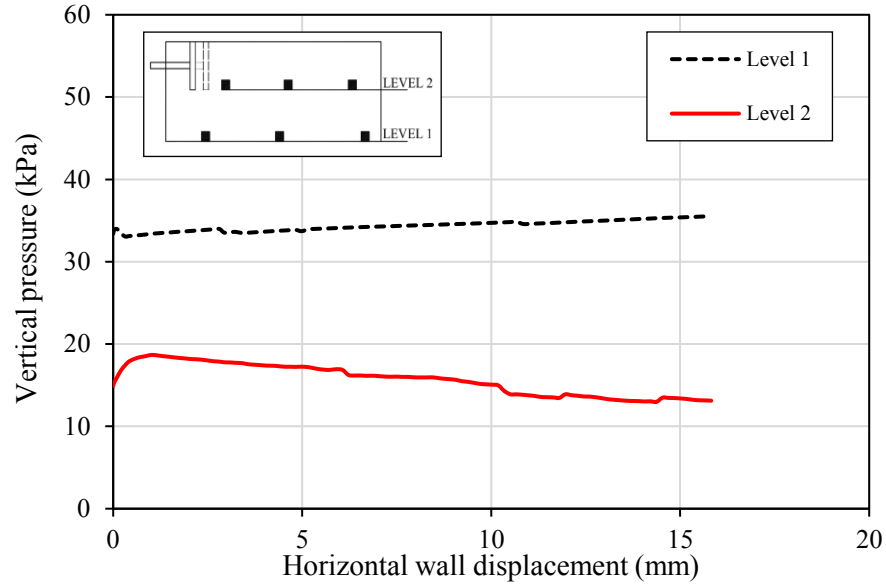
### Results for test no. 3



(a)



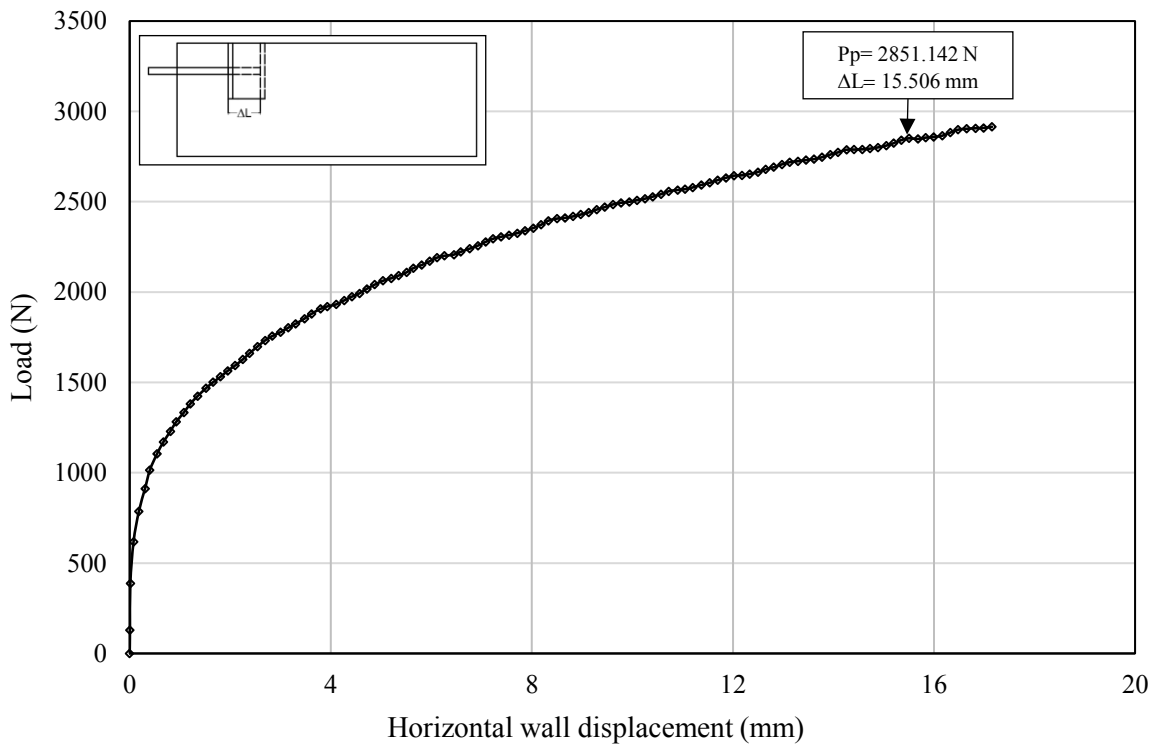
(b)



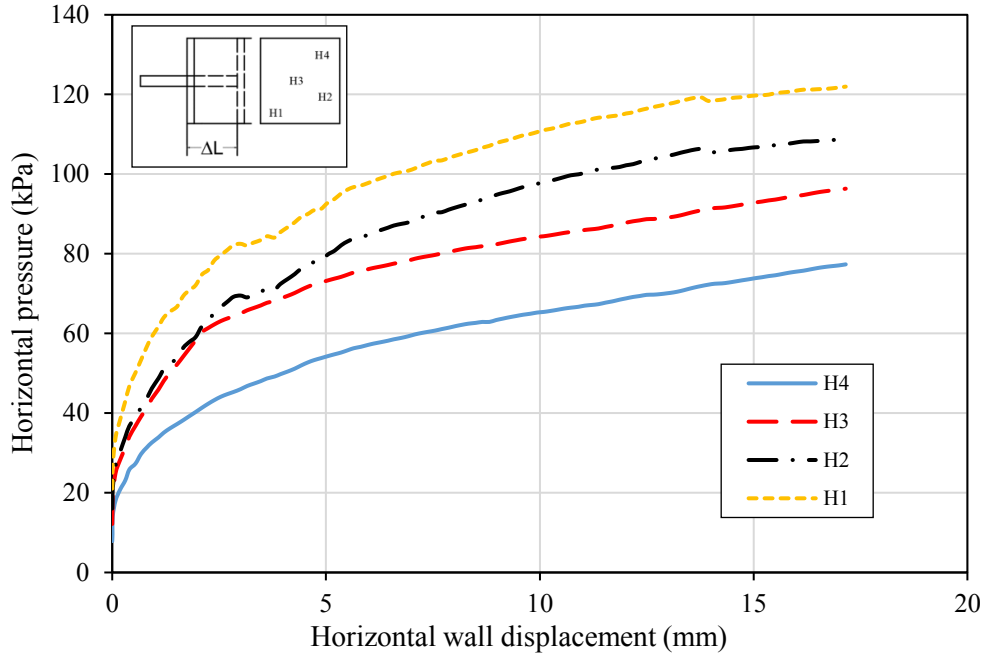
(c)

**Fig. 5.3.** Results for test no. 3 (Se1) for soil C –  $C_p = 12.5\%$ : (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the soil mass versus the wall displacement

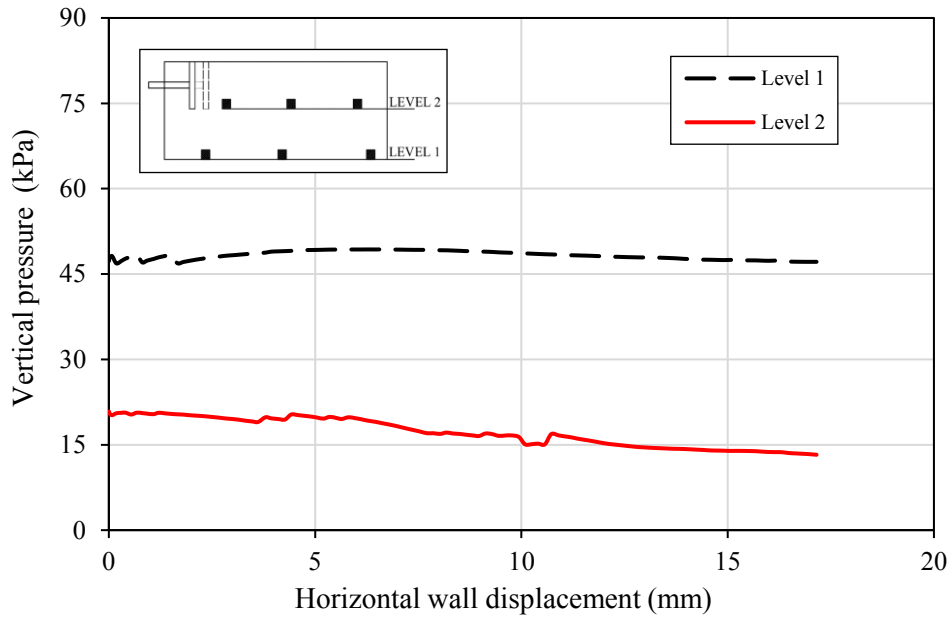
**Results for test no. 4**



(a)



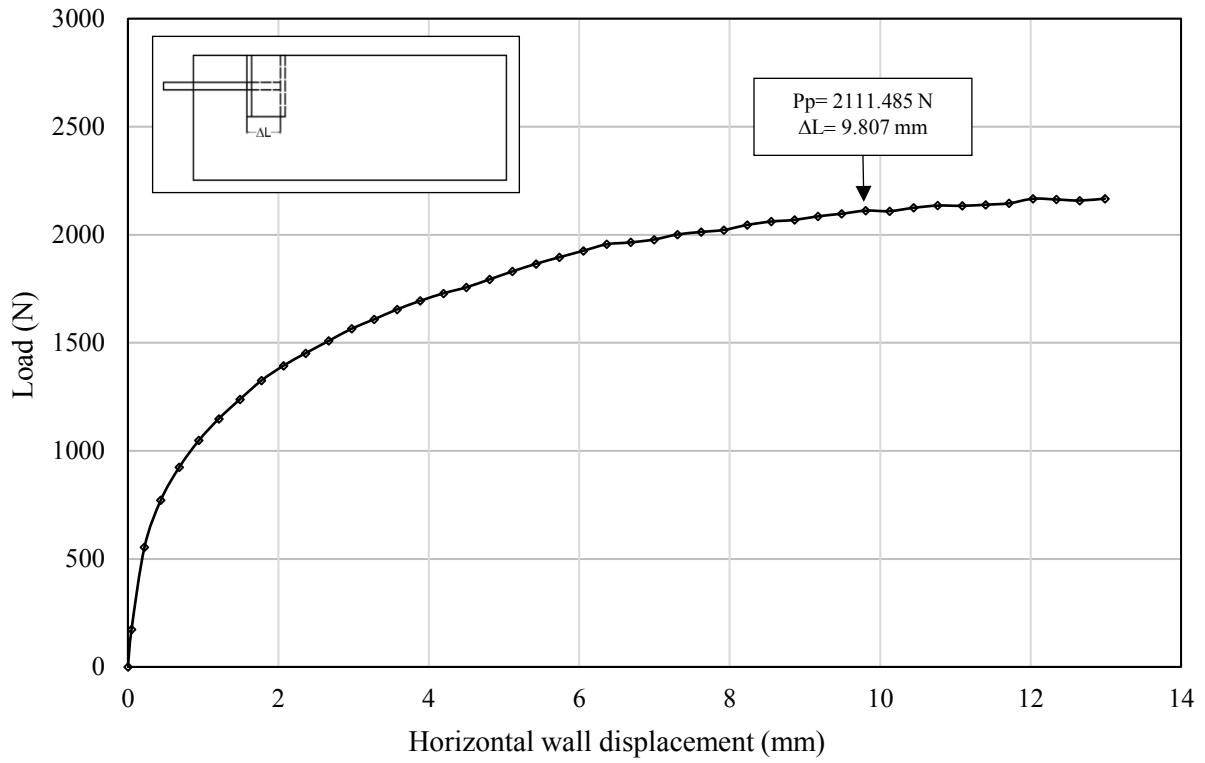
(b)



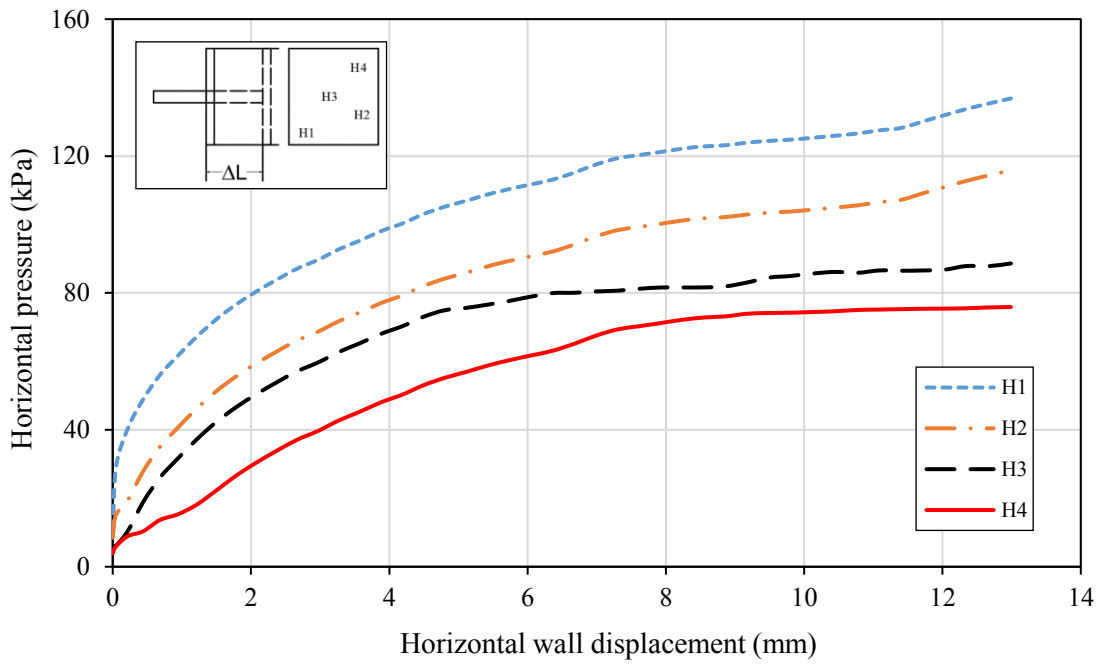
(c)

**Fig. 5.4.** Results for test no. 4 (Se1) for soil D –  $C_p = 18\%$ : (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the soil mass versus the wall displacement

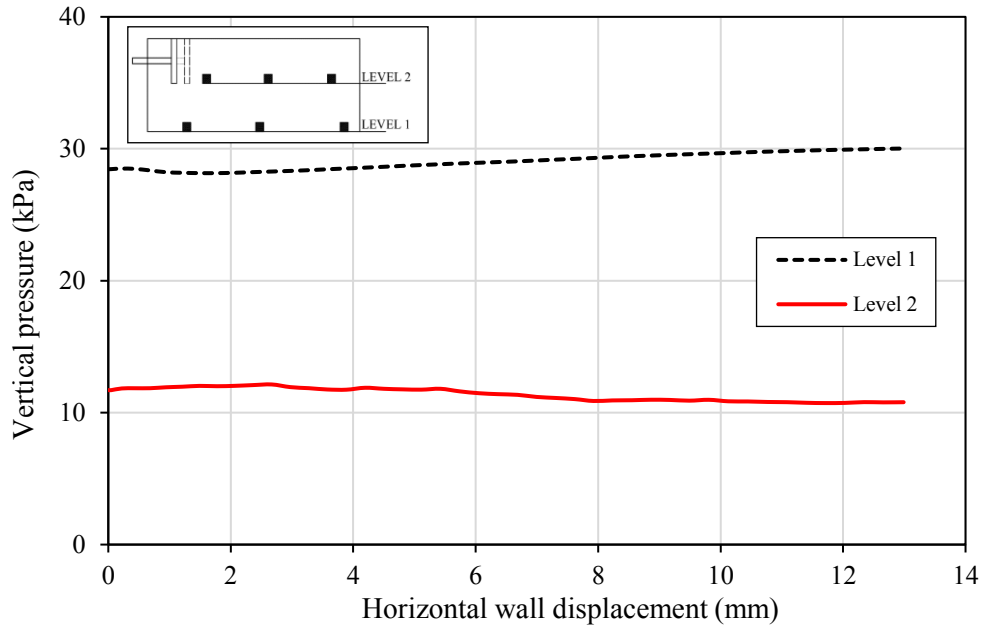
### Results for test no. 5



(a)



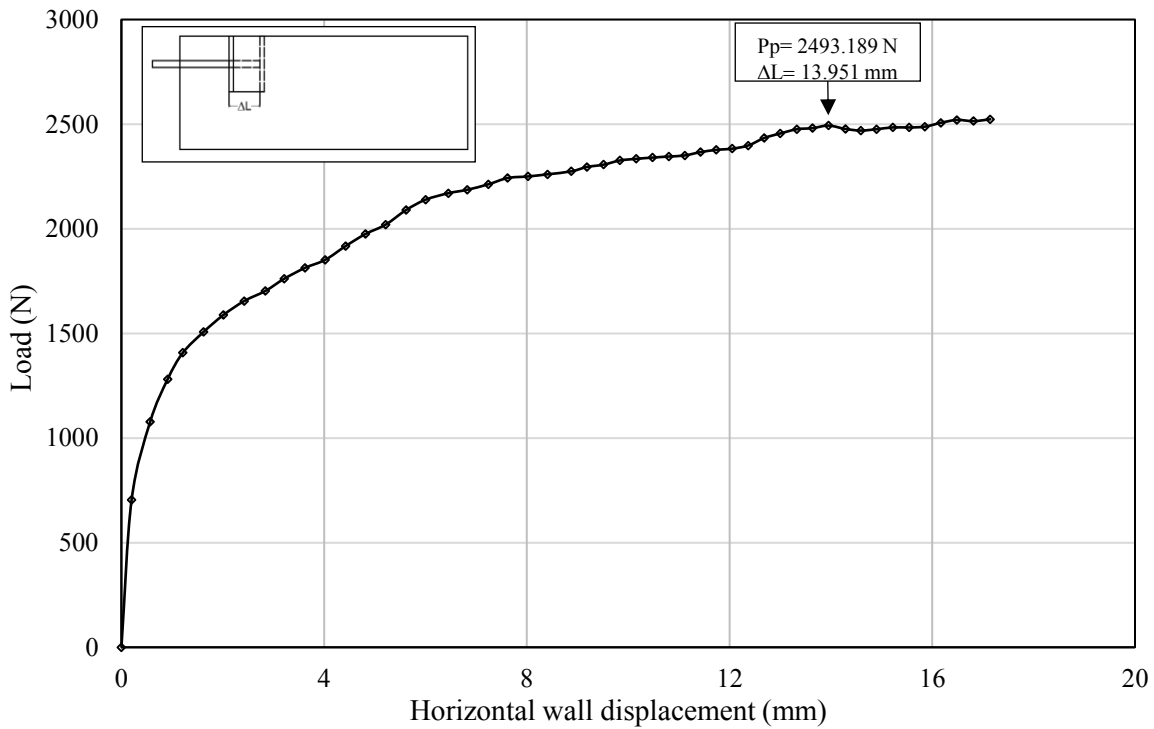
(b)



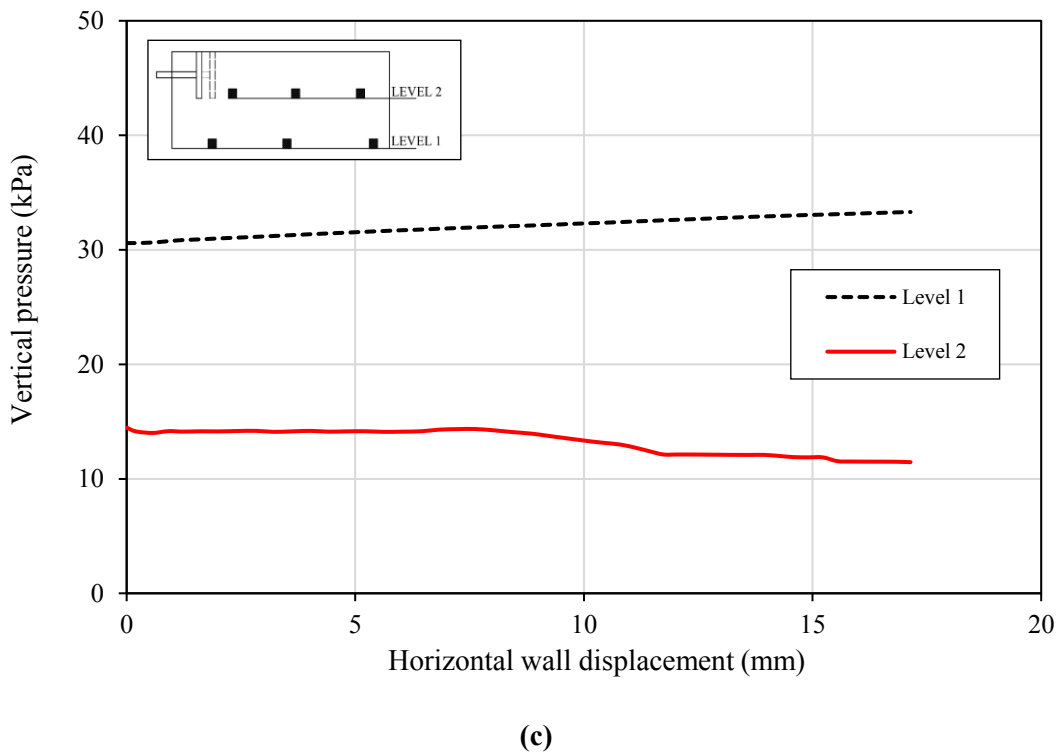
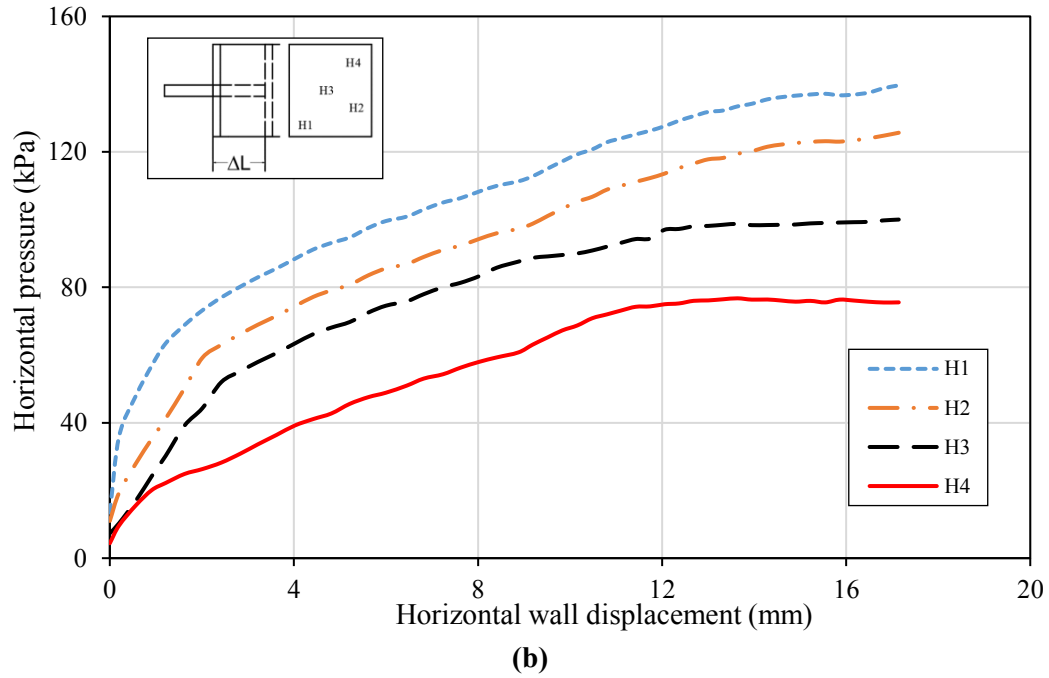
(c)

**Fig. 5.5.** Results for test no. 5 (Se2) for soil A –  $C_p = 4.2\%$ : (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the soil mass versus the wall displacement

**Results for test no. 6**

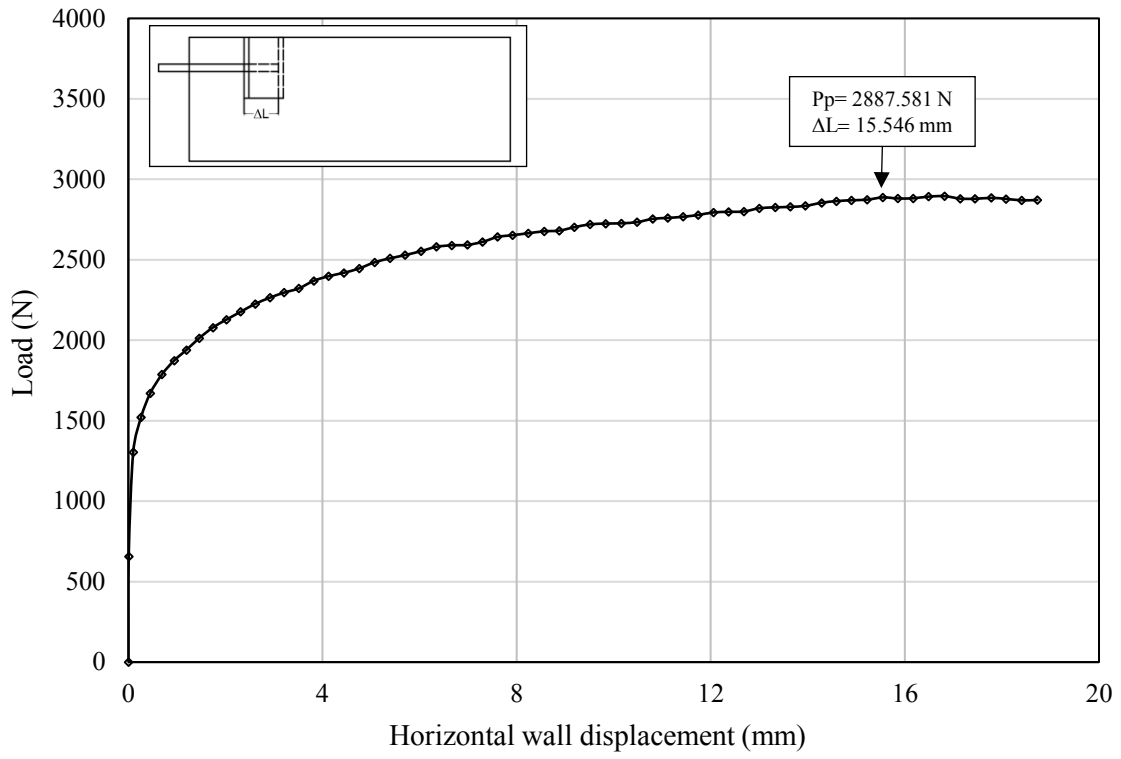


(a)

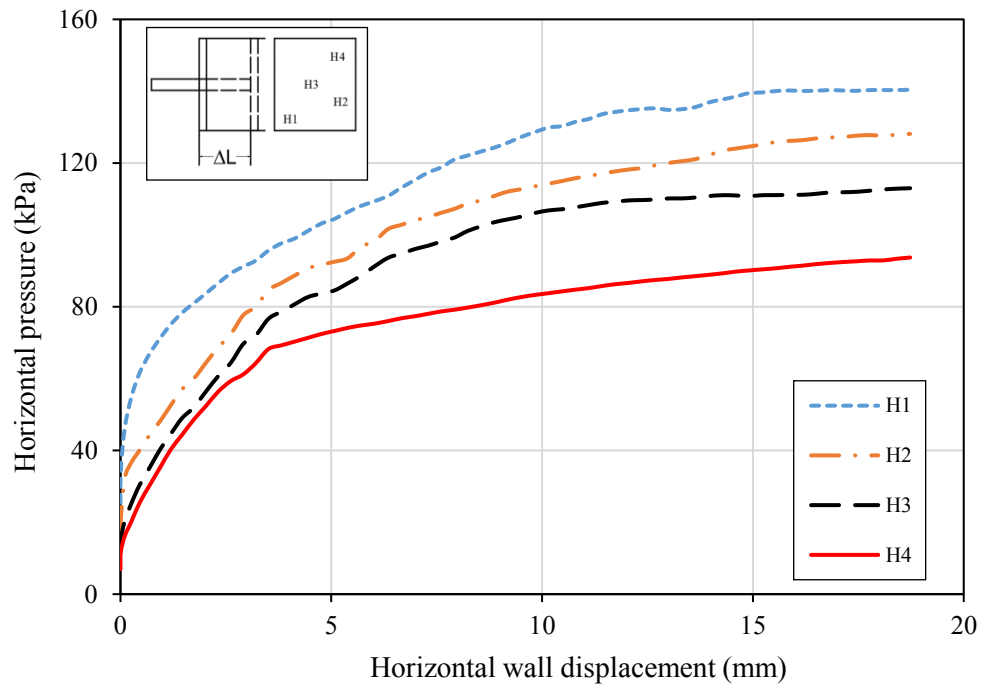


**Fig. 5.6.** Results for test no. 6 (Se2) for soil B –  $C_p = 9.0\%$ : (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the soil mass versus the wall displacement

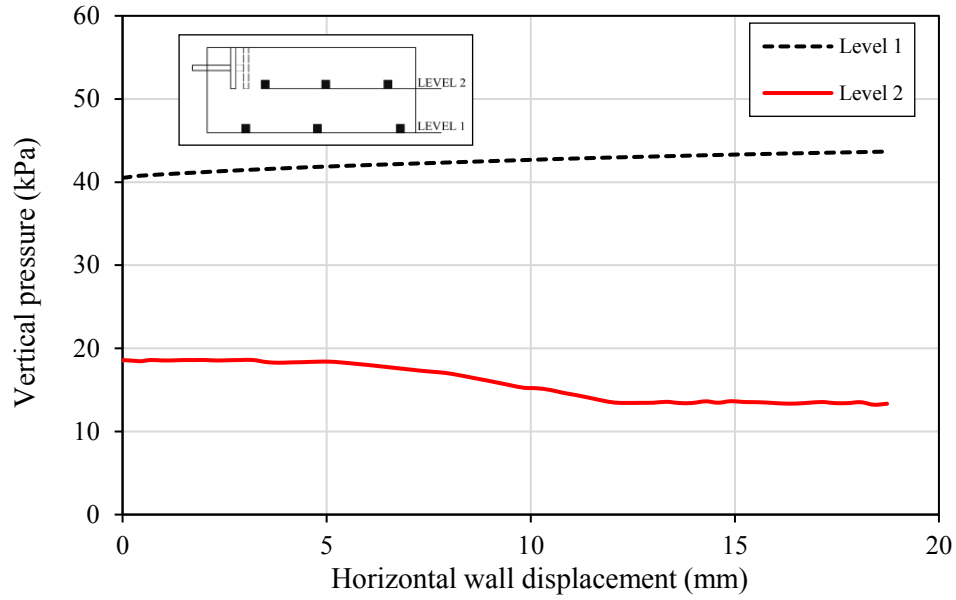
**Results for test no. 7**



**(a)**



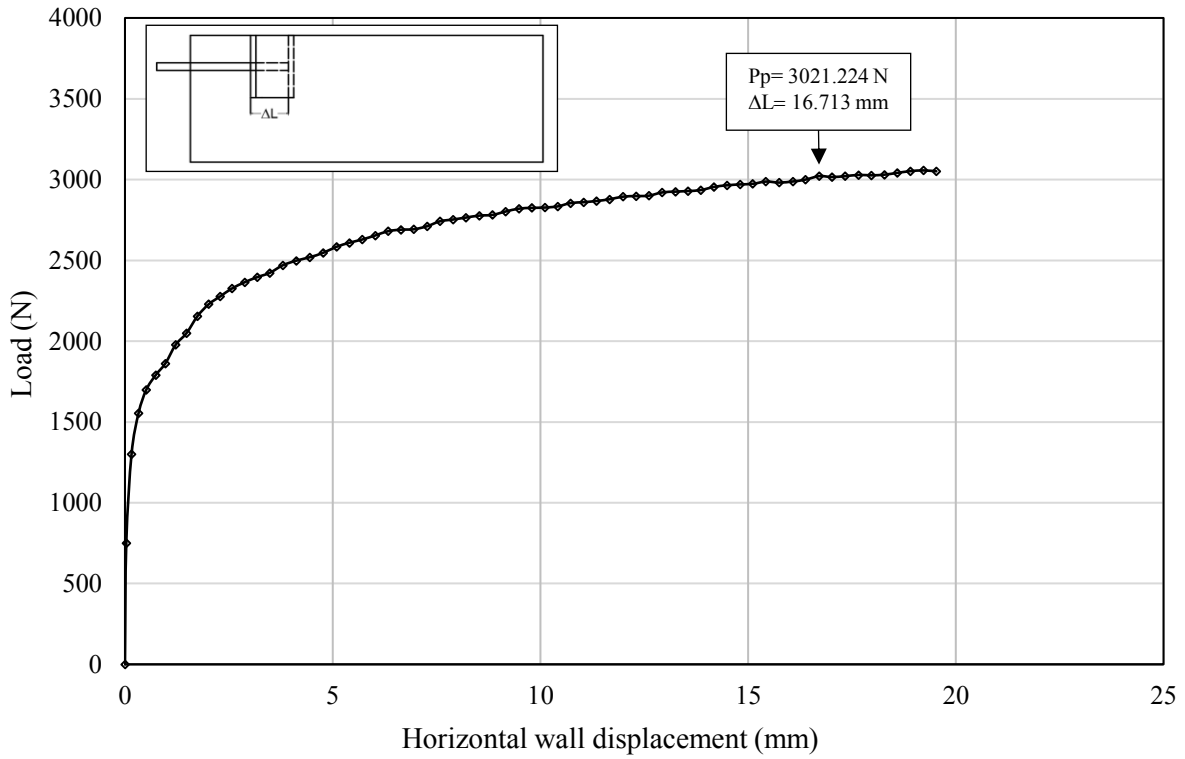
**(b)**



(c)

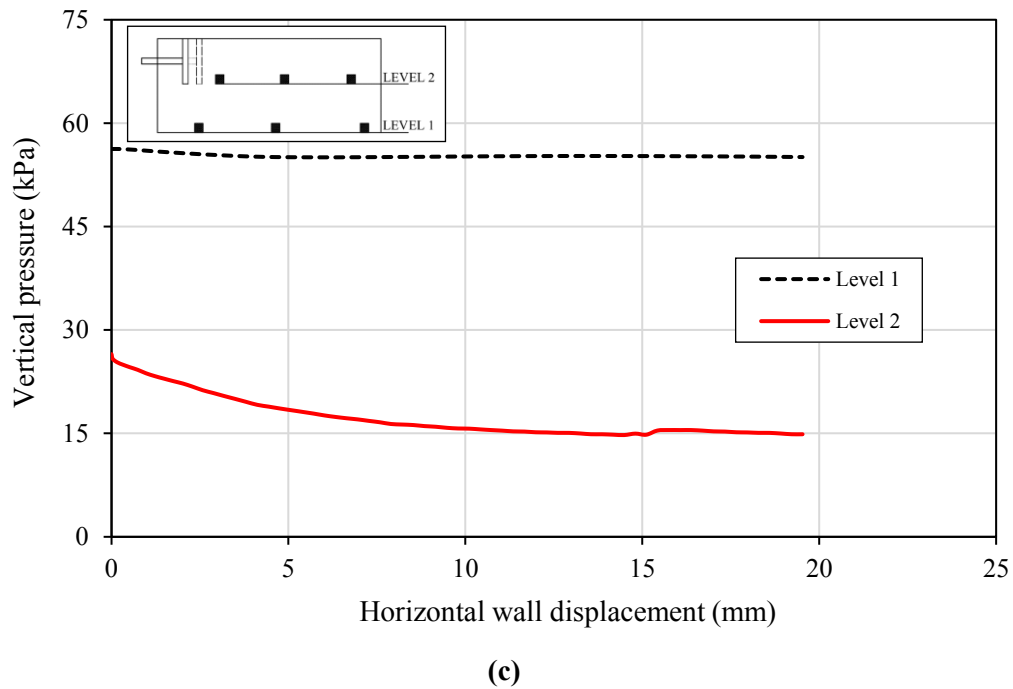
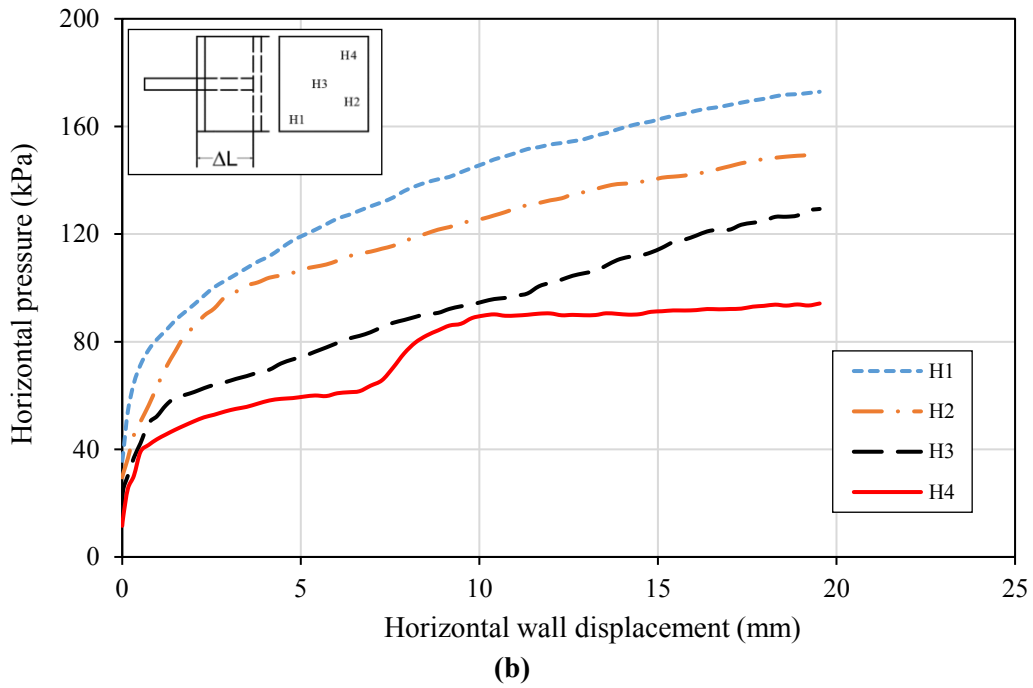
**Fig. 5.7.** Results for test no. 7 (Se2) for soil C –  $C_p = 12.5\%$ : (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the soil mass versus the wall displacement

**Results for test no. 8**



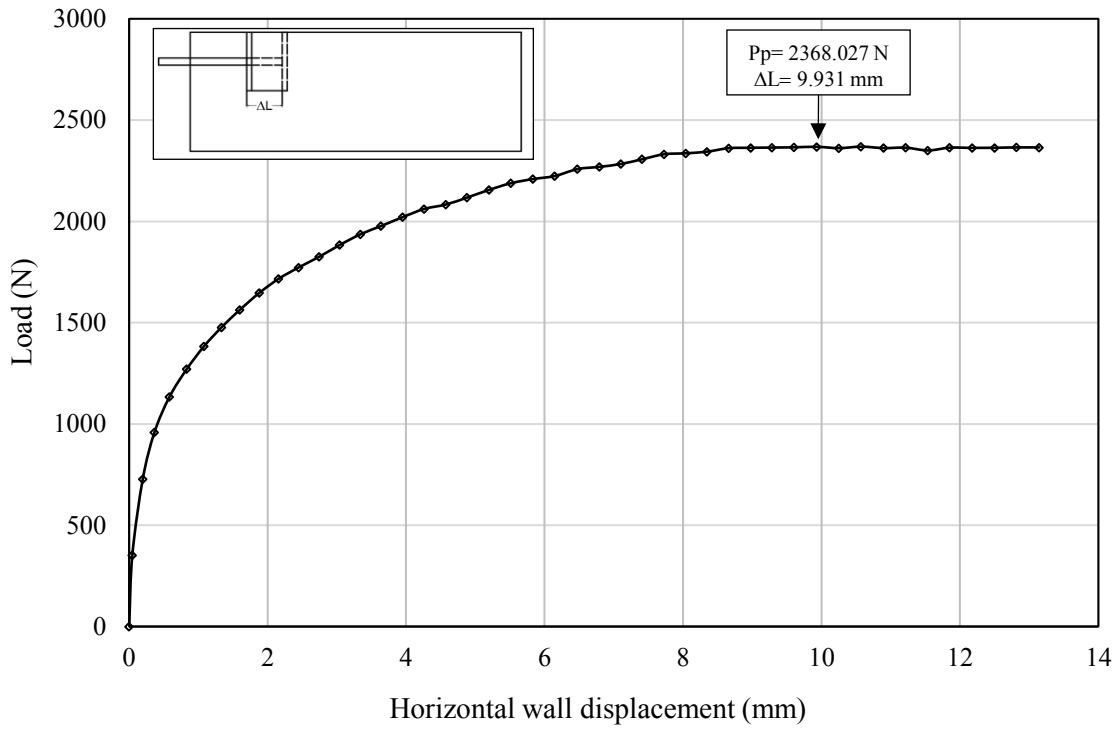
(a)



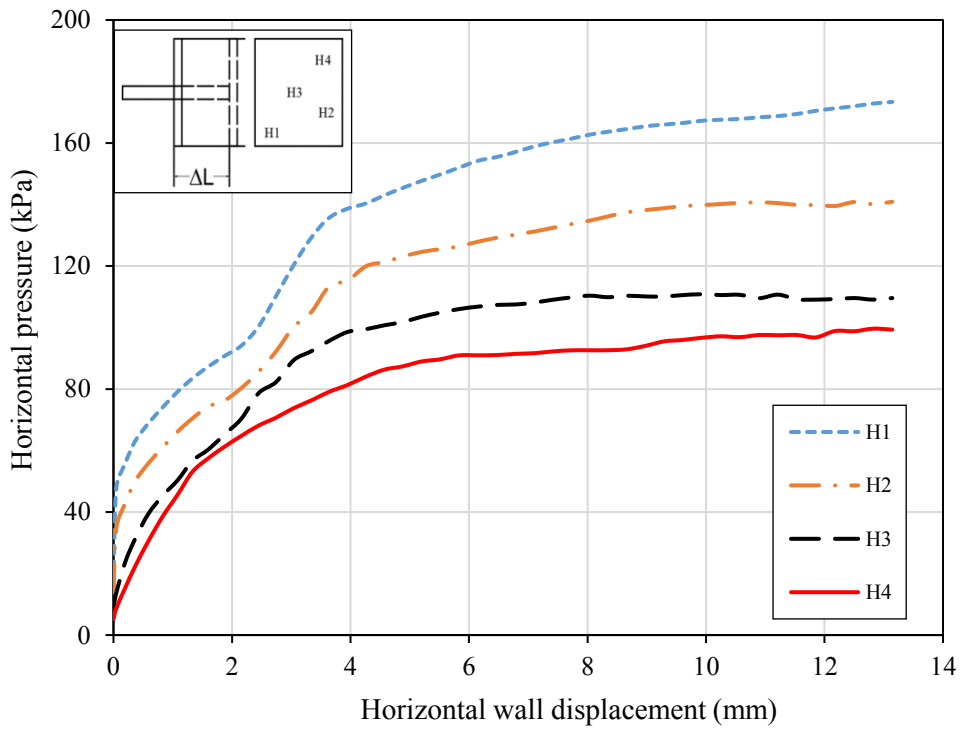


**Fig. 5.8.** Results for test no. 8 (Se2) for soil D –  $C_p = 18\%$ : (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the soil mass versus the wall displacement

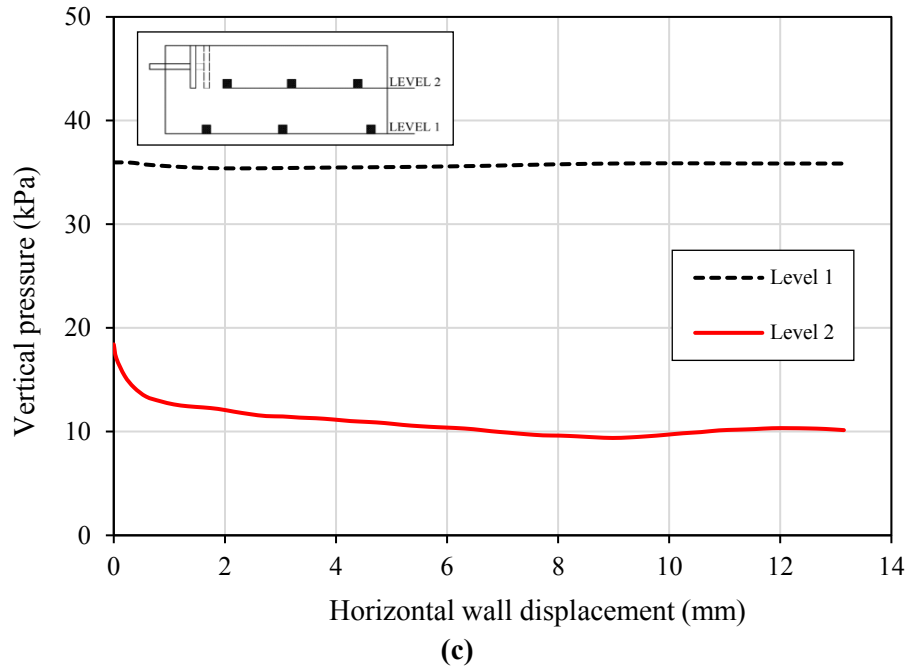
**Results for test no. 9**



**(a)**

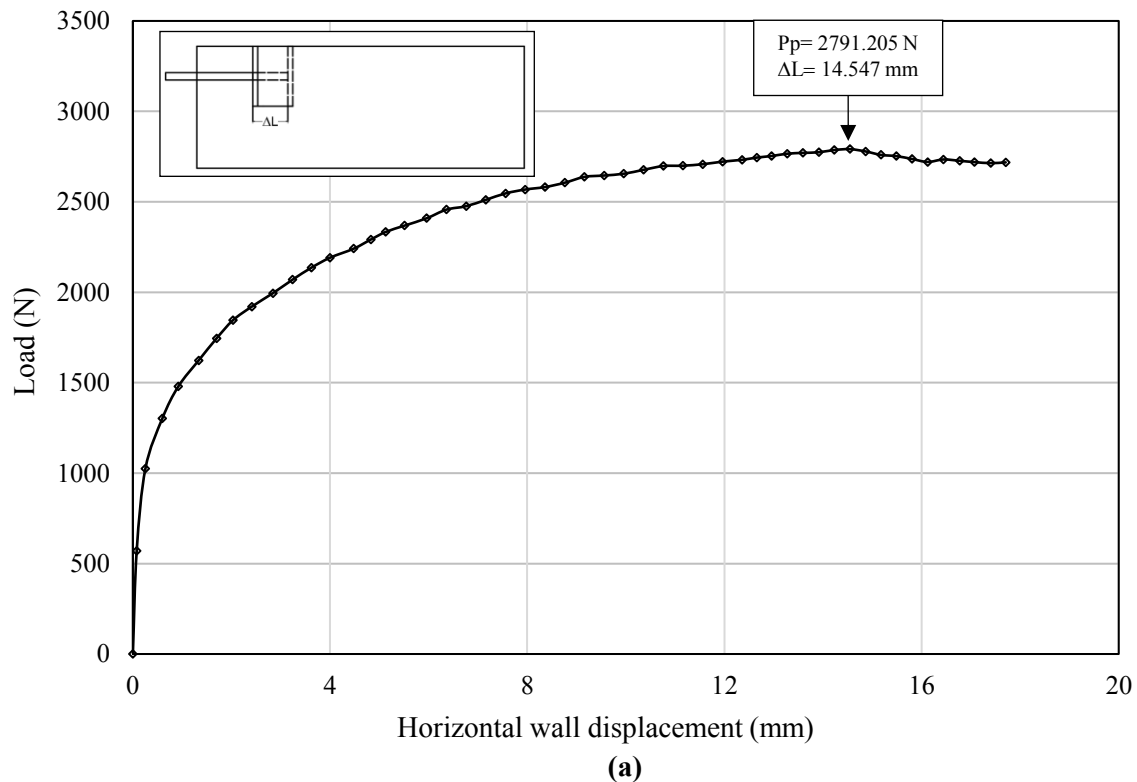


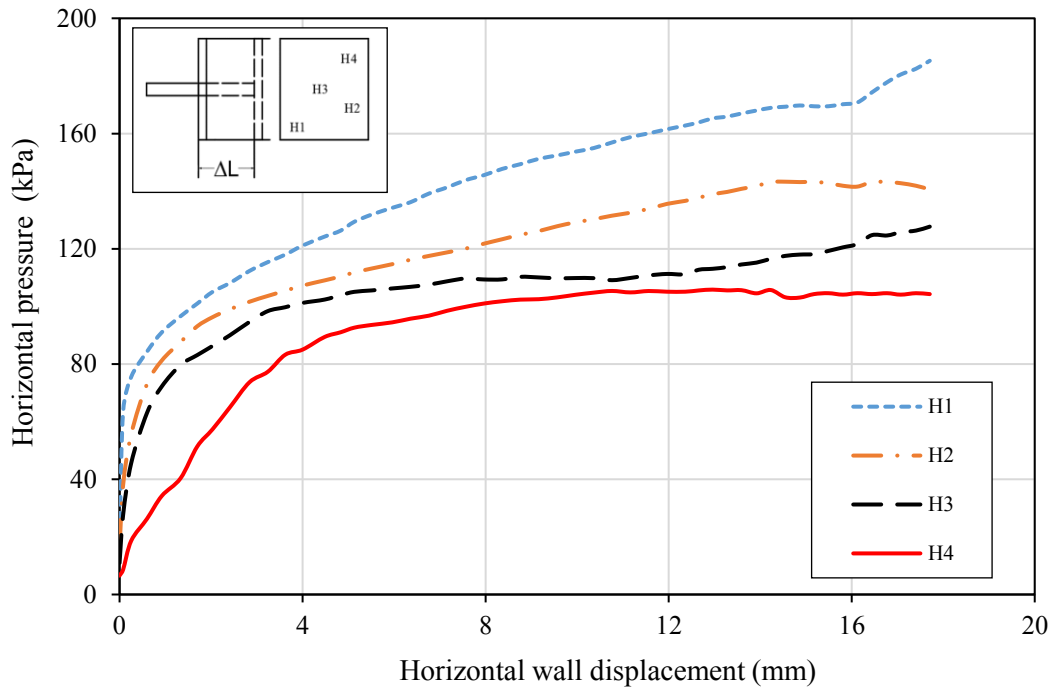
**(b)**



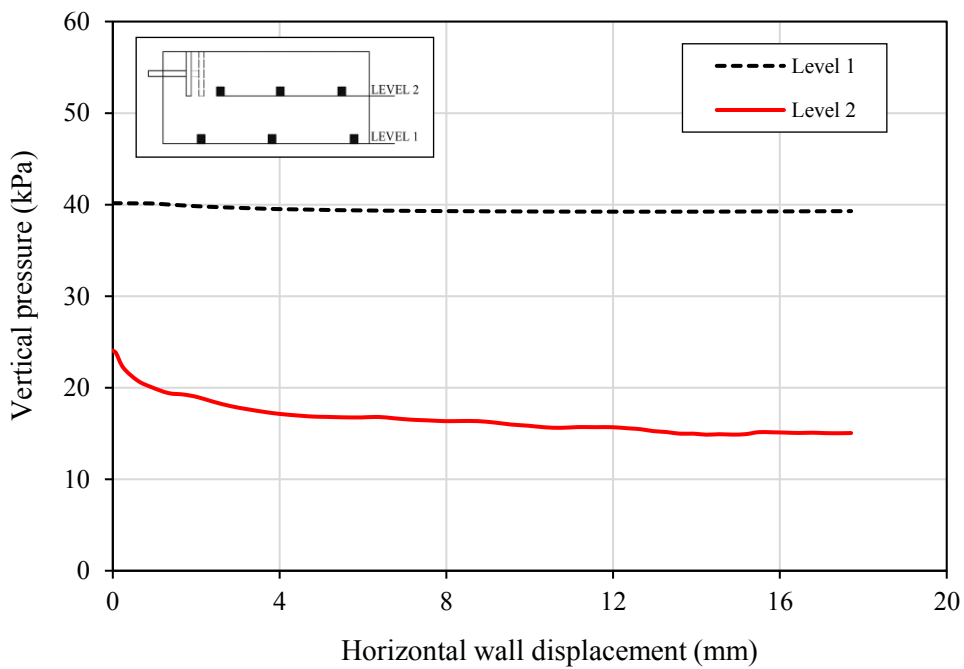
**Fig. 5.9.** Results for test no. 9 (Se3) for soil A –  $C_p = 4.2\%$ : (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the soil mass versus the wall displacement

**Results for test no. 10**





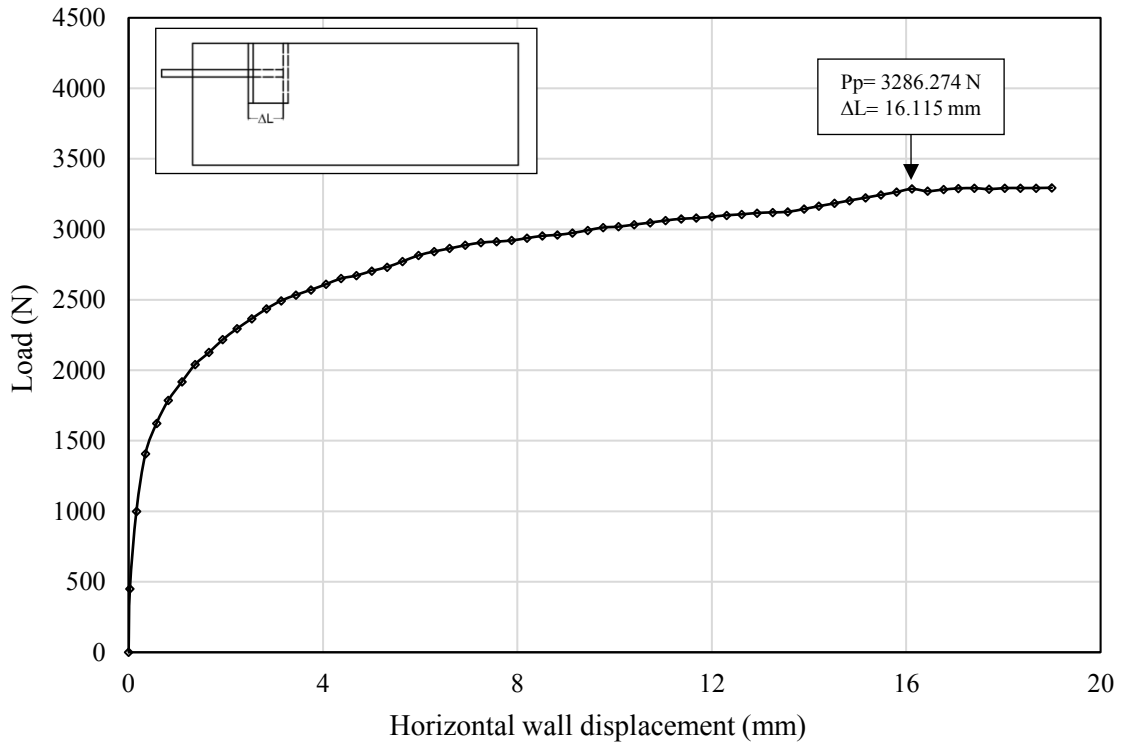
(b)



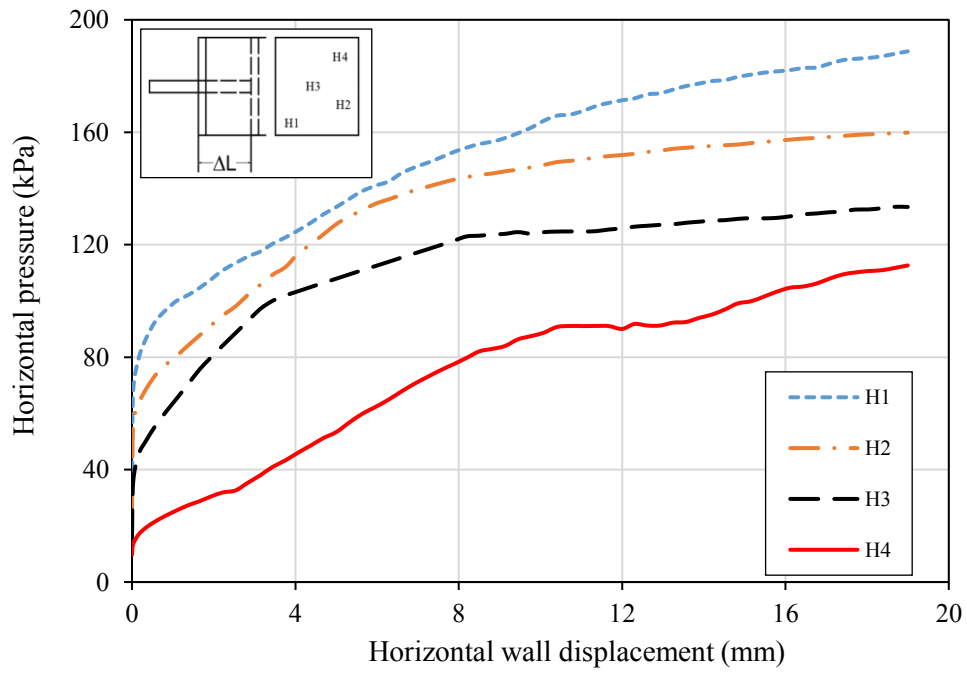
(c)

**Fig. 5.10.** Results for test no.10 (Se3) for soil B –  $C_p=9.0\%$ : (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the soil mass versus the wall displacement

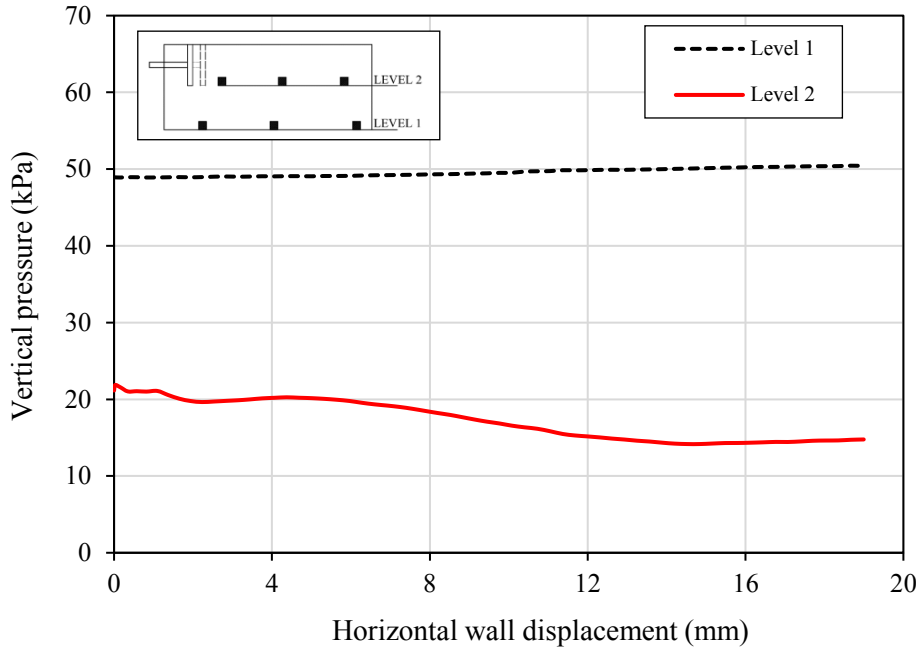
**Results for test no. 11**



**(a)**



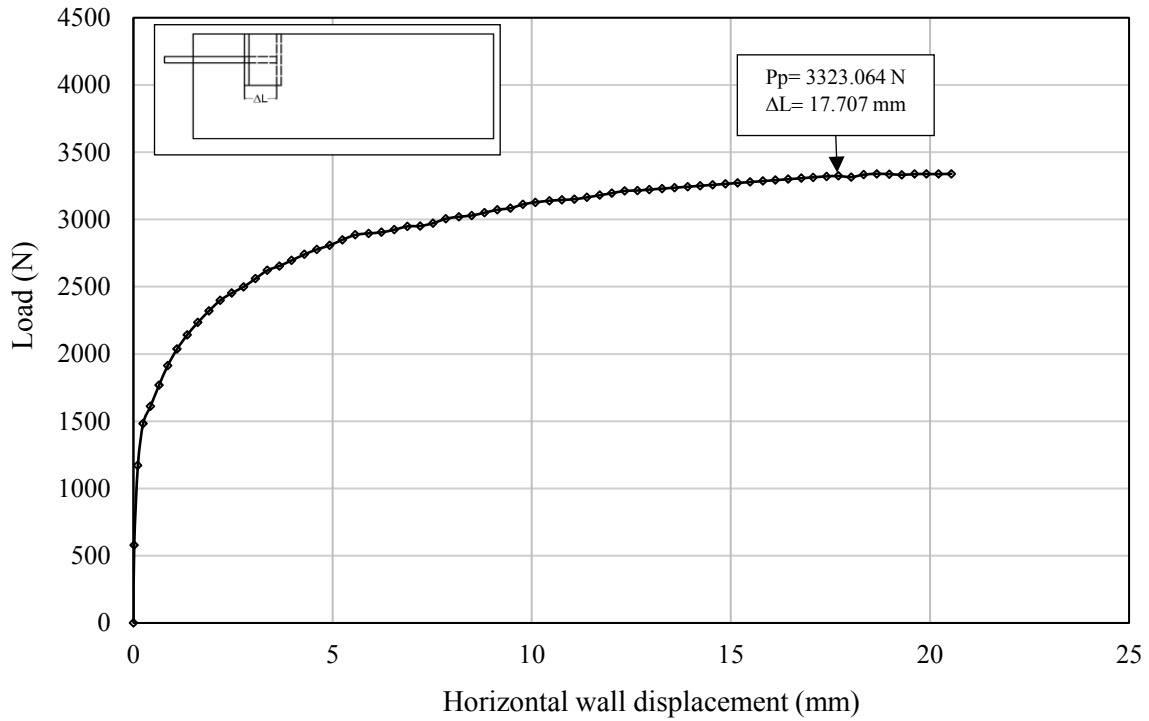
**(b)**



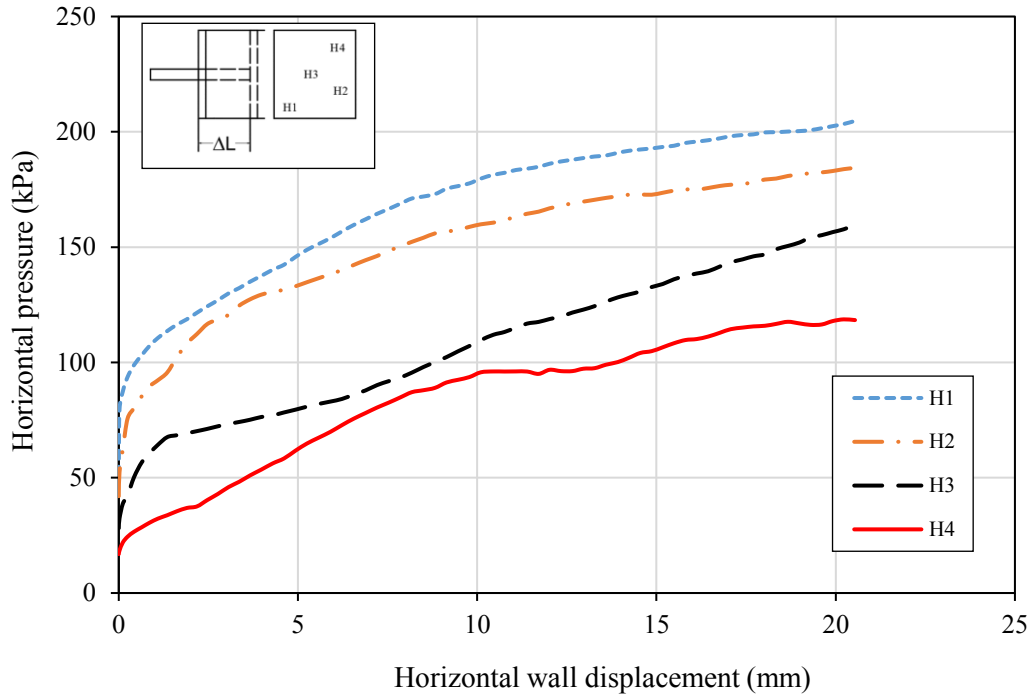
(c)

**Fig. 5.11.** Results for test no. 11 (Se3) for soil C –  $C_p = 12.5\%$ : (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the soil mass versus the wall displacement

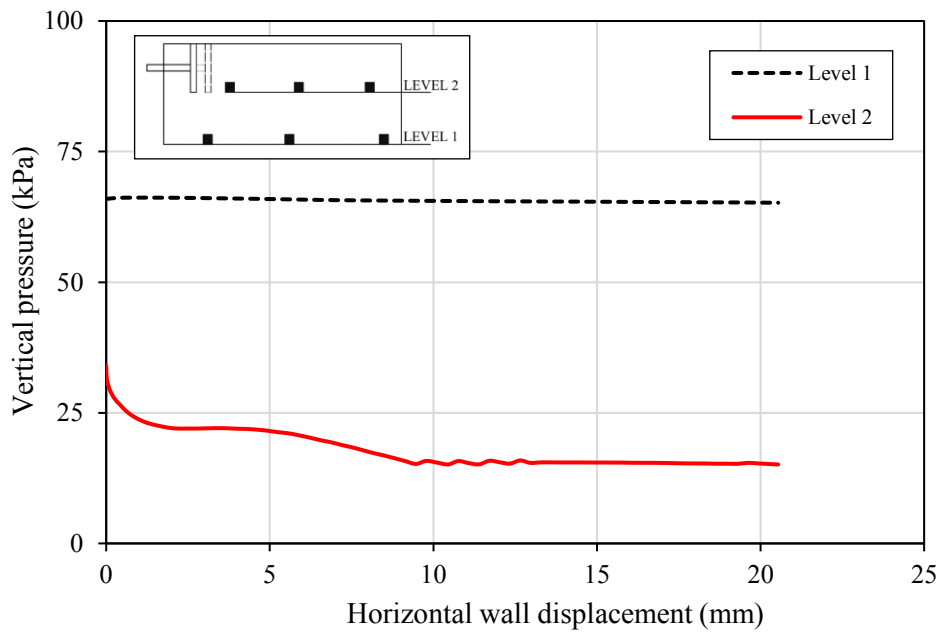
**Results for test no. 12**



(a)



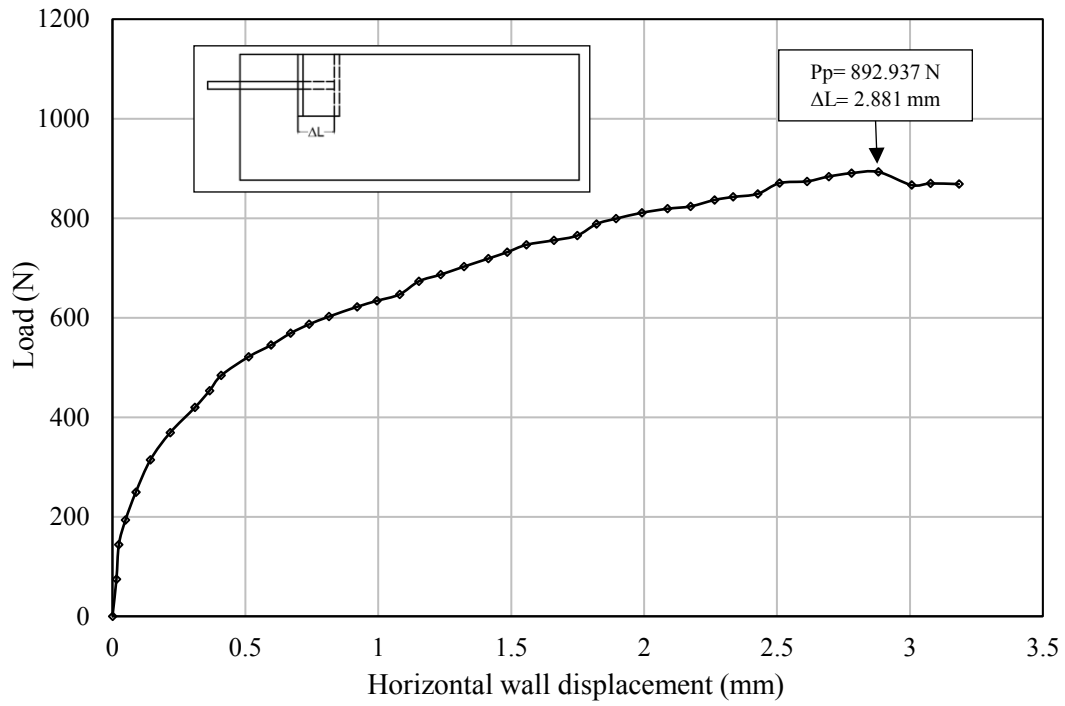
(b)



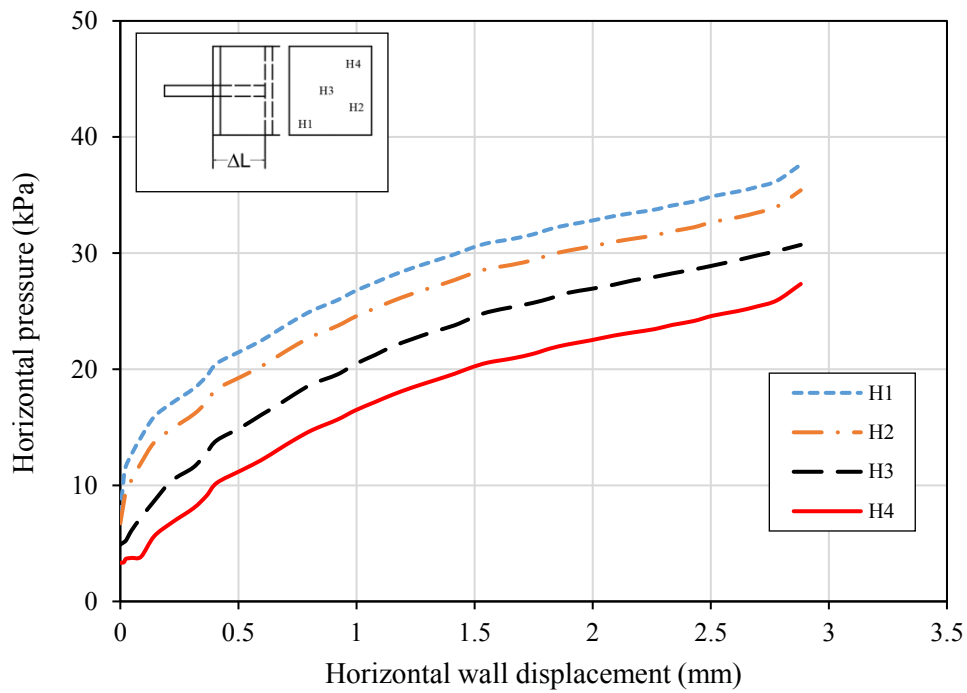
(c)

**Fig. 5.12.** Results for test no. 12 (Se3) for soil D –  $C_p = 18\%$ : (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the soil mass versus the wall displacement

**Results for test no. 13**

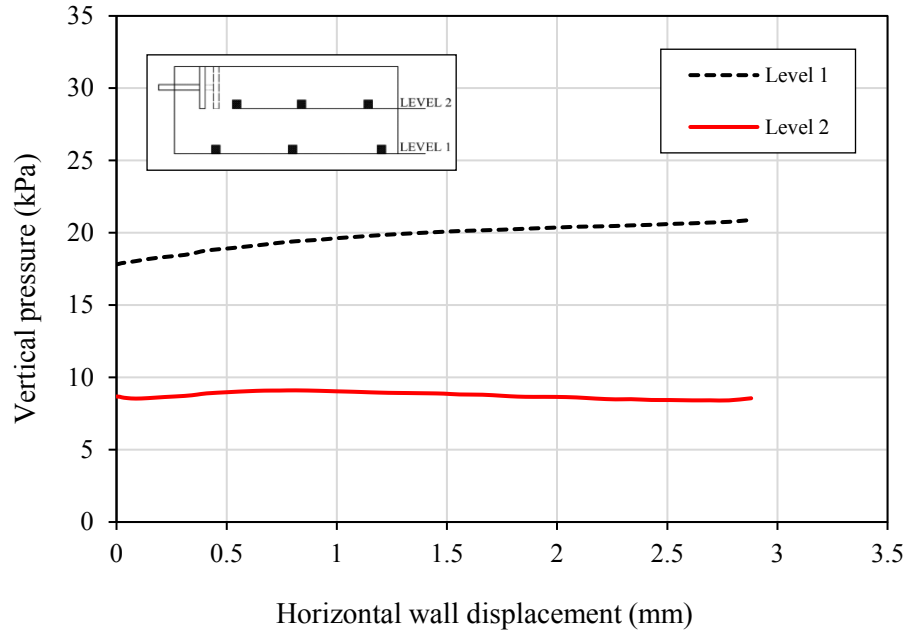


**(a)**



**(b)**

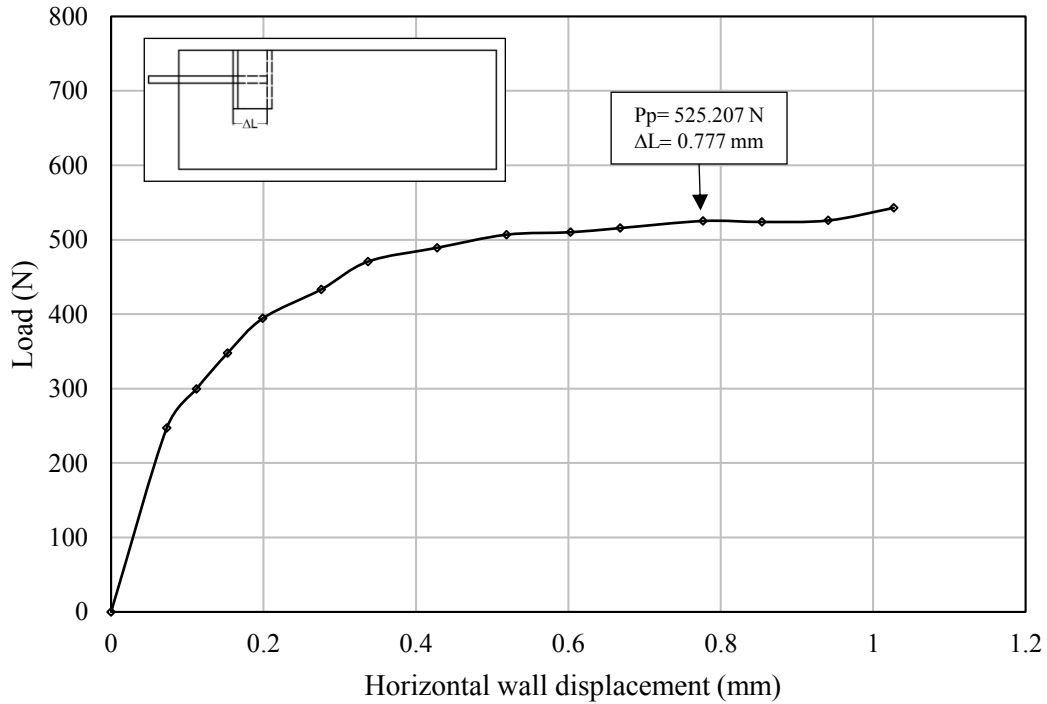




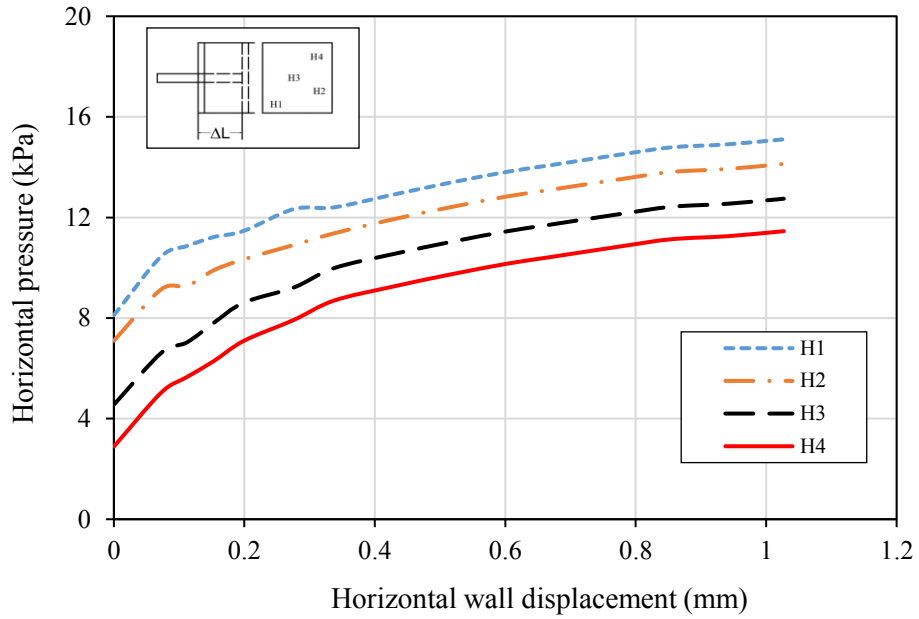
(c)

**Fig. 5.13.** Results for test no.13 (Se1') for soil A –  $C_p = 4.2\%$ : (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the soil mass versus the wall displacement

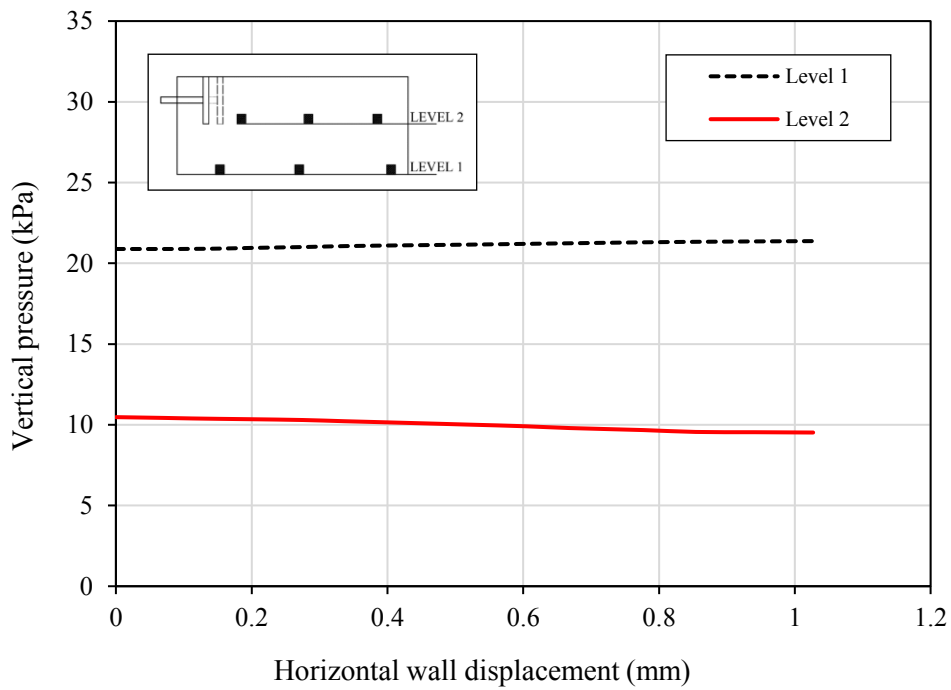
**Results for test no. 14**



(a)



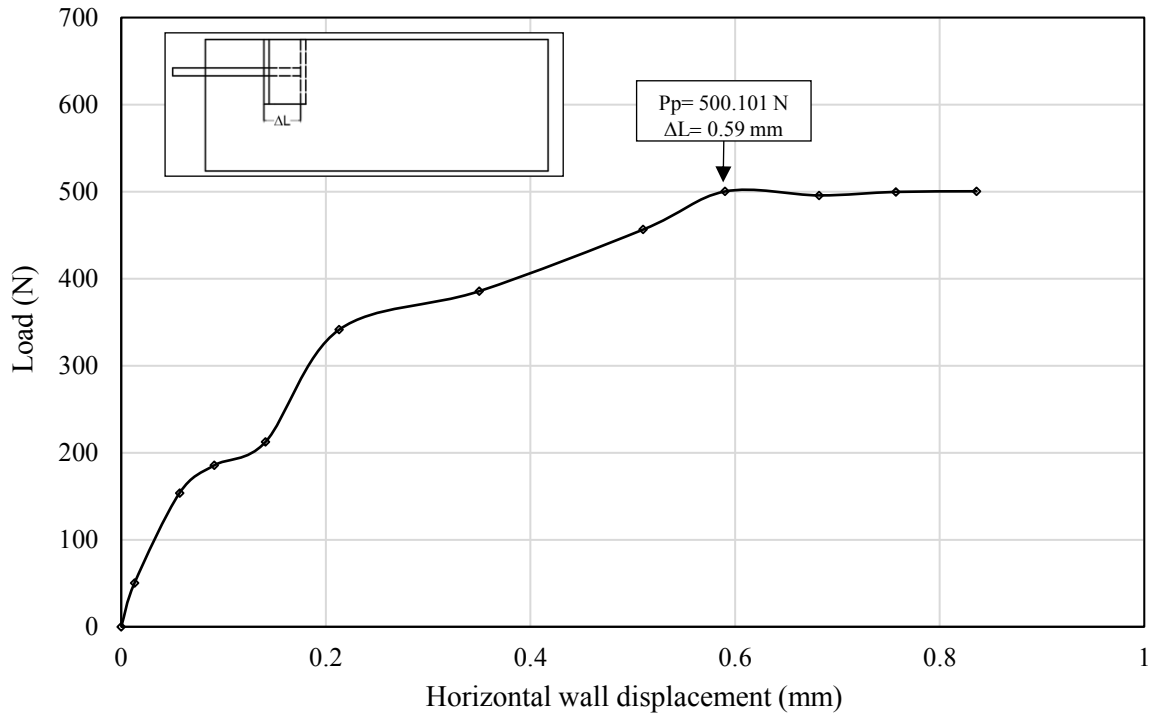
(b)



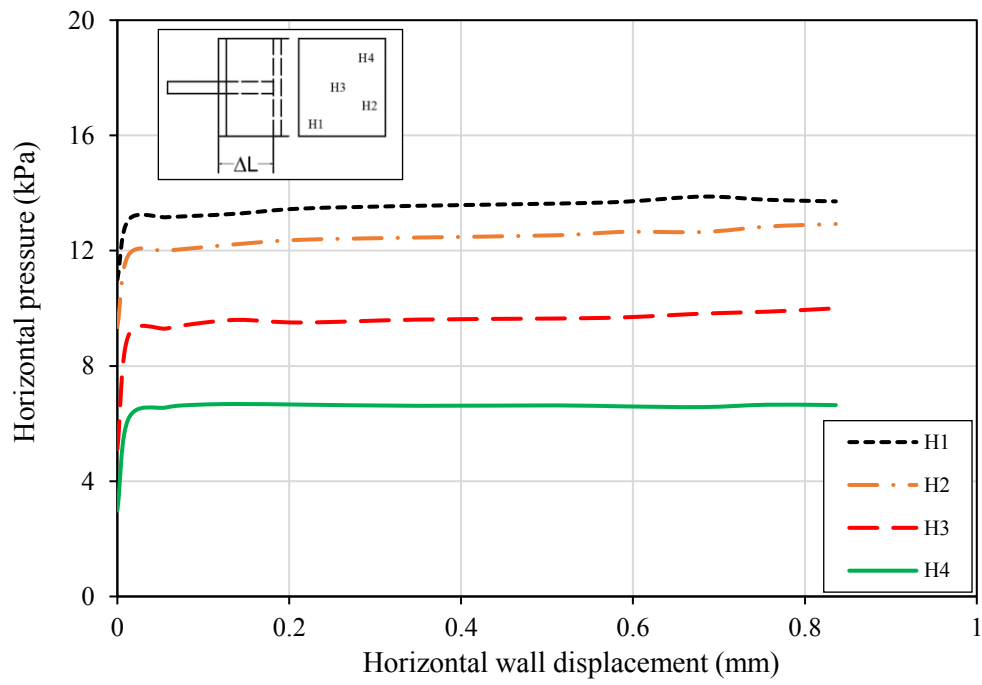
(c)

**Fig. 5.14.** Results for test no.14 (Se1') for soil B –  $C_p = 9.0\%$ : (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the soil mass versus the wall displacement

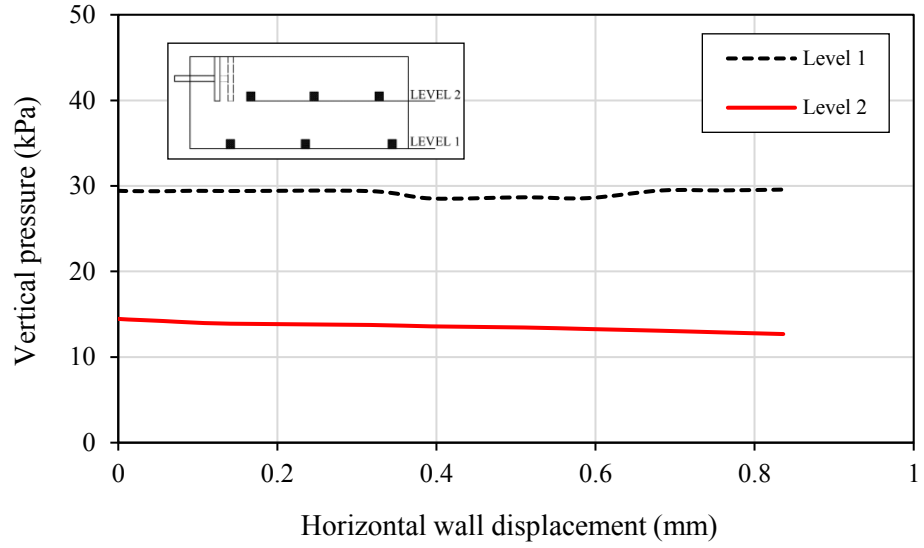
**Results for test no. 15**



**(a)**



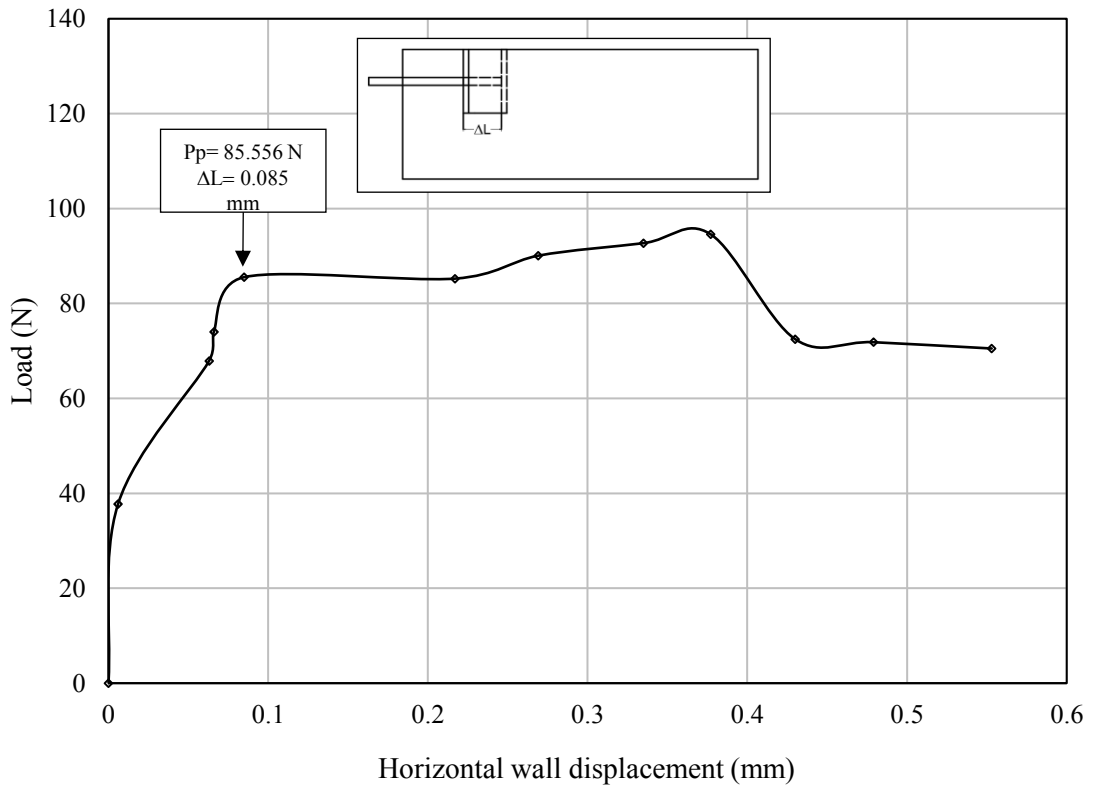
**(b)**



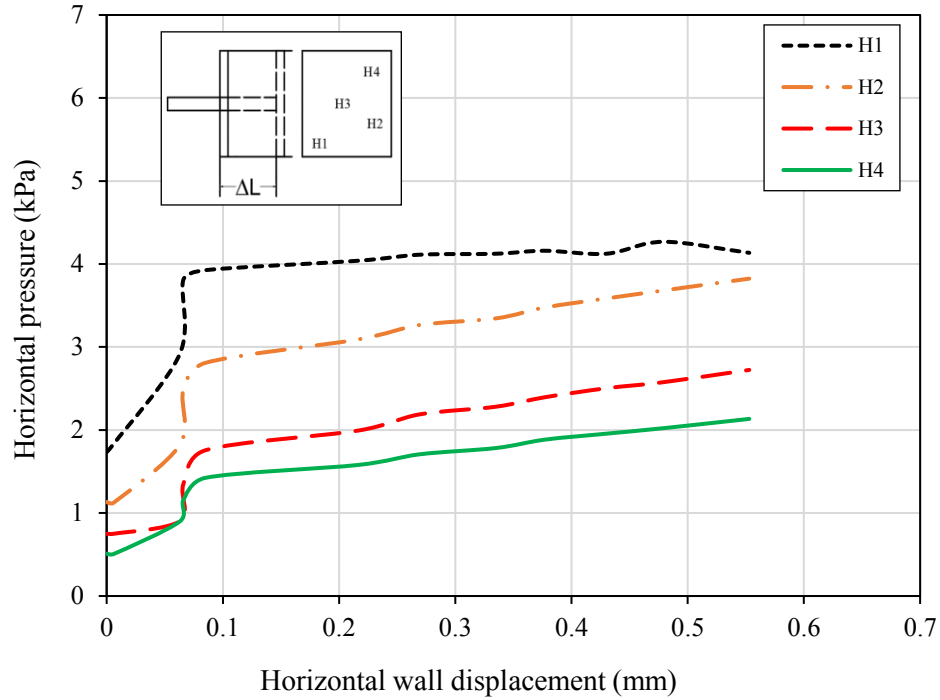
(c)

**Fig. 5.15.** Results for test no. 15 (Se1') for soil C –  $C_p = 12.5\%$ : (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the soil mass versus the wall displacement

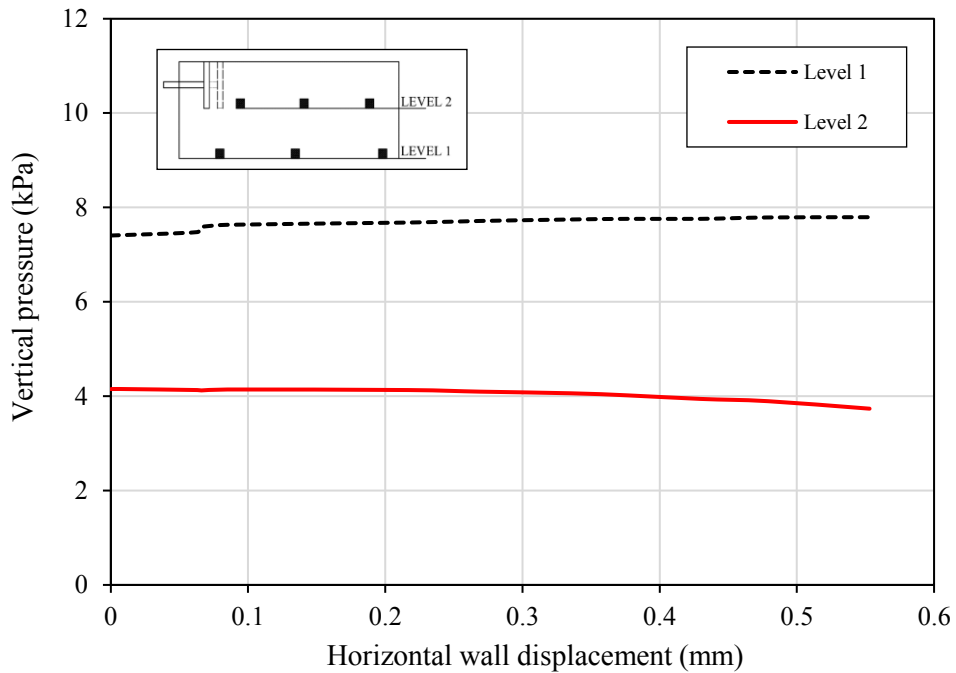
**Results for test no. 16**



(a)



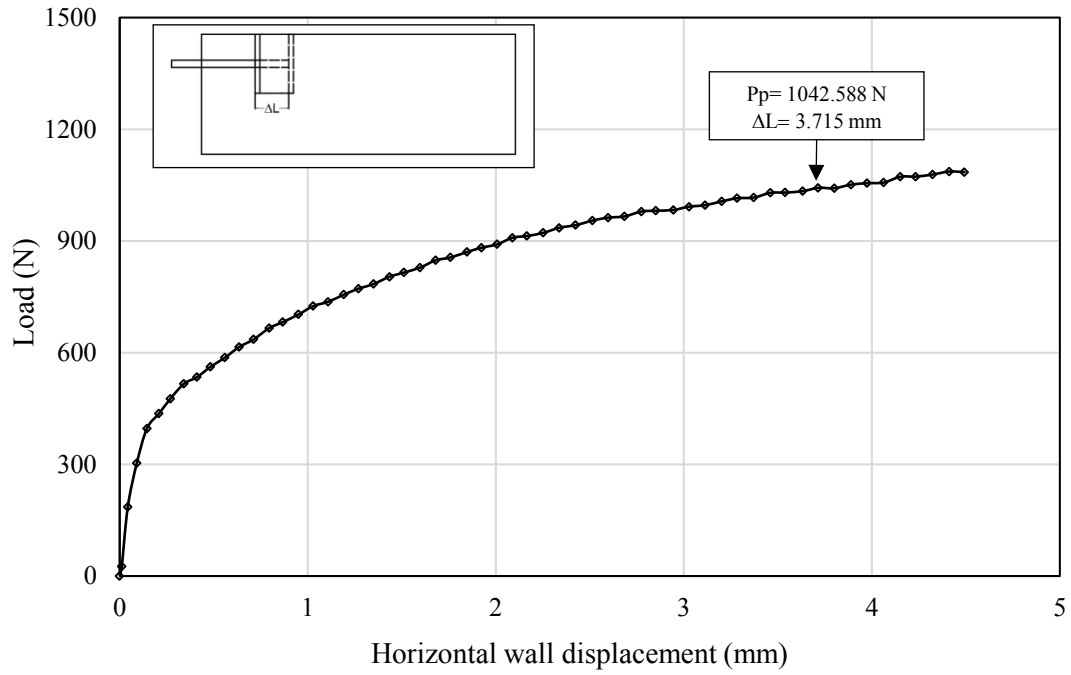
(b)



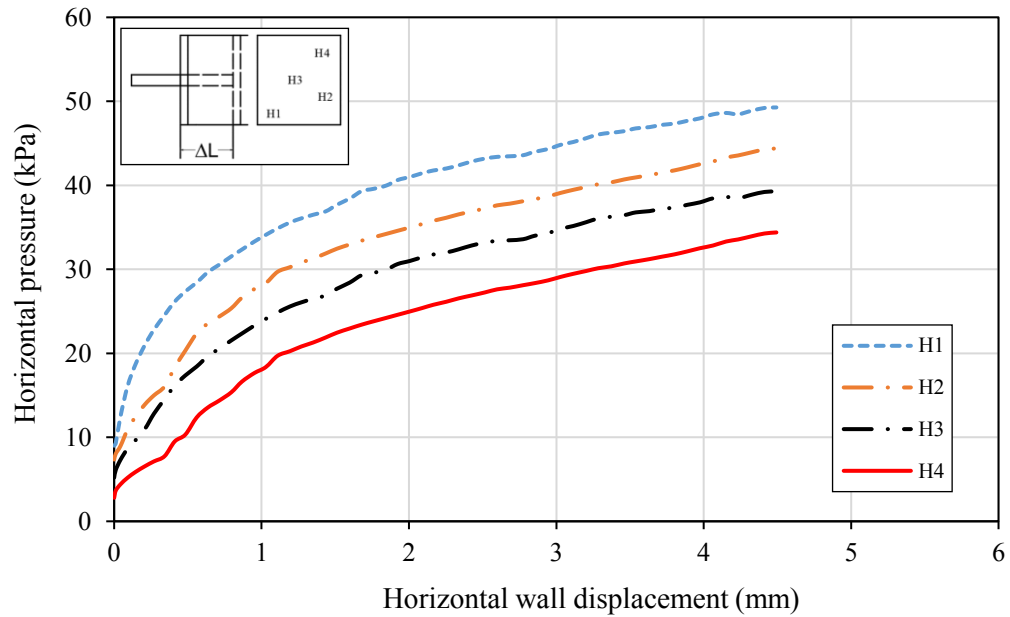
(c)

**Fig. 5.16.** Results for test no.16 (Se1') for soil D –  $C_p = 18\%$ : (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the soil mass versus the wall displacement

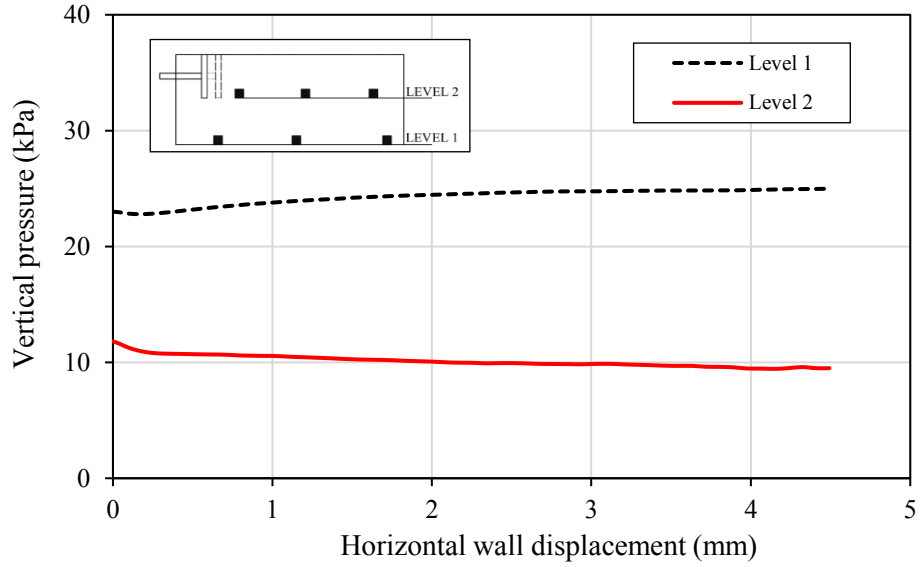
**Results for test no. 17**



**(a)**



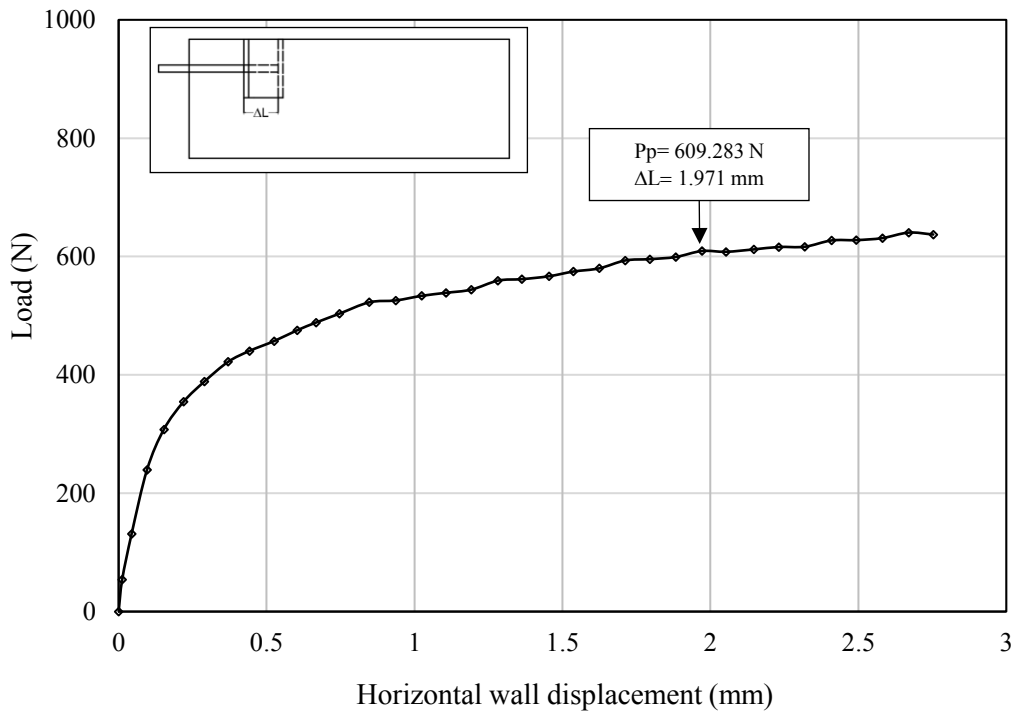
**(b)**



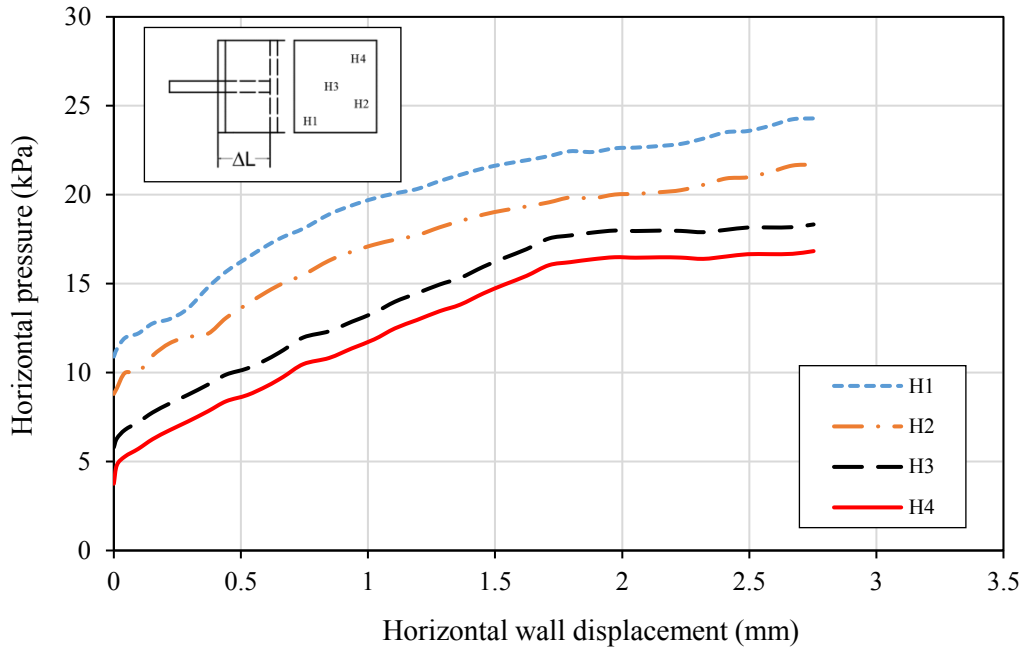
(c)

**Fig. 5.17.** Results for test no. 17 (Se2') for soil A –  $C_p = 4.2\%$ : (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the soil mass versus the wall displacement

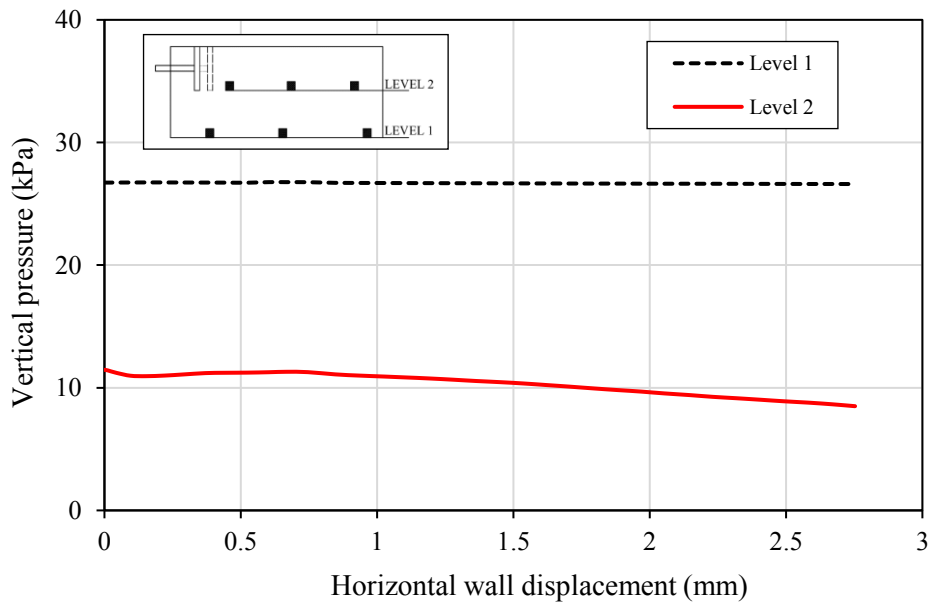
**Results for test no. 18**



(a)



(b)

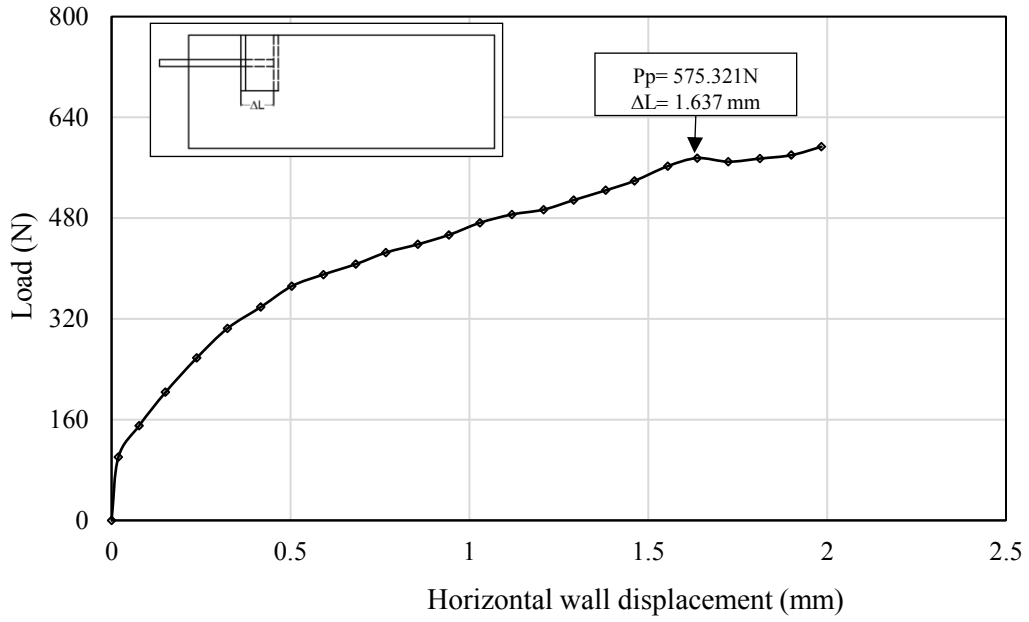


(c)

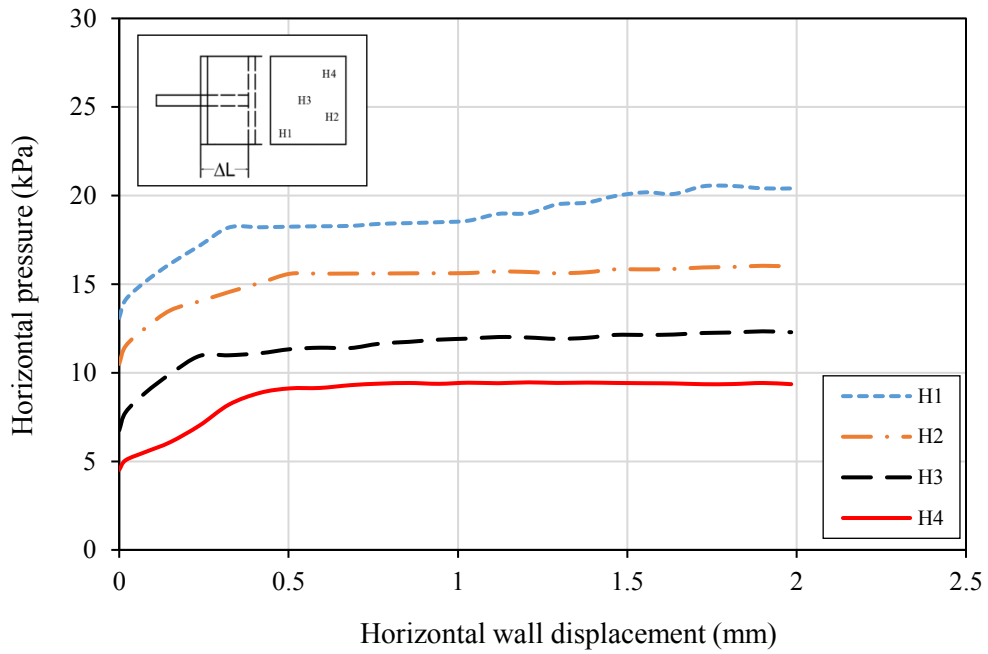
**Fig. 5.18.** Results for test no.18 (Se2') for soil B –  $C_p = 9.0\%$ : (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the soil mass versus the wall displacement



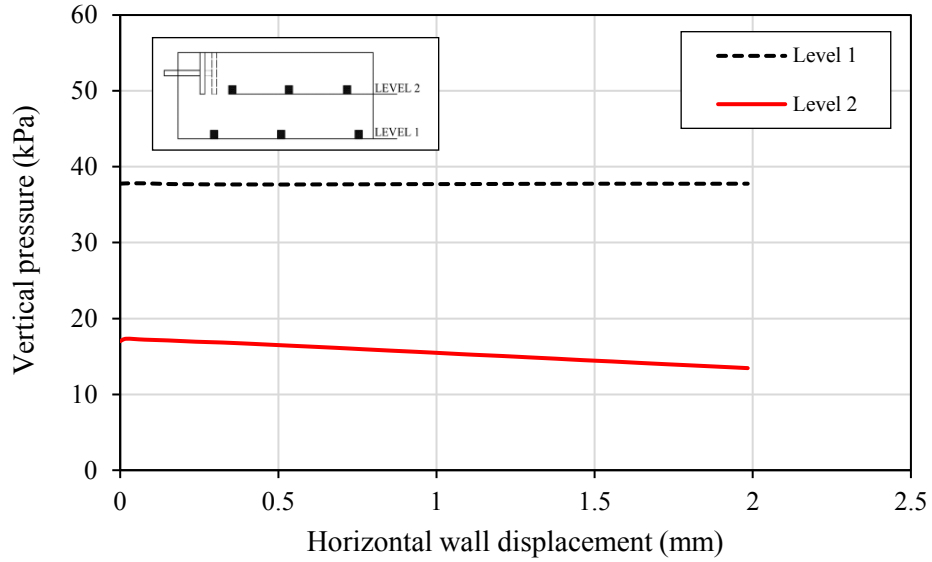
**Results for test no. 19**



**(a)**



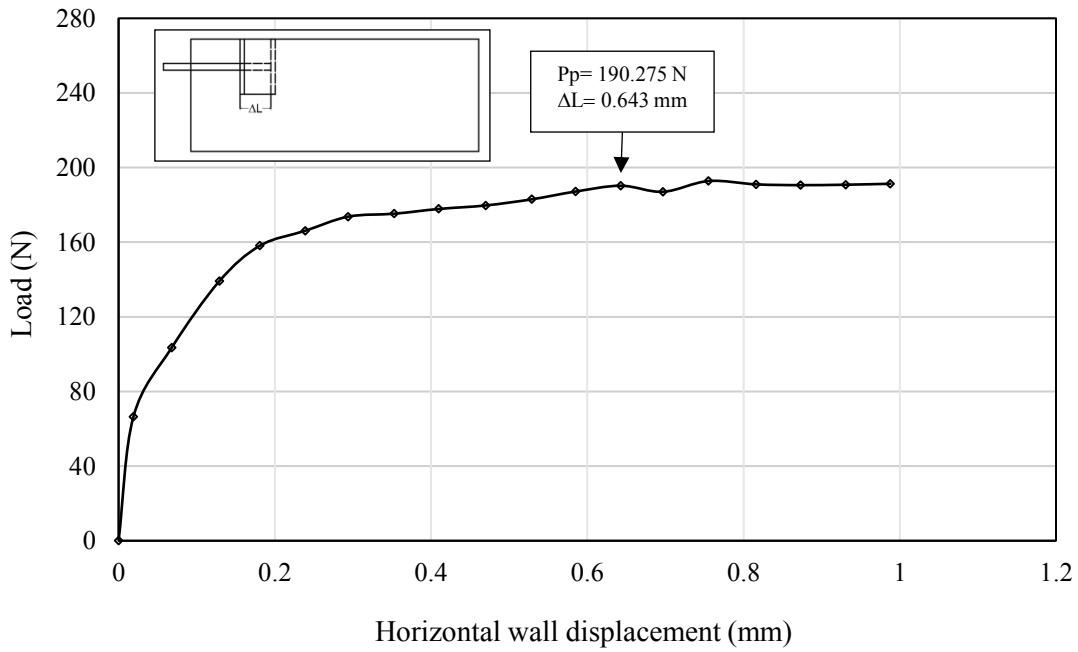
**(b)**



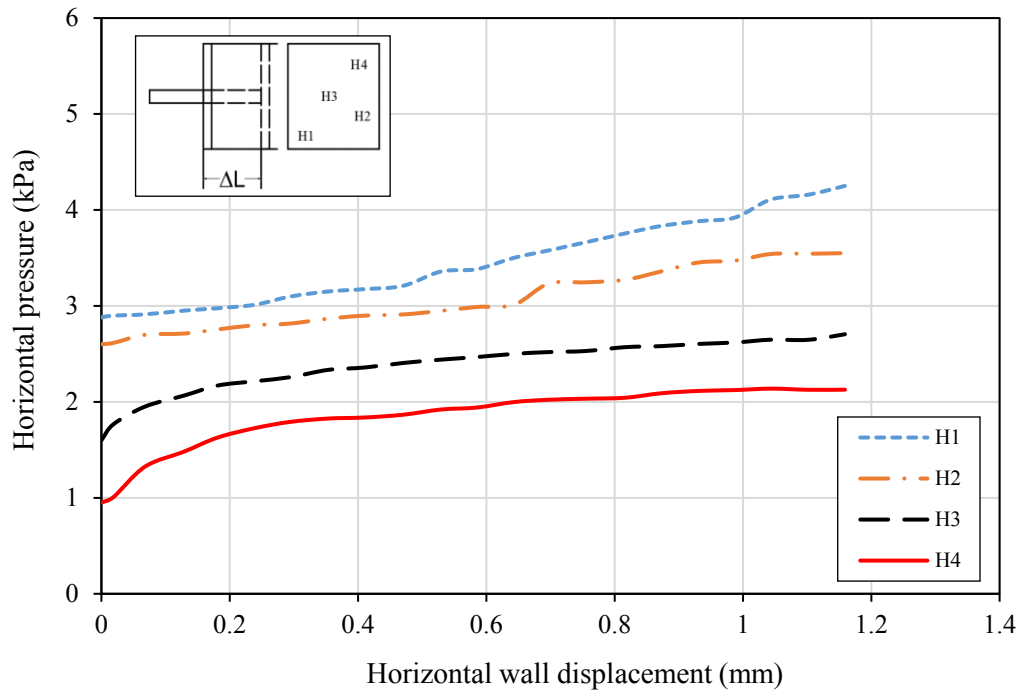
(c)

**Fig. 5.19.** Results for test no.19 (Se2') for soil C –  $C_p = 12.5\%$ : (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the soil mass versus the wall displacement

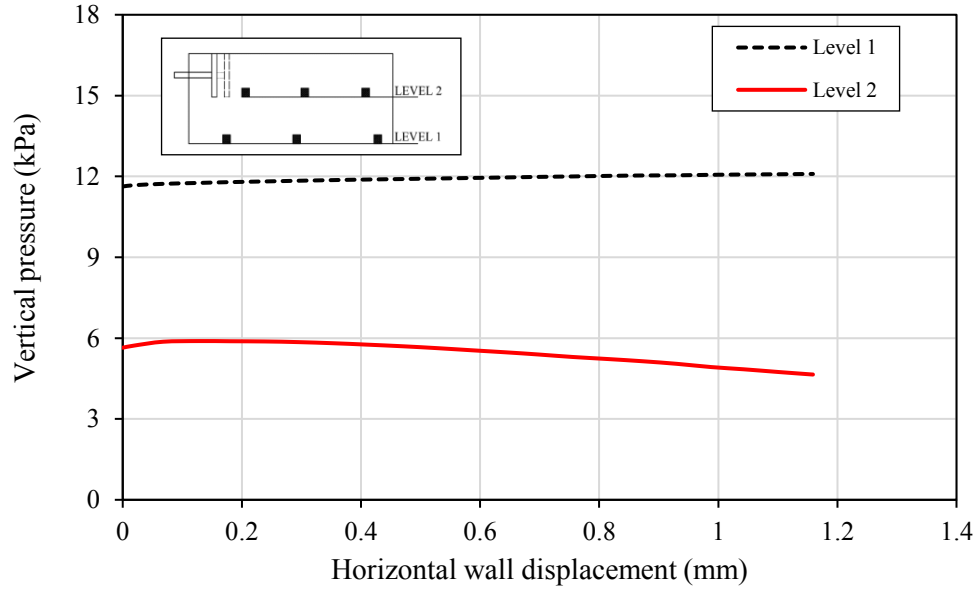
**Results for test no. 20**



(a)



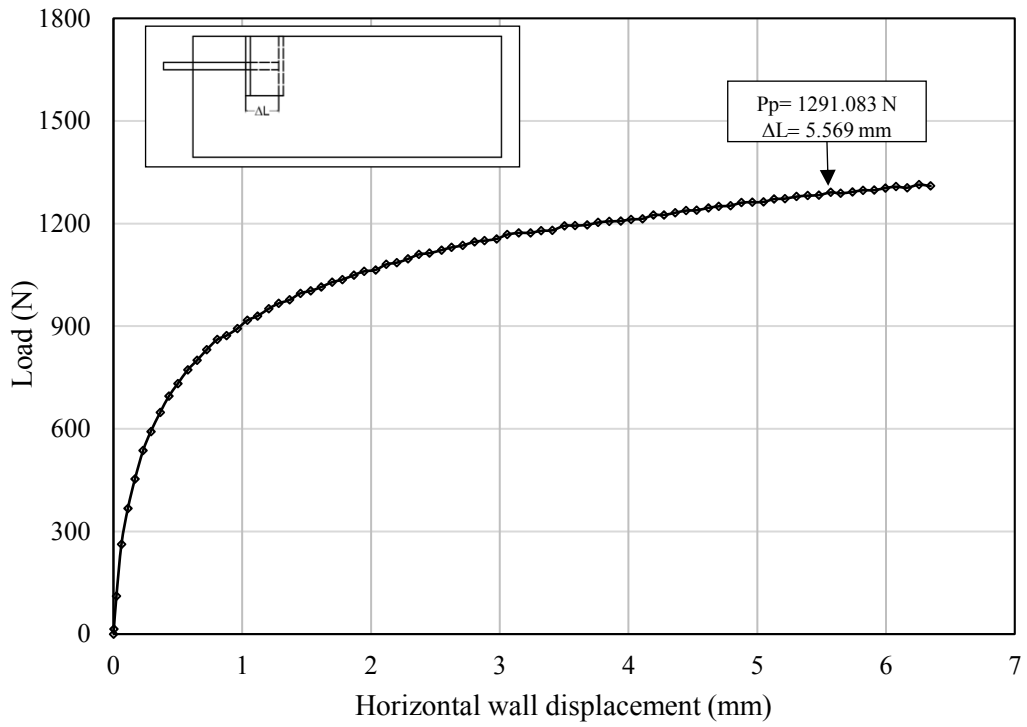
(b)



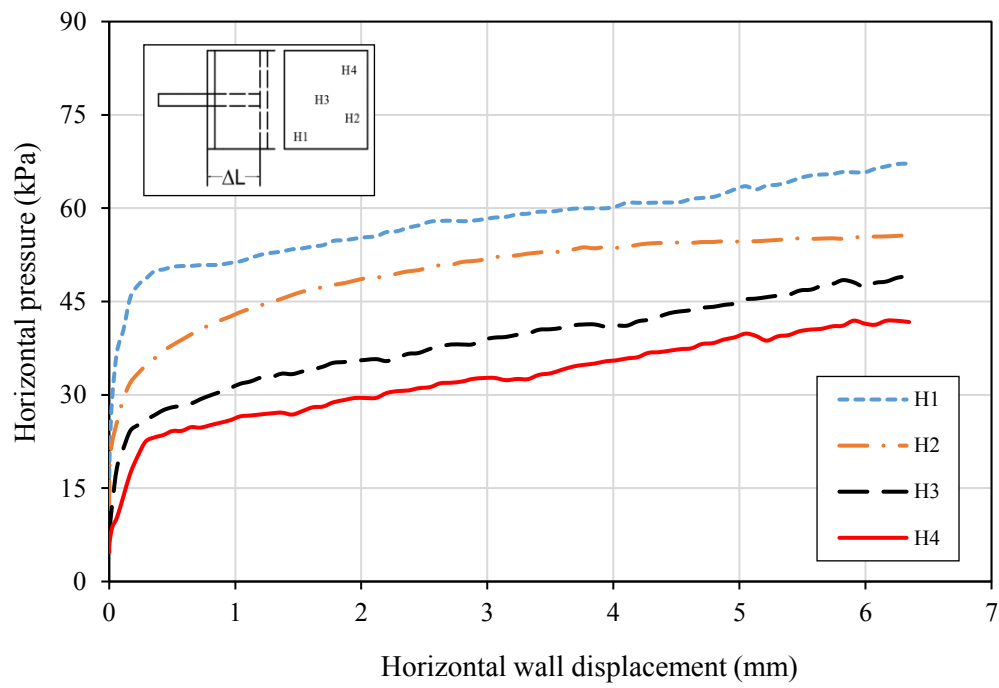
(c)

**Fig. 5.20.** Results for test no. 20 (Se2') for soil D –  $C_p = 18\%$ : (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the soil mass versus the wall displacement

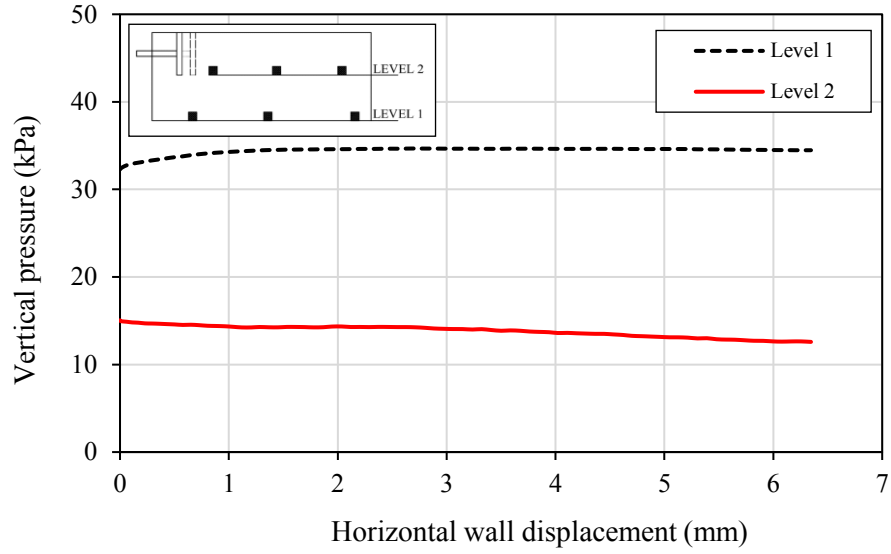
**Results for test no. 21**



**(a)**



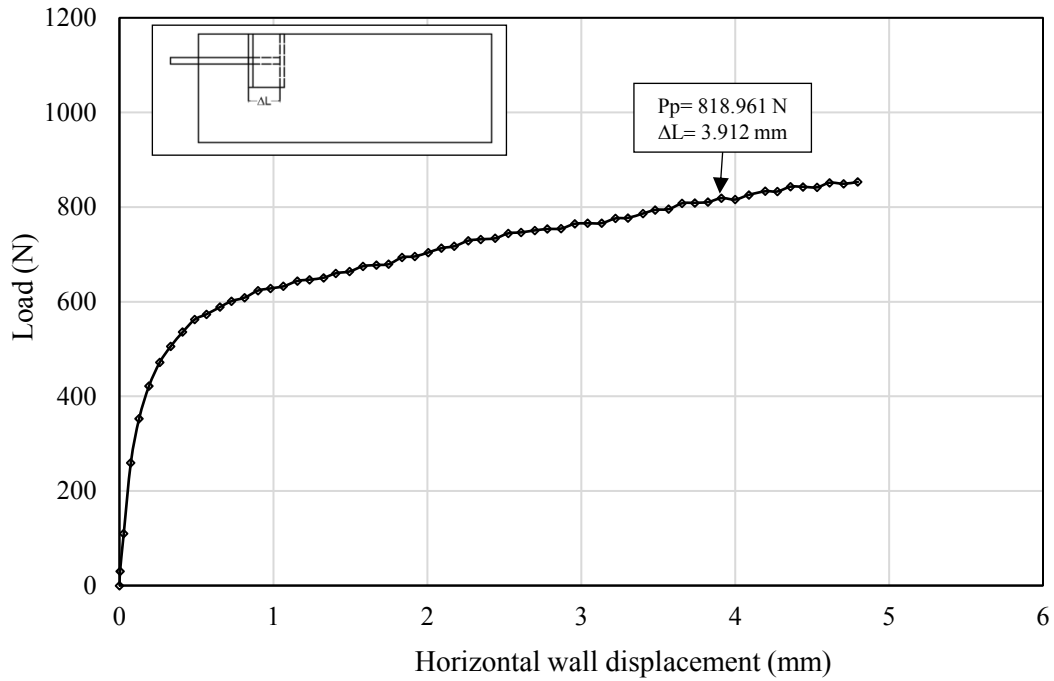
**(b)**



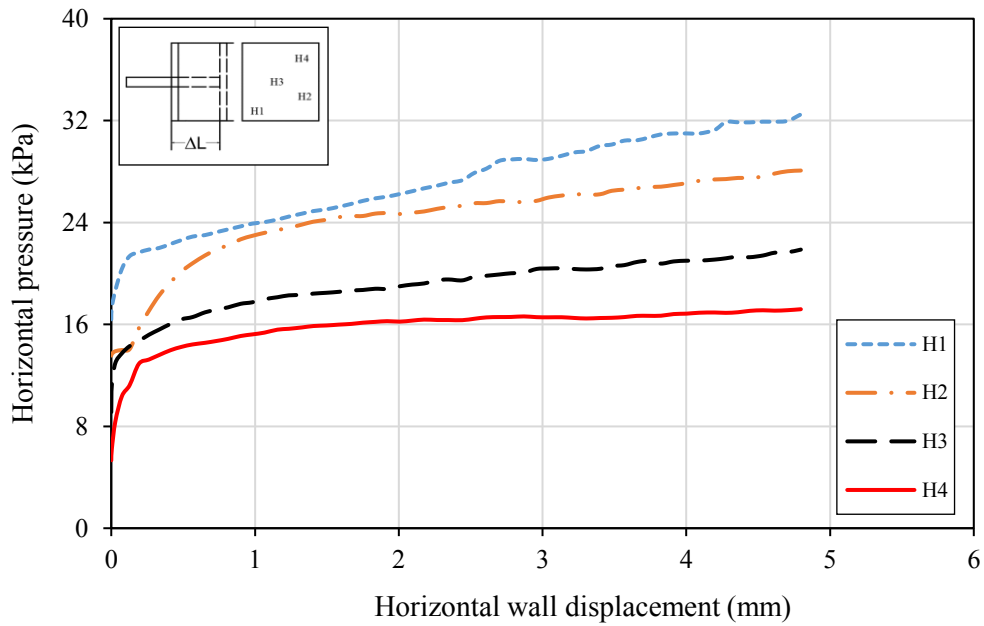
(c)

**Fig. 5.21.** Results for test no.21 (Se3') for soil A –  $C_p = 4.2\%$ : (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the soil mass versus the wall displacement

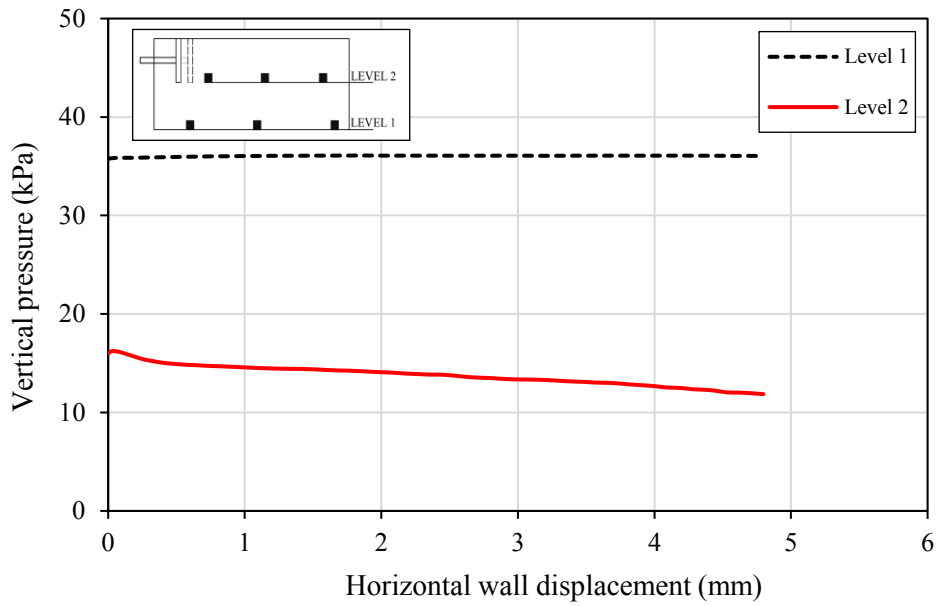
**Results for test no. 22**



(a)



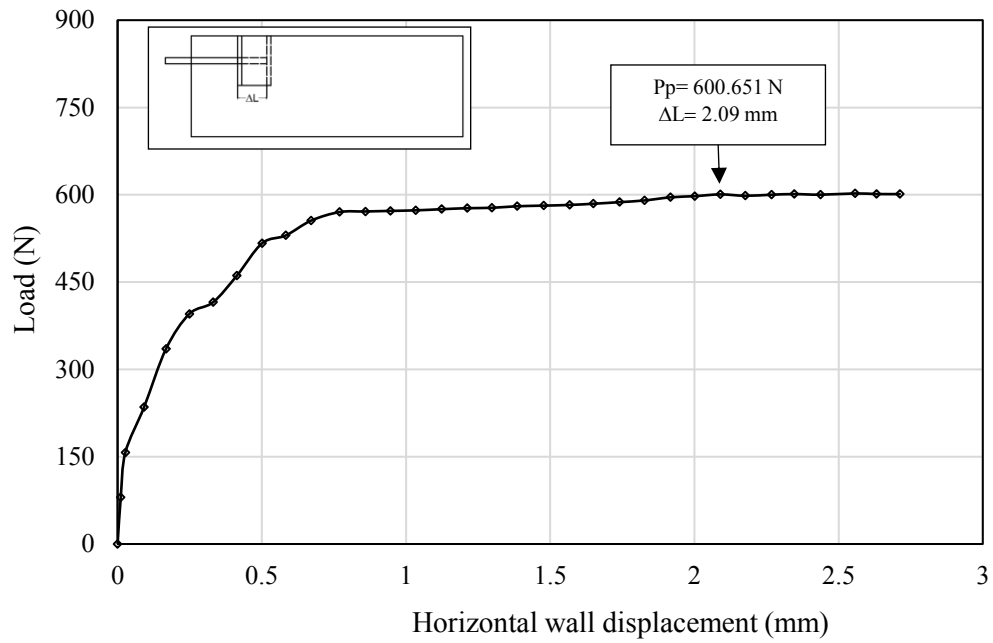
(b)



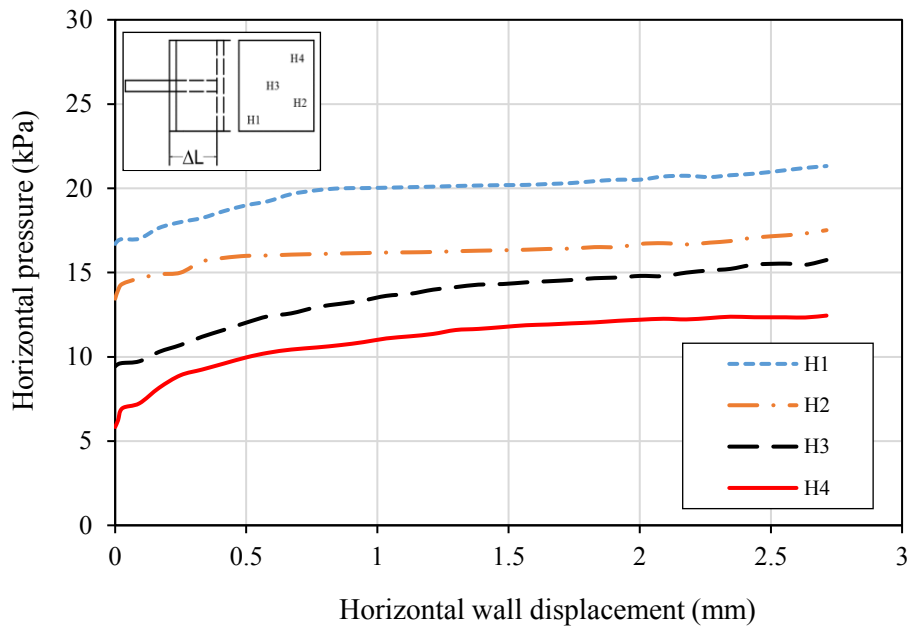
(c)

**Fig. 5.22.** Results for test no. 22 (Se3') for soil B –  $C_p = 9.0\%$ : (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the soil mass versus the wall displacement

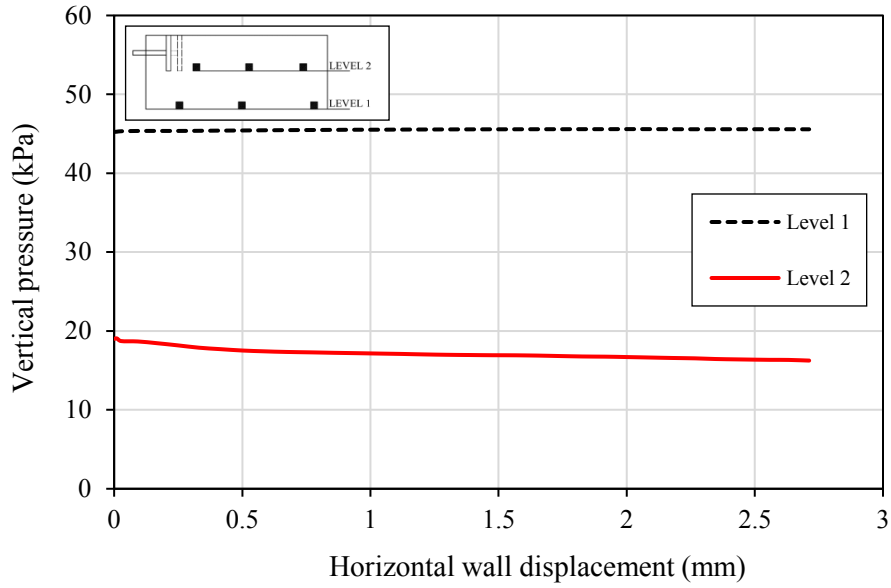
Results for test no. 23



(a)



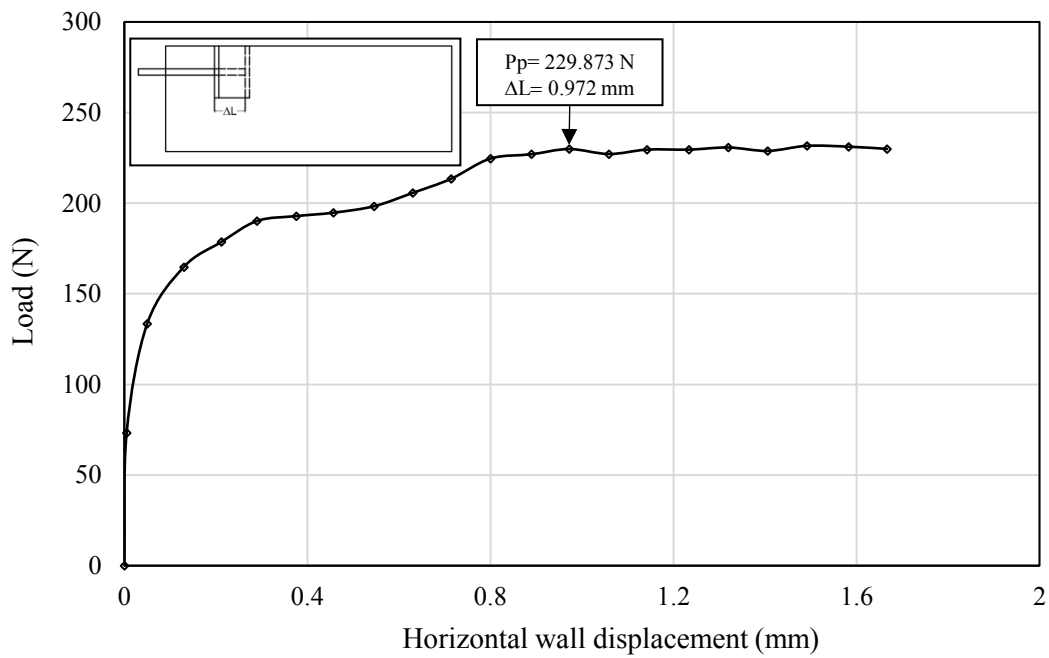
(b)



(c)

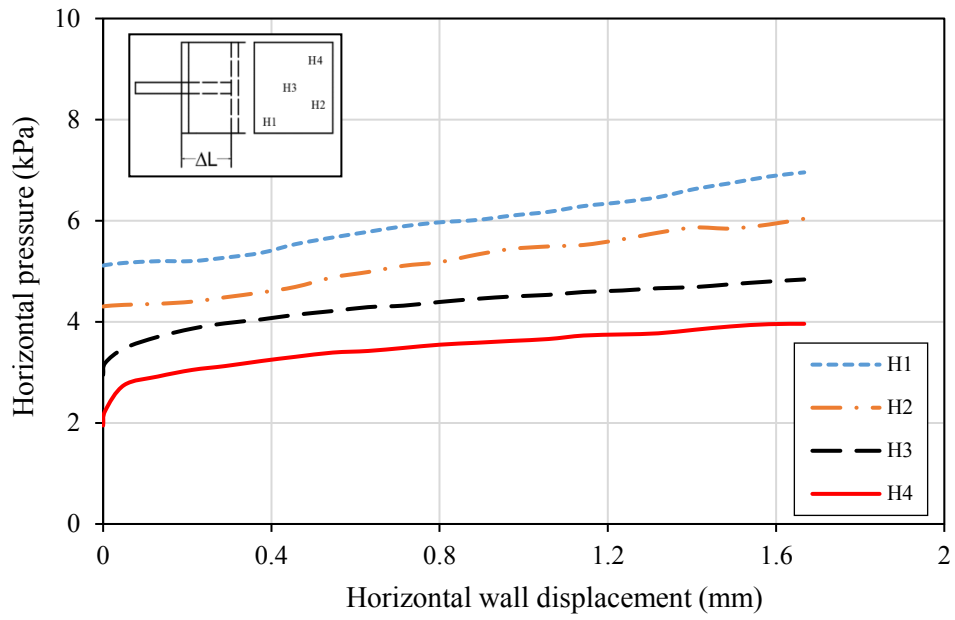
**Fig. 5.23.** Results for test no. 23 (Se3') for soil C –  $C_p=12.5\%$ : (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the soil mass versus the wall displacement

**Results for test no. 24**

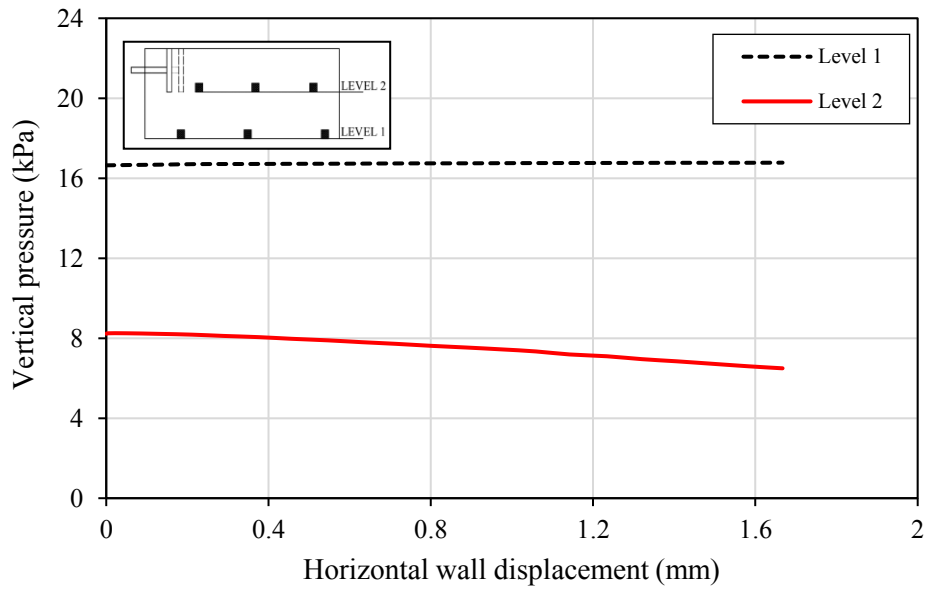


(a)





(b)

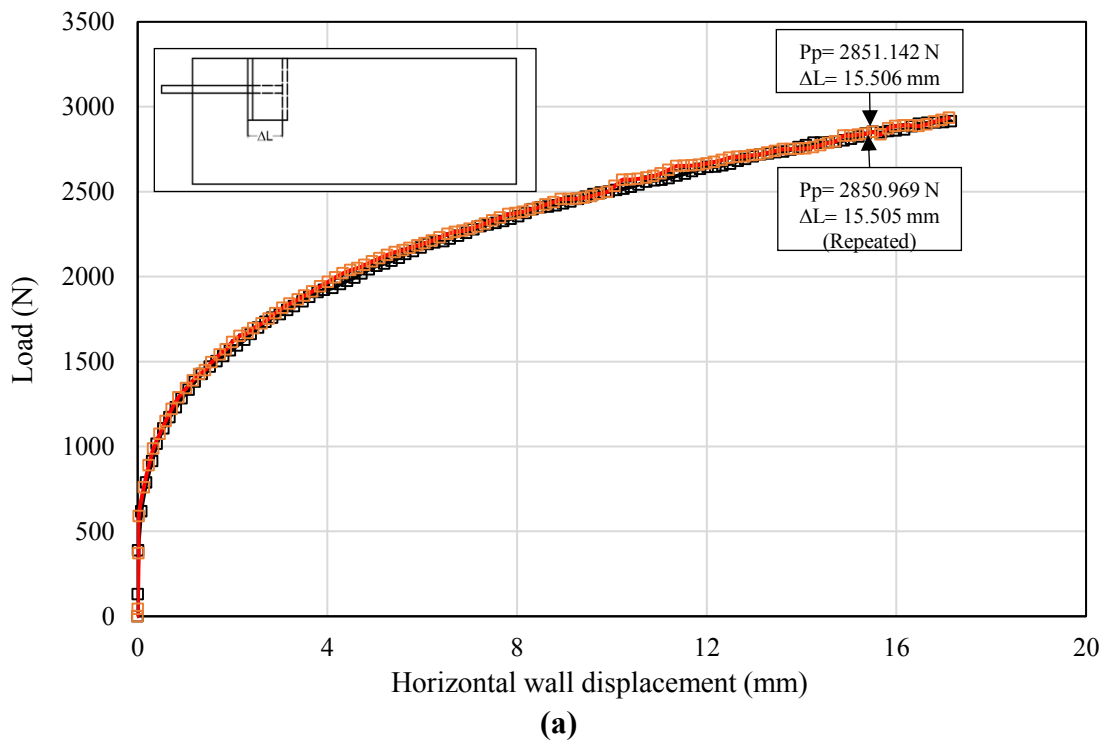


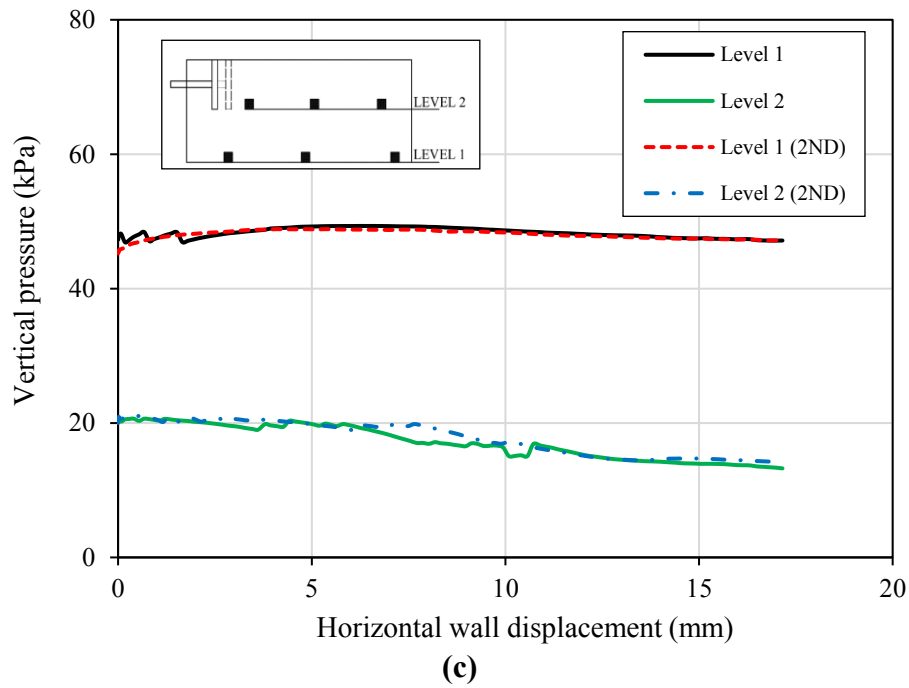
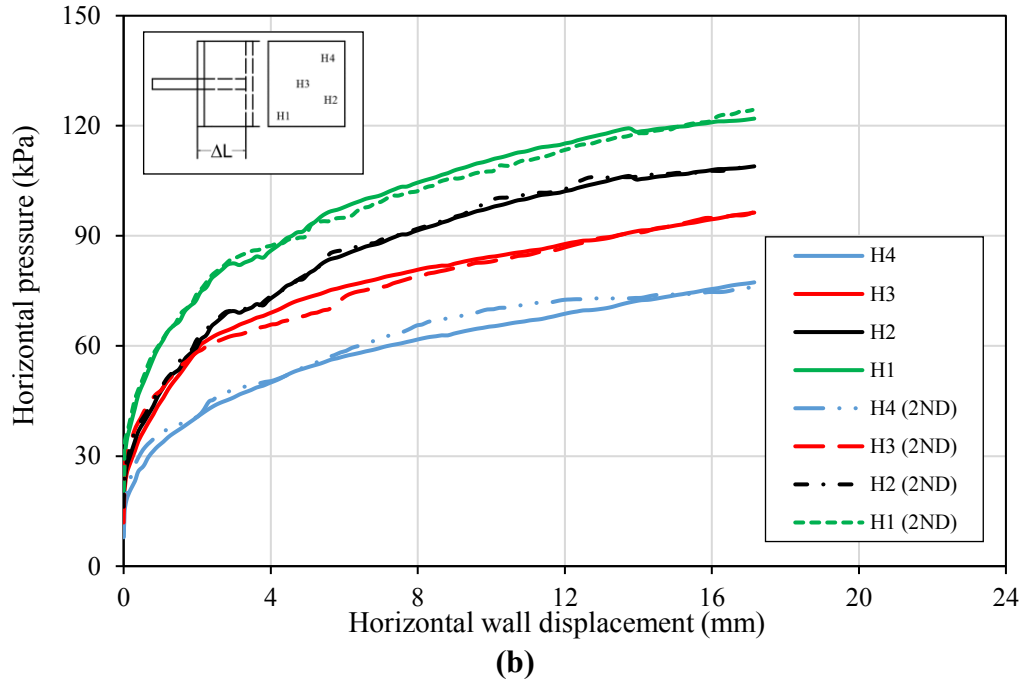
(c)

**Fig. 5.24.** Results for test no. 24 (Se3') for soil D –  $C_p=18\%$ : (a) Load – displacement curve (load cell reading); (b) Load – displacement curves for each transducer; (c) Vertical pressure in the soil mass versus the wall displacement

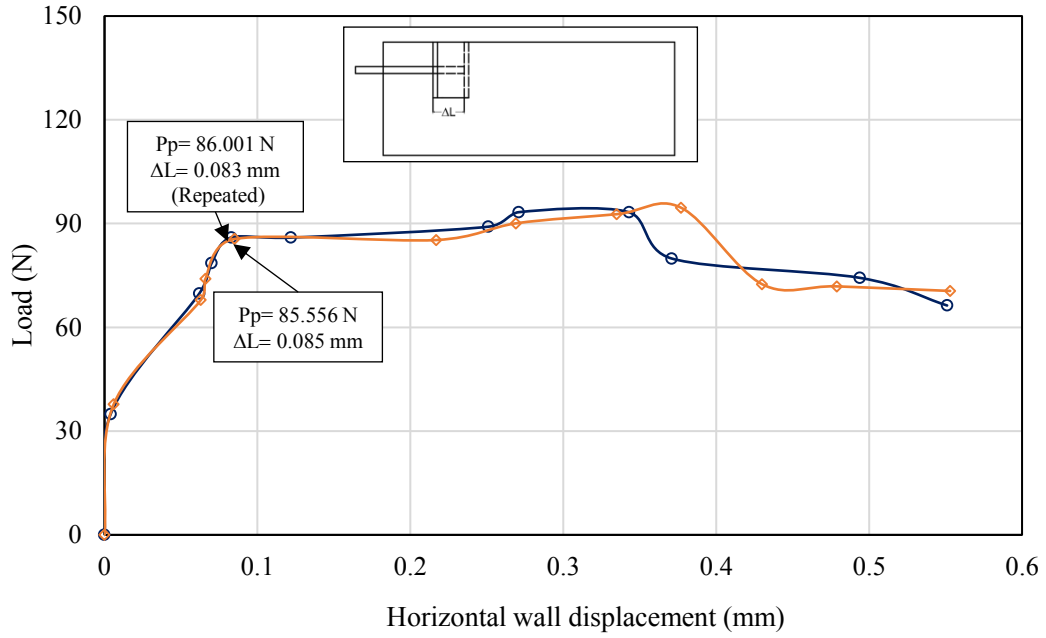
**Results from repeatability tests (for studying the repeatability test results):** These results are presented graphically in Figures 5.25 to 5.26.

The first set of repeated tests were performed on the dry soil. The second set of the repeated tests were conducted on saturated soil. The results of two tests (original and repeated) performed under the same conditions with the same soil were compared together. Figures 5.25 and 5.26 present the original and repeated passive earth pressure test results for the dry and saturated soil  $D - C_p = 18\%$ , respectively. It can be generally seen from these figures that the results obtained from two tests performed on the same soil under the same conditions are almost identical. Thus, the comparison shows acceptable consistency in the test results.

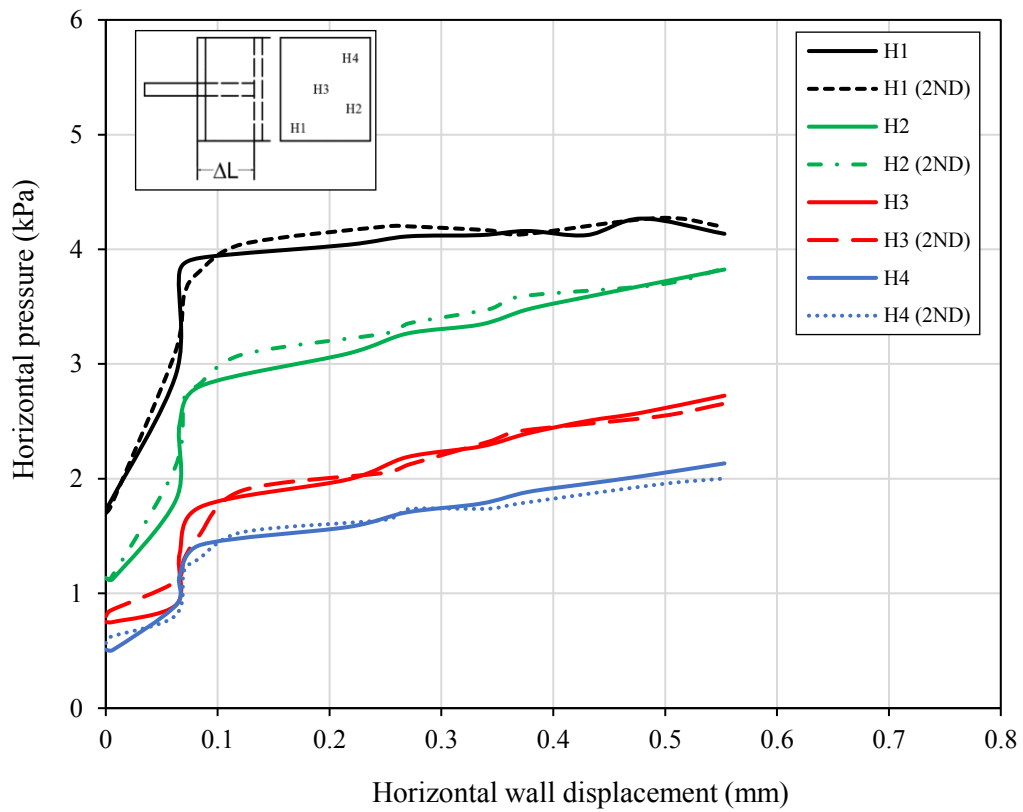




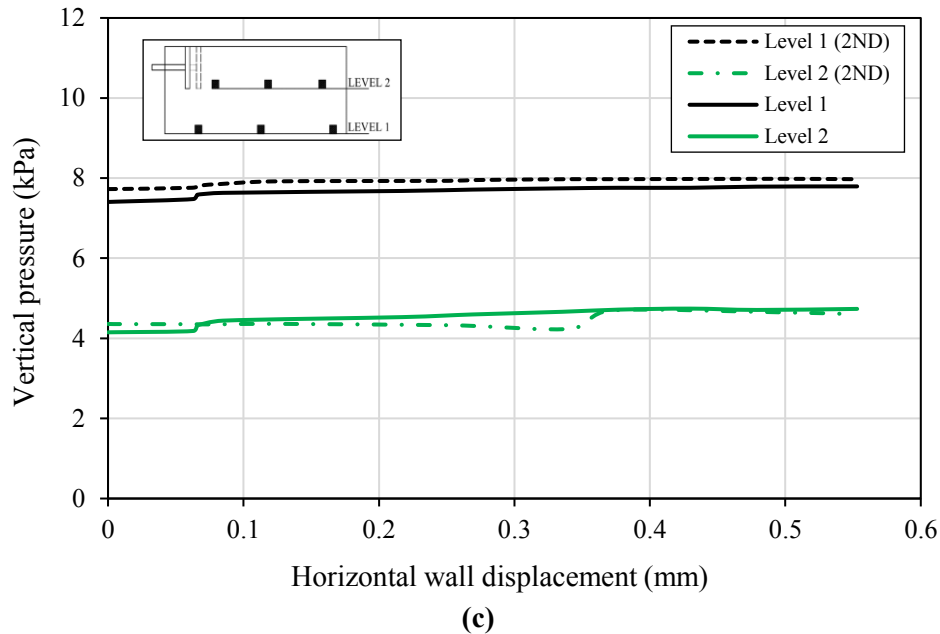
**Fig. 5.25.** Results from test no. 4 and test no. 4\* (2<sup>ND</sup>) performed on the dry soil D –  $C_p=18\%$ : (a) Load – displacement curve (load cell reading); (b) Load-displacement curve (transducers readings); (c) Vertical pressure in the soil mass versus horizontal wall displacement



(a)



(b)

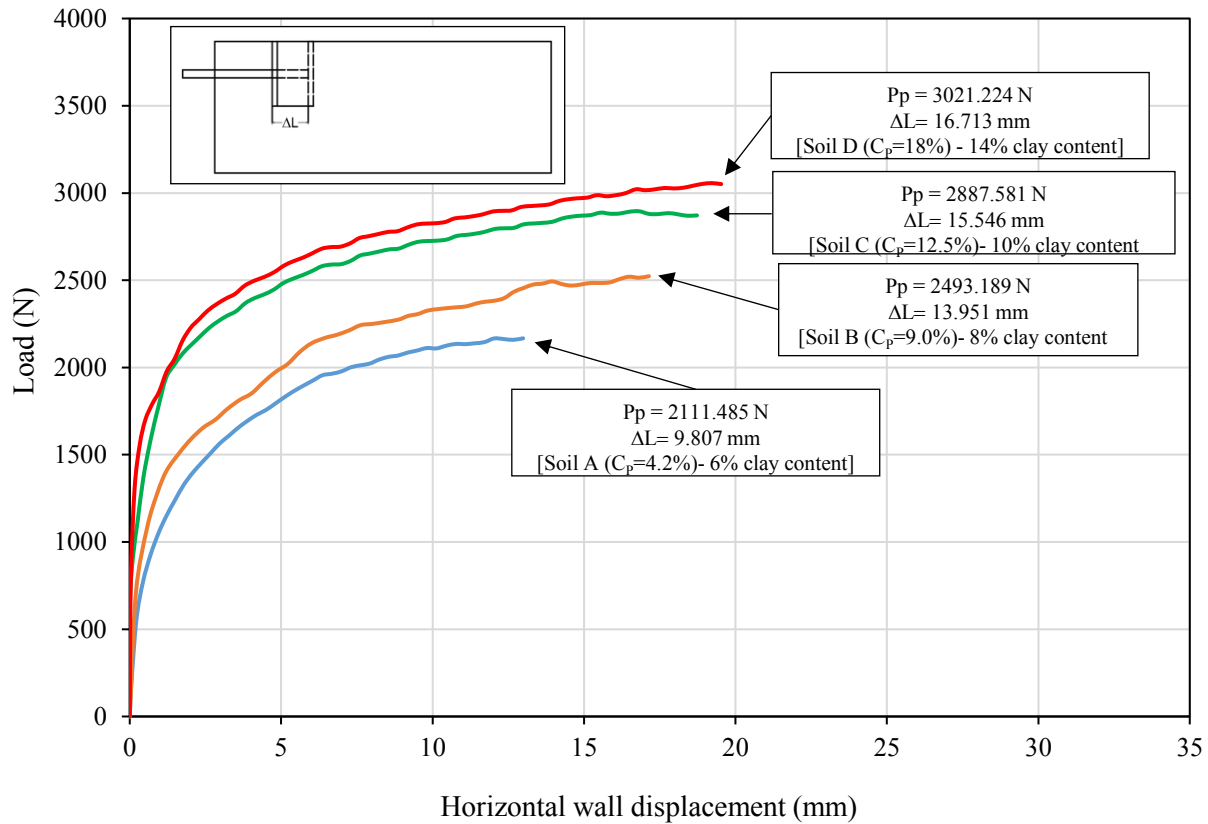


**Fig. 5.26.** Results from test no.16 and test no.16\* (2<sup>ND</sup>) performed on the saturated soil D –  $C_p=18\%$ : (a) Load – displacement curve (load cell reading); (b) Load-displacement curve (transducers readings); (c) Vertical pressure in the soil mass versus horizontal wall displacement

### 5.3. Parametric study

#### *Effect of clay content (collapse potential) on the passive earth pressure for dry collapsible soil*

Figure 5.27 shows the variation in the passive earth force with respect to the horizontal wall displacement for different soil mixtures with different clay content, accordingly different collapse potential. It can be seen that the passive earth force and the wall displacement at failure increase with the increase of the collapse potential  $C_p$  of the soil. Accordingly, increasing the clay content in the mixture will increase the passive resistance of the soil. This can be explained by the fact that the clay is acting as a bonding agent, which increases the strength for the retained soil. It is of interest to note from this figure that by increasing the clay content from 6% to 8%, the passive earth force increases about 18%. The increase of the passive earth force is approximately 43% when increasing clay content from 6% to 14%.

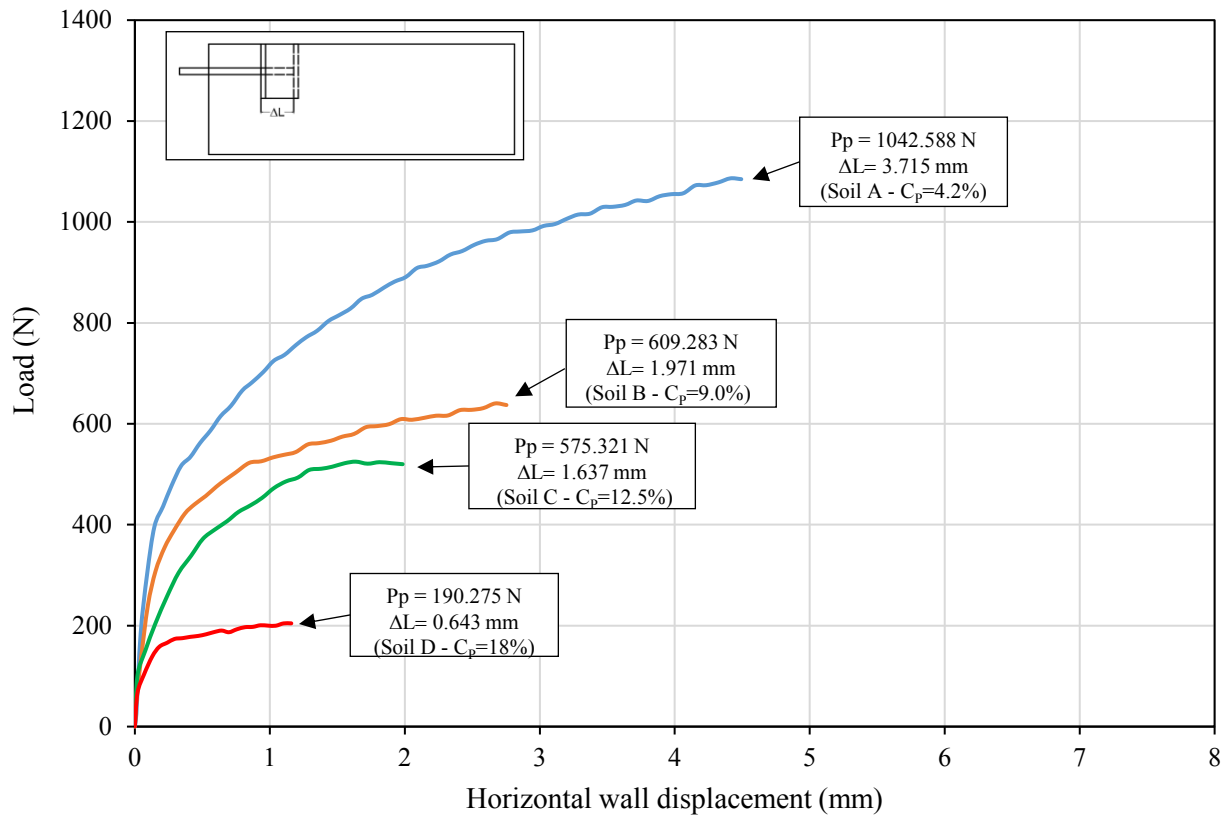


**Fig. 5.27.** Load – displacement curves (Se2) for dry collapsible soils with different clay content (different collapse potential)

***Effect of collapse potential on the passive earth pressure for saturated collapsible soil***

Figure 5.28 shows the variation in the passive earth force with respect to the horizontal wall displacement for different soil mixtures having different collapse potential  $C_p$  for the case of after full inundation. It can be observed that the passive earth force and horizontal wall displacement at failure decrease considerably with the increase of the collapse potential  $C_p$  of the soil in saturated condition. Increasing the collapse potential  $C_p$  of the soil results in the decrease of the passive resistance of the soil. It can be seen from this figure that by increasing the collapse potential  $C_p$  from 4.2% to 9.0%, the passive earth force decreases about 42%. The decrease of the passive earth force is around 82% when increasing the collapse potential  $C_p$  from 4.2% to 18%. Generally, for

the case of after full inundation, the higher the collapse potential of the soil, the larger the decrease in the passive earth force and the wall displacement at failure.

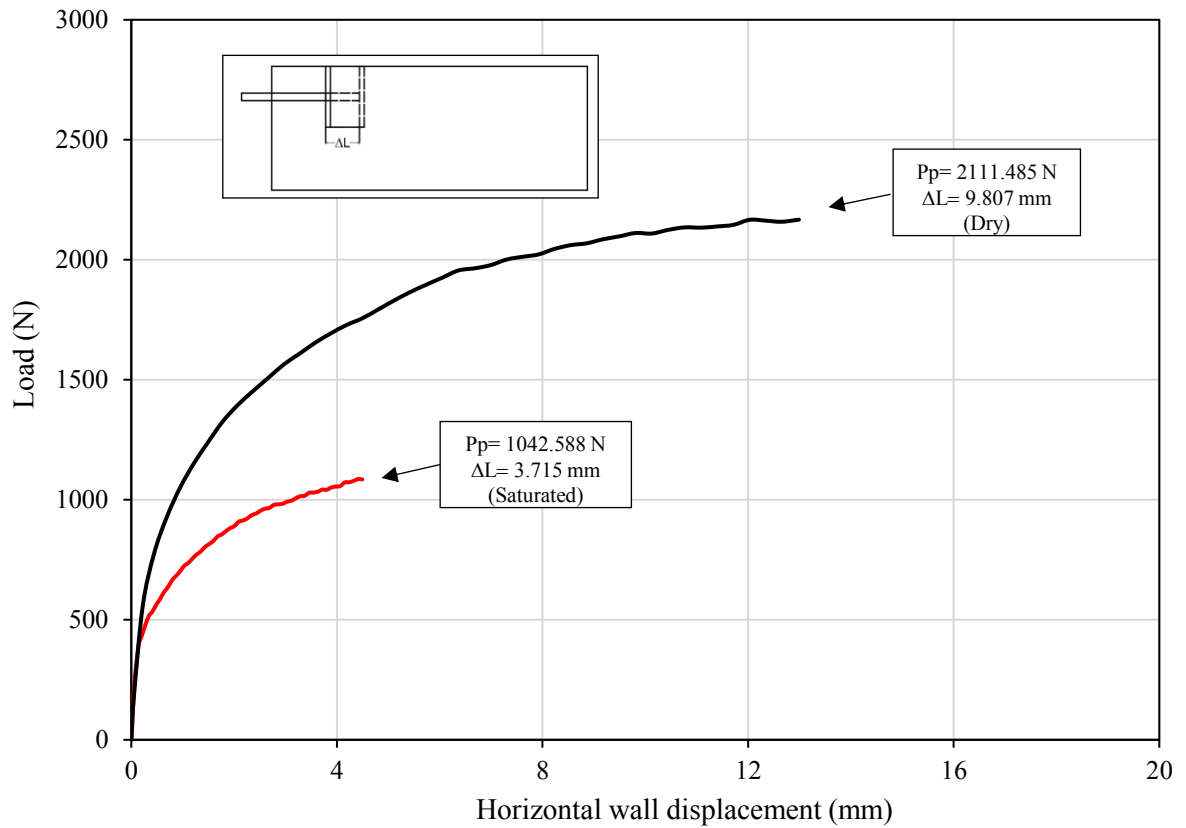


**Fig. 5.28.** Load – displacement curves (Se2') for saturated collapsible soils having different collapse potential

### ***Effect of inundation of collapsible soil on passive earth pressure***

In order to examine the effect of inundation on passive earth pressure of collapsible soils, load – displacement curves for the tests conducted on the same soil mixture under dry and saturated conditions are plotted in the same figure. Figure 5.29 presents the load-displacement curves for the tests performed on dry and saturated soil A -  $C_p = 4.2\%$  (tests no. 5 and 17). It can be observed from this figure that the passive earth force of the dry soil is reduced by 51% after inundation.

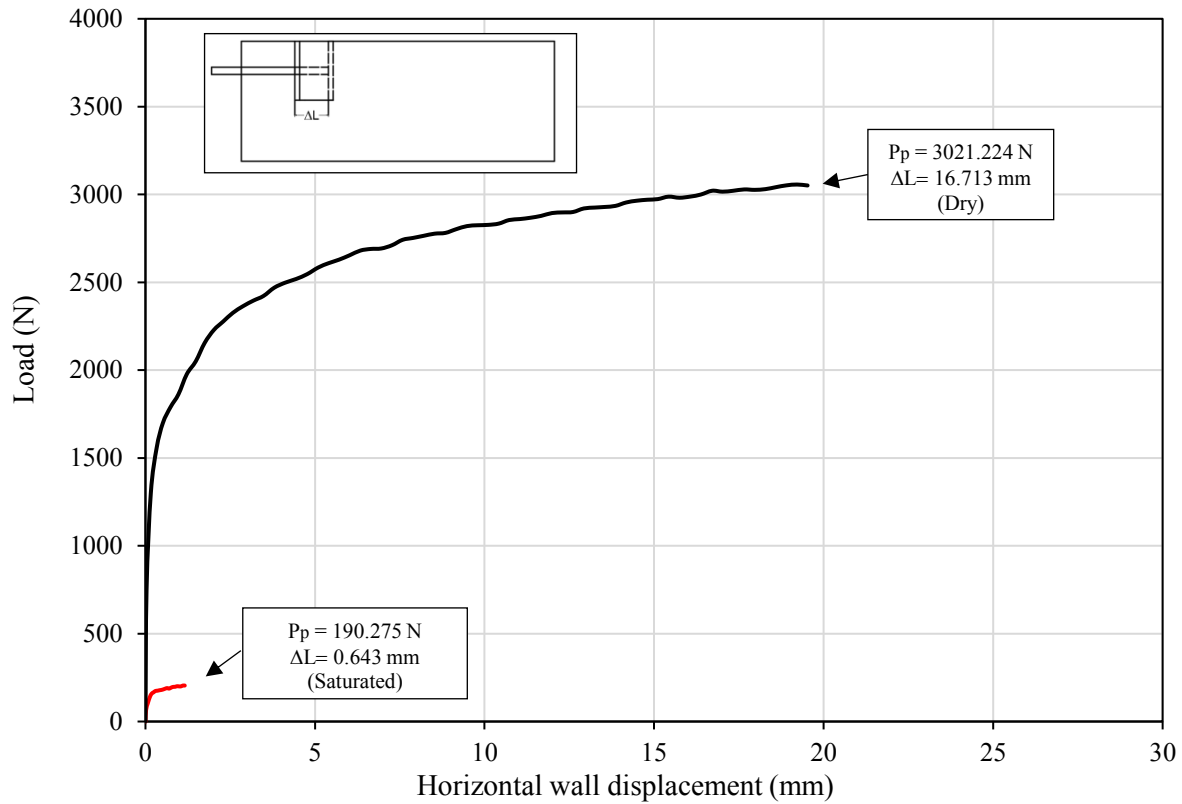
Similarly, the wall displacement at failure is decreased about 62% for the case of after full inundation.



**Fig. 5.29.** Load – displacement curves for dry and saturated collapsible soil A -  $C_p = 4.2\%$  (tests no. 5 and 17)

Figure 5.30 illustrates the load-displacement curves for the tests performed on dry and saturated soil D -  $C_p = 18\%$  (tests no. 8 and 20). It can be noted that the passive earth force of the dry soil is dropped by 94% after inundation. Similar observation is noted for the wall displacement. This can be explained by the fact that at full saturation, the soil loses most of its strength, which leads to premature failure at low value of wall displacement.





**Fig. 5.30.** Load – displacement curves for dry and saturated collapsible soil D -  $C_p = 18\%$  (tests no. 8 and 20)

It can be noted from Figures 5.29 and 5.30 that the passive earth pressure of collapsible soils are significantly affected by the inundation. Upon full inundated, the decrease of passive earth pressure of the soil A ( $C_p = 4.2\%$ ) is moderate. However, the decrease of passive earth pressure of the soil D ( $C_p = 18\%$ ) is very remarkable. It can be firstly concluded that the higher the collapse potential, the larger the decrease in passive earth pressure of collapsible soils under inundation condition.

In this investigation, the experimental values of the earth force  $P_p$  acting on the wall were used to determine the coefficient of passive earth pressure  $K_p$  for the dry and saturated collapsible soils, as follows:

$$P_{p(dry)} = \frac{1}{2} K_p \gamma' b H^2 + 2c' b H \sqrt{K_p} \quad (5.1)$$

$$P_{p(saturated)} = \frac{1}{2} K_p \gamma' b H^2 + 2c' b H \sqrt{K_p} + \frac{1}{2} \gamma_w b H^2 \quad (5.2)$$

Where

$P_p$  = passive earth force at the failure point measured by the load cell;

$\gamma'$  = effective unit weight of the soil;

$\gamma_w$  = unit weight of water;

$c'$  = effective cohesion of the soil;

$H$  = the height of the wall;

$b$  = the width of the wall.

Table 5.1 presents a summary of the test results of the present experimental investigation for passive case.

It can be seen from this table that the coefficient of passive earth pressure  $K_p$  of dry collapsible soils decreases gradually by increasing the collapse potential  $C_p$ , and increases considerably with the increase of the OCR; while at full saturation, it decreases with the increase of the collapse potential  $C_p$  of the soil and increases with the increase of the OCR.

In general, at full saturation, the higher the collapse potential  $C_p$  the larger the decrease in coefficient of passive earth pressure  $K_p$ .

**Table 5.1.** Summary of the test results of passive earth pressure

Test No.	Soil type	Test series	Testing condition	OCR	P <sub>p</sub> (N)	K <sub>p</sub>
1	A (C <sub>p</sub> = 4.2%)	Se1	Dry soil	3.3	1839.329	4.15
2	B (C <sub>p</sub> = 9.0%)		Dry soil	3.9	2255.907	3.61
3	C (C <sub>p</sub> = 12.5%)		Dry soil	5.5	2659.729	3.45
4	D (C <sub>p</sub> = 18%)		Dry soil	7.2	2851.142	3.10
5	A (C <sub>p</sub> = 4.2%)	Se2	Dry soil	4.5	2111.485	5.14
6	B (C <sub>p</sub> = 9.0%)		Dry soil	5.1	2493.189	4.31
7	C (C <sub>p</sub> = 12.5%)		Dry soil	6.9	2887.581	4.00
8	D (C <sub>p</sub> = 18%)		Dry soil	9.9	3021.224	3.40
9	A (C <sub>p</sub> = 4.2%)	Se3	Dry soil	6.0	2368.027	6.25
10	B (C <sub>p</sub> = 9.0%)		Dry soil	6.6	2791.205	5.20
11	C (C <sub>p</sub> = 12.5%)		Dry soil	8.2	3286.274	4.80
12	D (C <sub>p</sub> = 18%)		Dry soil	11.7	3323.064	4.06
13	A (C <sub>p</sub> = 4.2%)	Se1'	Saturated soil	2.8	892.937	1.05
14	B (C <sub>p</sub> = 9.0%)		Saturated soil	3.3	525.207	0.20
15	C (C <sub>p</sub> = 12.5%)		Saturated soil	4.7	500.101	0.149
16	D (C <sub>p</sub> = 18%)		Saturated soil	1.1	85.556	0.001
17	A (C <sub>p</sub> = 4.2%)	Se2'	Saturated soil	3.7	1042.588	1.41
18	B (C <sub>p</sub> = 9.0%)		Saturated soil	4.1	609.283	0.27
19	C (C <sub>p</sub> = 12.5%)		Saturated soil	5.9	575.321	0.20
20	D (C <sub>p</sub> = 18%)		Saturated soil	1.8	190.275	0.01
21	A (C <sub>p</sub> = 4.2%)	Se3'	Saturated soil	4.9	1291.083	1.91
22	B (C <sub>p</sub> = 9.0%)		Saturated soil	5.7	818.961	0.37
23	C (C <sub>p</sub> = 12.5%)		Saturated soil	7.0	600.651	0.29
24	D (C <sub>p</sub> = 18%)		Saturated soil	2.7	229.873	0.015

#### 5.4. Analytical model

Analytical model using limit equilibrium method was developed to simulate the case of a vertical wall retaining overconsolidated collapsible soil, as shown in Figure 5.31.

The model uses the constitutive law of Mohr-Coulomb criterion and Spencer method of analysis. The Spencer method considers both shear and normal inter-slice forces, and assumes a constant inter-slice force function. The Spencer factor of safety satisfies all conditions of equilibrium, including moment and force equilibrium. The Spencer method works well for any shape of slip surface.

The model operates with the angle of shearing resistance ( $\phi_{cr}'$ ), cohesion ( $c'$ ), and the unit weight ( $\gamma'$ ). The material properties of the collapsible soil used in the experimental investigation (Table 3.4) was used as an input data set for this model.

In this analysis, a line load representing the passive earth force was applied at 1/3 of the wall.

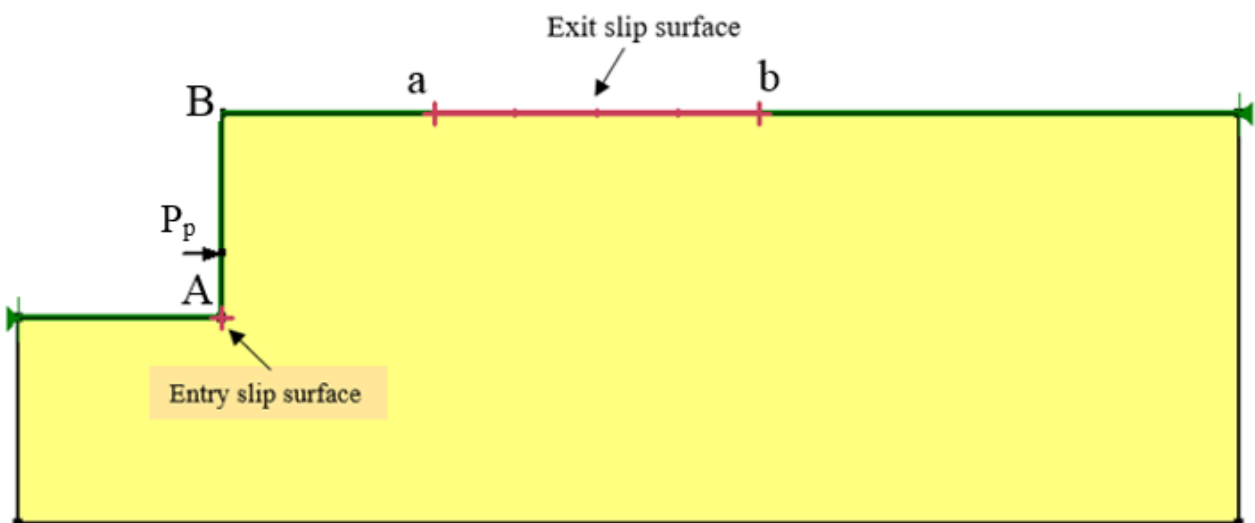


Fig. 5.31. Geometry of the model used

After obtaining the load on the wall at failure, the analysis proceeded by trial and error to find the critical slip surface, which could produce the experimental value of the passive earth force, and accordingly the factor of safety being about one.

It should be reported herein that based on the limit equilibrium method of analysis, the factor of safety is the ratio between the forces/moments resisting (R) movement and the forces/moments motivating (M) movement.

$$FS = \frac{\sum R}{\sum M} \quad (5.3)$$

In performing these trials, the entry point of the slip surface was assigned at the toe of the wall (A), while the exit points were taken within the range of points (a) to (b) as shown in Figure 5.31. Point (a) was taken at a distance from the wall equal to the wall height, while point (b) was taken at a distance from the wall equal four times of the wall height.

This range was wisely chosen to cover all possible critical slip surface. About 20 trials were performed for each soil type to search for the critical slip surface. The program “GeoStudio” was used to perform the analysis using planar and curved surfaces in order to determine the critical slip surface.

Figures 5.32 to 5.35 present results of all the trial slip surfaces, including the critical slip surface (accordingly the factor of safety of about one), for soils A, B, C and D, respectively.

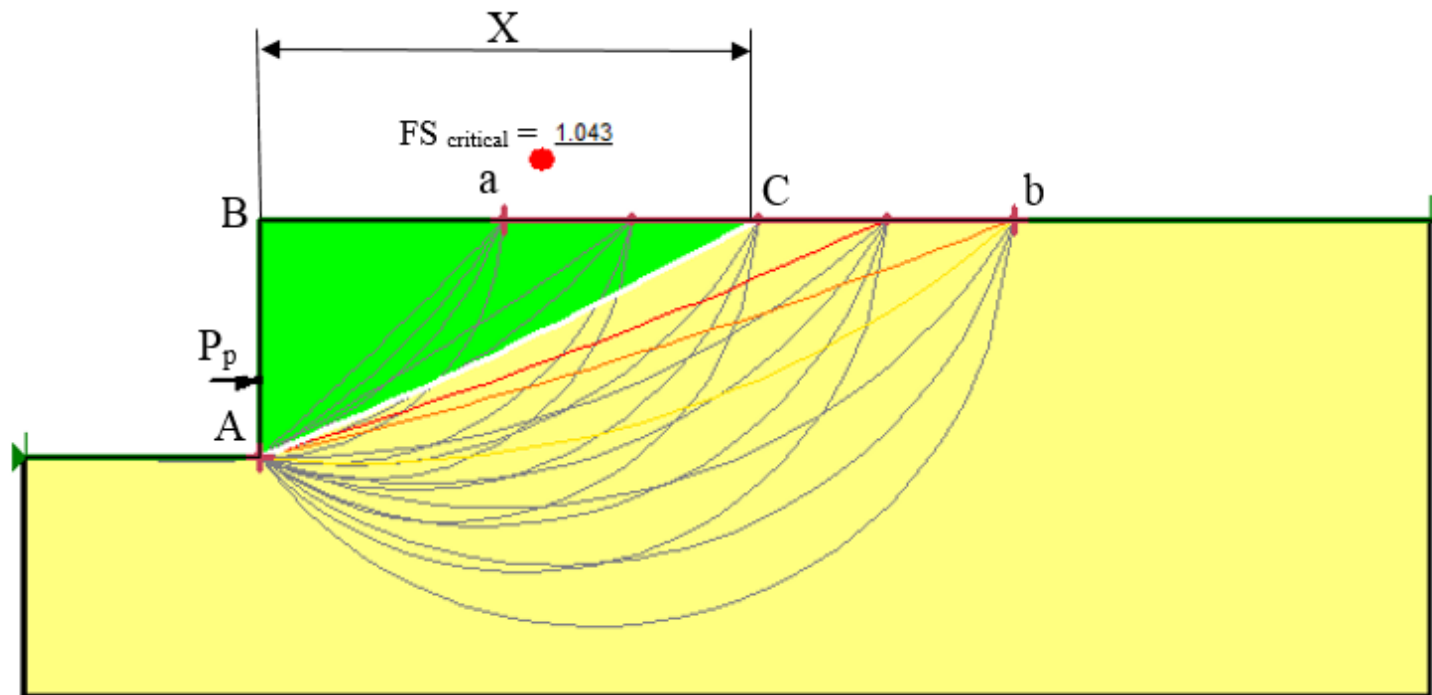


Fig. 5.32. Trial slip surfaces for soil A

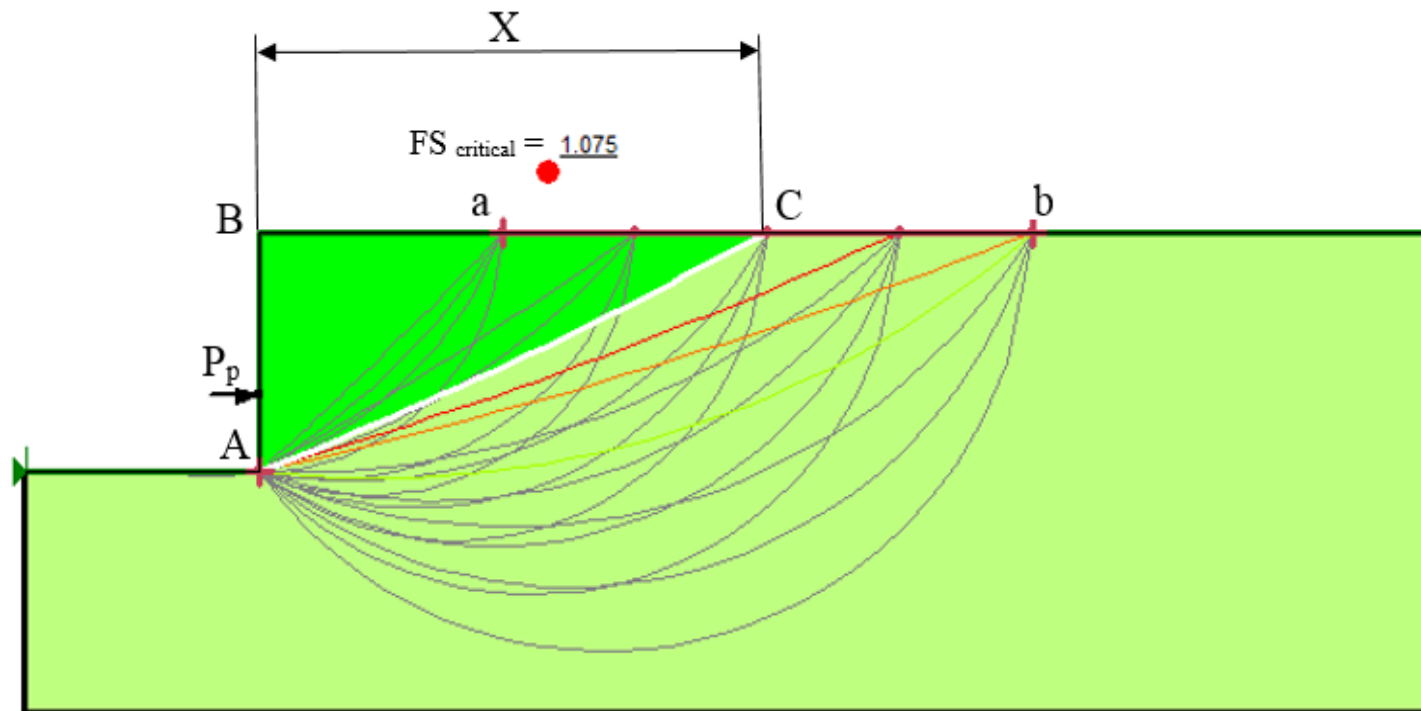


Fig. 5.33. Trial slip surfaces for soil B

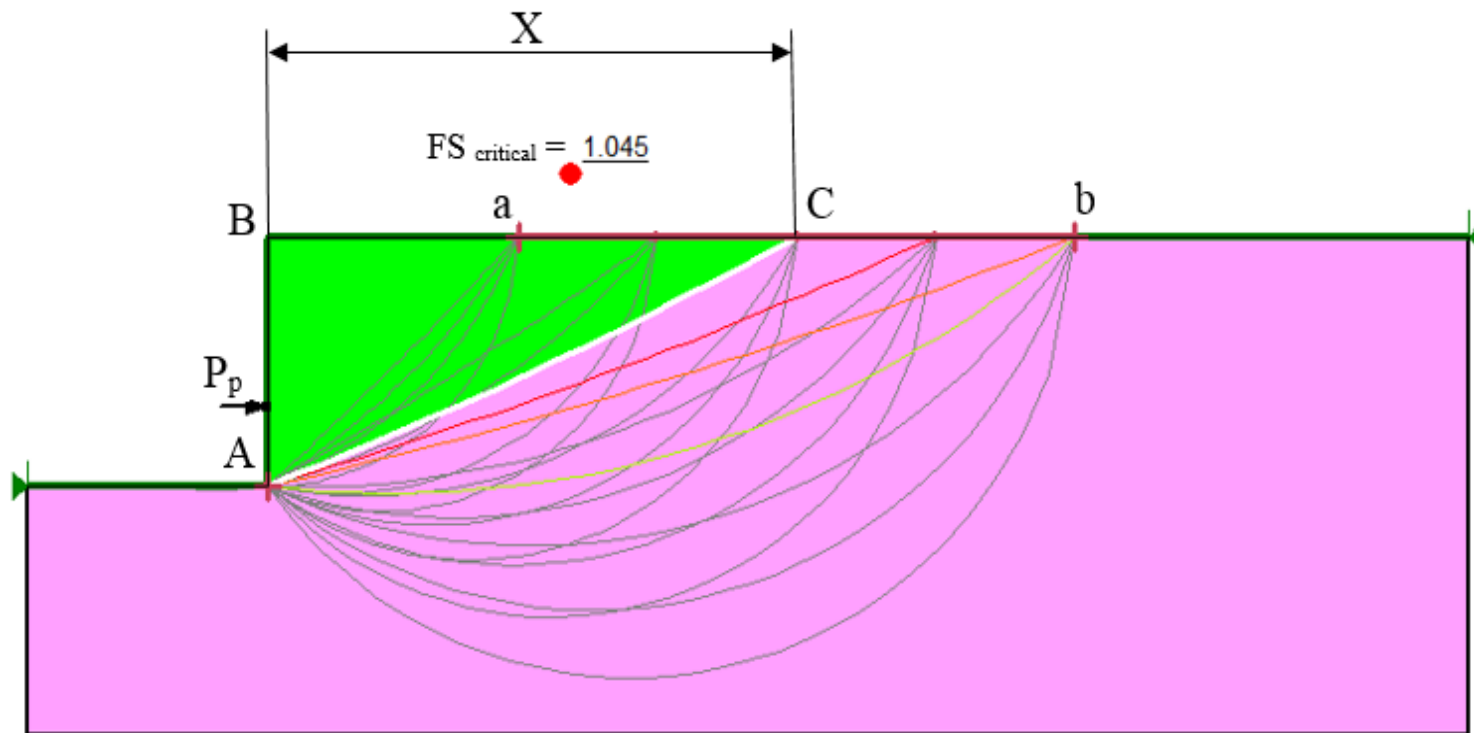


Fig. 5.34. Trial slip surfaces for soil C



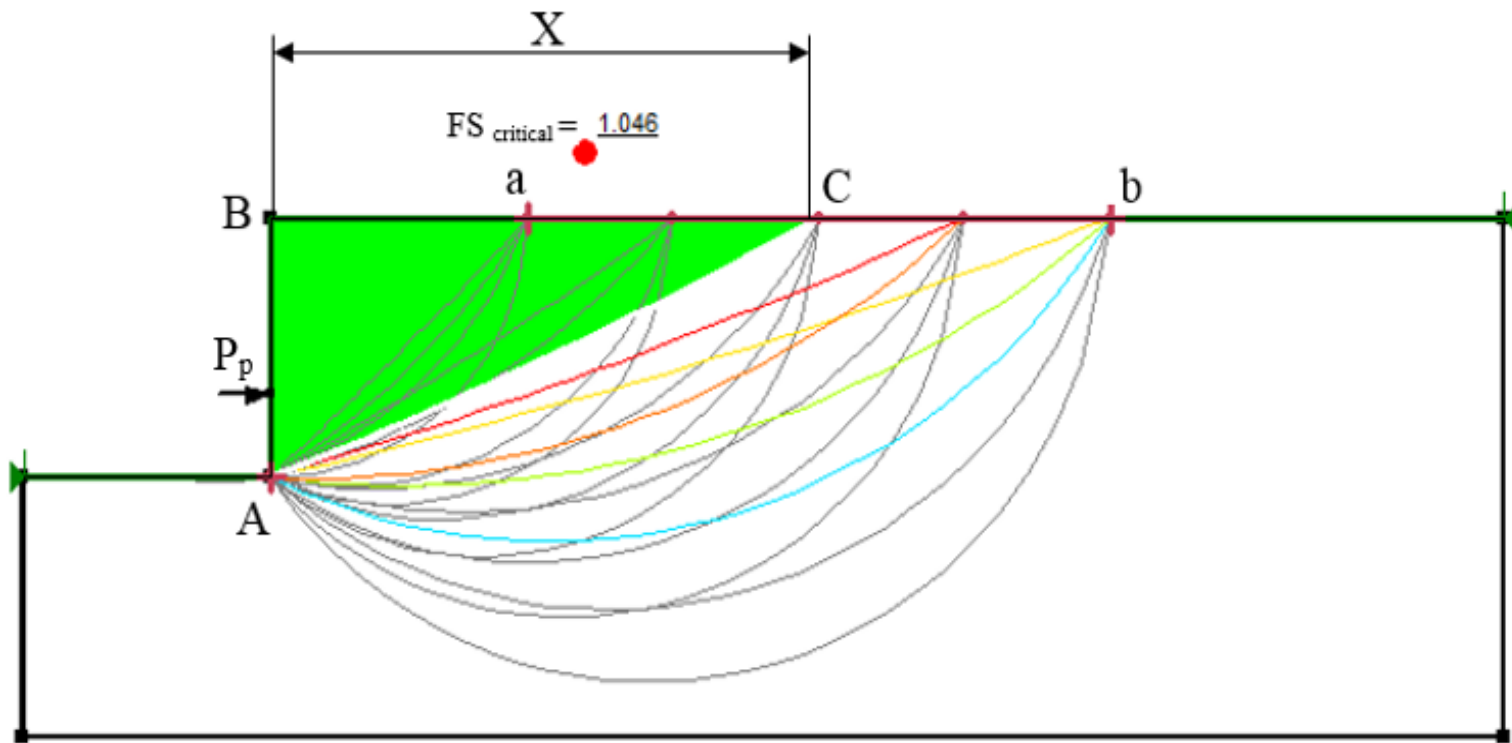


Fig. 5.35. Trial slip surfaces for soil D

Tables 5.2 to 5.5 present the results of this analysis performed on samples A, B, C and D respectively, and accordingly, the critical slip surfaces of each sample were identified.

It can be noted that the factor of safety of about one for all samples tested in this investigation was determined by planar slip surface. Therefore, the planar slip surface is the critical slip surface. Furthermore, the factor of safety increases with increasing the distance “X”, which reflect on the angle ( $\alpha$ ), defined as the slope of the slip surface with the horizontal. It is of interest to note that with the same value of “X”, the curved slip surfaces give higher factor of safety than that of planar slip surfaces.

**Table 5.2.** Summary of the trials considered for soil A ( $C_p = 4.2\%$ )

Trial slip #	Shape of trial slip	X (m) Distance from the wall to the exit point of the slip surface	$\alpha$ ( $^\circ$ ) angle of inclination of the slip surface	FS Factor of safety
1	Planar	0.69	17.574	1.163
2		0.57	21.045	1.108
<b>3</b>		<b>0.45</b>	<b>26.000</b>	<b>1.043</b>
4		0.35	32.152	0.559
5		0.22	45.258	0.497
6	Curved	0.57	-0.512	1.726
7		0.45	-4.967	1.321
8		0.45	-9.1879	1.545
9		0.35	-10.376	1.185
10		0.57	-16.686	2.138
11		0.35	-21.808	1.336
12		0.69	-21.967	2.824
13		0.45	-22.999	1.811
14		0.57	-31.311	2.573
15		0.45	-35.324	2.093
16		0.69	-37.118	3.475
17		0.57	-44.082	3.038
18		0.69	-50.109	4.176

**Table 5.3.** Summary of the trials considered for soil B ( $C_p = 9.0\%$ )

Trial slip #	Shape of trial slip	X (m) Distance from the wall to the exit point of the slip surface	$\alpha$ ( $^{\circ}$ ) angle of inclination of the slip surface	FS Factor of safety
1	Planar	0.71	17.055	1.229
2		0.60	20.002	1.137
<b>3</b>		<b>0.47</b>	<b>25.012</b>	<b>1.075</b>
4		0.36	31.429	0.564
5		0.22	44.889	0.501
6	Curved	0.60	-1.155	1.798
7		0.47	-5.606	1.384
8		0.47	-9.928	1.603
9		0.36	-11.026	1.218
10		0.60	-17.445	2.236
11		0.36	-22.509	1.374
12		0.71	-22.697	2.971
13		0.47	-23.823	1.884
14		0.60	-32.149	2.699
15		0.47	-36.198	2.182
16		0.71	-37.917	3.668
17		0.60	-44.957	3.195
18		0.71	-50.931	4.419

**Table 5.4.** Summary of the trials considered for soil C ( $C_p = 12.5\%$ )

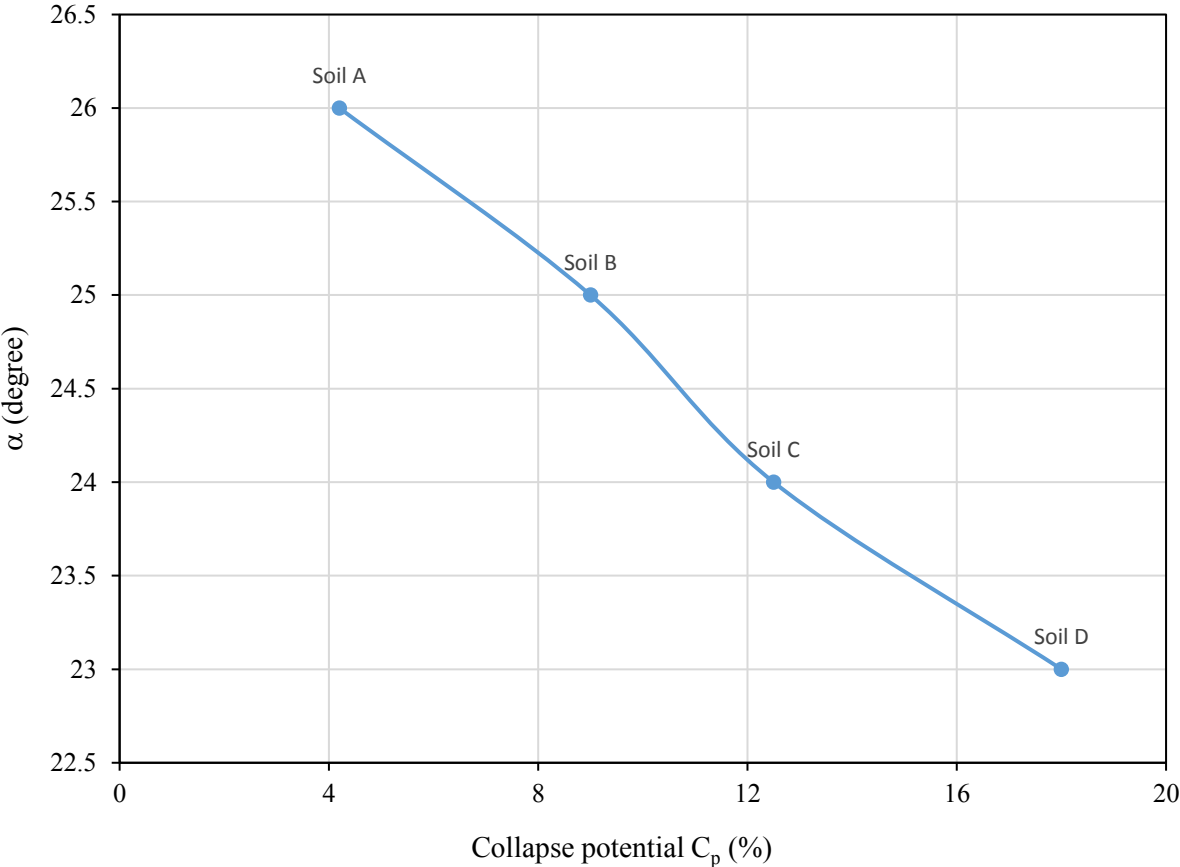
Trial slip #	Shape of trial slip	X (m) Distance from the wall to the exit point of the slip surface	$\alpha$ ( $^\circ$ ) angle of inclination of the slip surface	FS Factor of safety
1	Planar	0.73	16.625	1.225
2		0.61	19.830	1.119
<b>3</b>		<b>0.49</b>	<b>24.100</b>	<b>1.045</b>
4		0.37	30.735	0.759
5		0.23	44.131	0.540
6	Curved	0.61	-1.466	1.758
7		0.49	-5.904	1.367
8		0.49	-10.29	1.564
9		0.37	-11.346	1.183
10		0.61	-17.813	2.189
11		0.37	-22.991	1.325
12		0.73	-23.049	2.917
13		0.49	-24.225	1.840
14		0.61	-32.554	2.647
15		0.49	-36.623	2.133
16		0.73	-38.301	3.608
17		0.61	-45.379	3.136
18		0.73	-51.325	4.352

**Table 5.5.** Summary of the trials considered for soil D ( $C_p = 18\%$ )

Trial slip #	Shape of trial slip	X (m) Distance from the wall to the exit point of the slip surface	$\alpha$ ( $^\circ$ ) angle of inclination of the slip surface	FS Factor of safety
1	Planar	0.75	16.350	1.262
2		0.64	18.924	1.139
<b>3</b>		<b>0.52</b>	<b>23.00</b>	<b>1.046</b>
4		0.39	29.695	0.757
5		0.24	42.851	0.477
6	Curved	0.64	-1.771	1.234
7		0.75	-6.195	1.396
8		0.52	-10.645	1.645
9		0.39	-11.663	1.811
10		0.64	-18.173	2.208
11		0.39	-23.195	1.336
12		0.75	-23.392	1.801
13		0.52	-24.62	1.851
14		0.64	-32.951	2.673
15		0.75	-38.676	3.653
16		0.64	-45.792	3.171
17	0.75	-51.71	4.413	

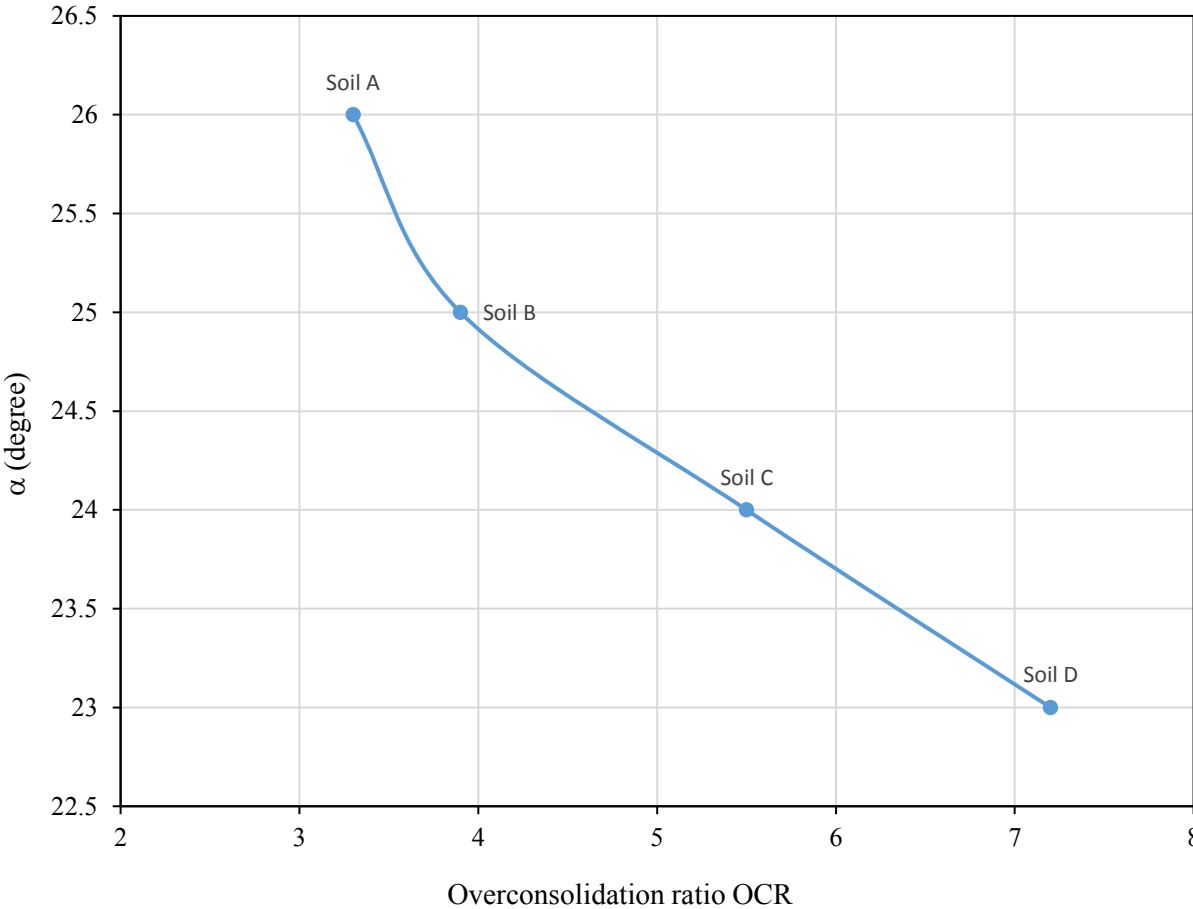
Furthermore, the deduced angle of inclination ( $\alpha$ ) of the failure plane to the horizontal versus the collapse potentials  $C_p$  of the soil samples is presented in Figure 5.36.

It can be noted from this figure that the angle ( $\alpha$ ) decreases due to the increase of the collapse potential  $C_p$ .



**Fig. 5.36.** Analytical results: the angle ( $\alpha$ ) versus the collapse potential  $C_p$  ( $P_s = 0$ )

Figure 5.37 presents the deduced angle ( $\alpha$ ) with the corresponding overconsolidation ratio OCR values, where the angle ( $\alpha$ ) decreases due to the increase of the overconsolidation ratio OCR.



**Fig. 5.37.** Analytical results: the angle ( $\alpha$ ) versus OCR ( $P_s = 0$ )

Figure 5.38 presents the deduced slip surface at failure from the analytical model. The failing wedge was analyzed using limit equilibrium method of analysis to determine the passive earth pressure for all samples tested, as shown in Figure 5.39. Thus

$$\sum F_x = 0$$

$$E + dE - E - R \sin(\alpha + \phi'_{cr}) + c' L \cos \alpha = 0 \quad (5.4)$$

Or

$$dE = R \sin(\alpha + \phi'_{cr}) + c' L \cos \alpha \quad (5.5)$$

Where:

$\alpha$  = Angle of inclination of the failure plane with the horizontal

$\phi'_{cr}$  = Critical state friction angle of the backfill soil;

$c'$  = Cohesion of the backfill soil;

$R$  = Resultant of the tangential and normal forces on the failure plane;

$E$  and  $E + dE$  = horizontal forces on the element;

$L$  = element length;

$W$  = element weight.

Furthermore,

$$\sum F_y = 0$$



$$W + X + dX - X - R \cos(\alpha + \phi'_{cr}) + c' L \sin \alpha = 0 \quad (5.6)$$

Where: X and X + dX = shear forces acting on the sides of the element

Therefore

$$W + dX + c' L \sin \alpha = R \cos(\alpha + \phi'_{cr}) \quad (5.7)$$

Or

$$R = \frac{W + dX + c' L \sin \alpha}{\cos(\alpha + \phi'_{cr})} \quad (5.8)$$

Substituting R from Eq. (5.8) into Eq. (5.5):

$$dE = W \tan(\alpha + \phi'_{cr}) + dX \tan(\alpha + \phi'_{cr}) + c' L \sin \alpha \tan(\alpha + \phi'_{cr}) + c' L \cos \alpha \quad (5.9)$$

The change of the vertical shear force across an element, dX, was equated to zero (Bishop, 1955).

Therefore, the passive earth force of this element:

$$dE = W \tan(\alpha + \phi'_{cr}) + c' L [\sin \alpha \tan(\alpha + \phi'_{cr}) + \cos \alpha] \quad (5.10)$$

Accordingly,

The passive earth force  $P_p$  of the failure wedge can be estimated as:

$$P_p = W_{wedge} \tan(\alpha + \phi'_{cr}) + c' L_{wedge} [\sin \alpha \tan(\alpha + \phi'_{cr}) + \cos \alpha] \quad (5.11)$$

In which:  $W_{wedge}$  = weight of the failure wedge,  $L_{wedge}$  = the length of the failure surface of the wedge.

And

$$P_p = 0.5\gamma H^2 K_p + (K_p - 1) \frac{c' H}{\tan \phi'} \quad (\text{Cai et al., 2016}) \quad (5.12)$$

Therefore, the coefficient of passive earth pressure  $K_p$ :

$$K_p = \frac{P_p \tan \phi'_{cr} + c' H}{0.5\gamma' H^2 \tan \phi'_{cr} + c' H} \quad (5.13)$$

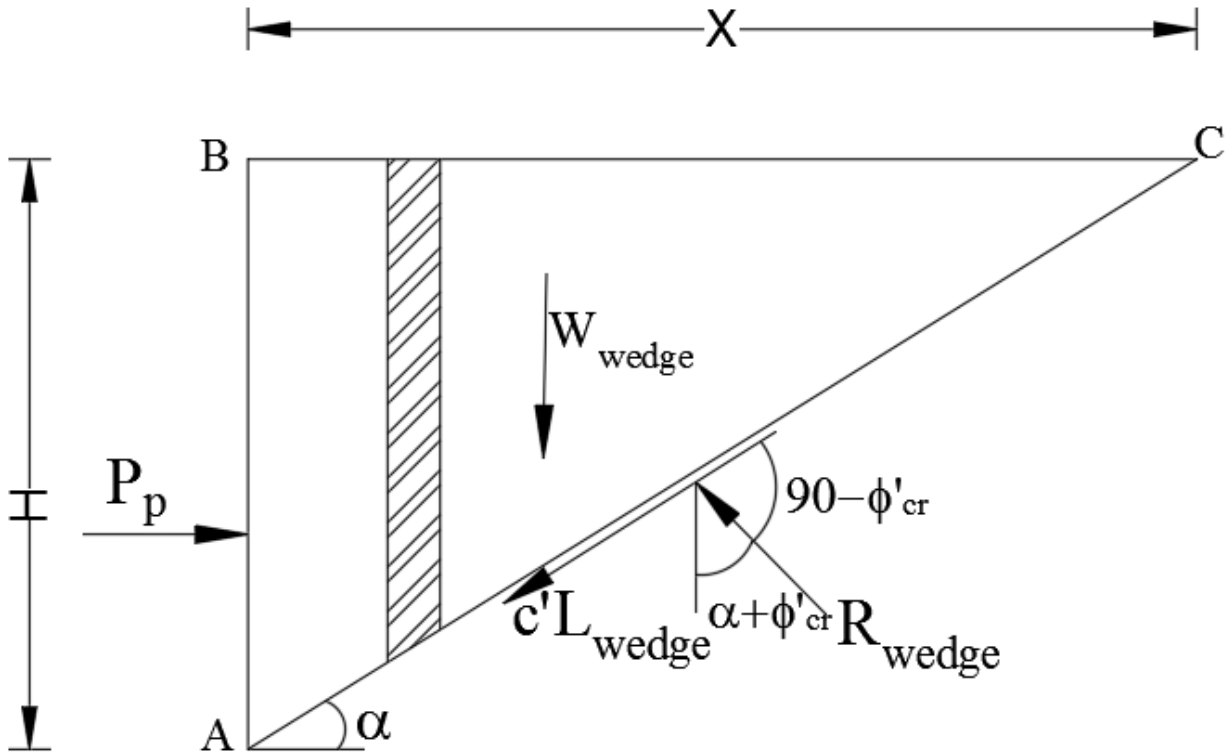
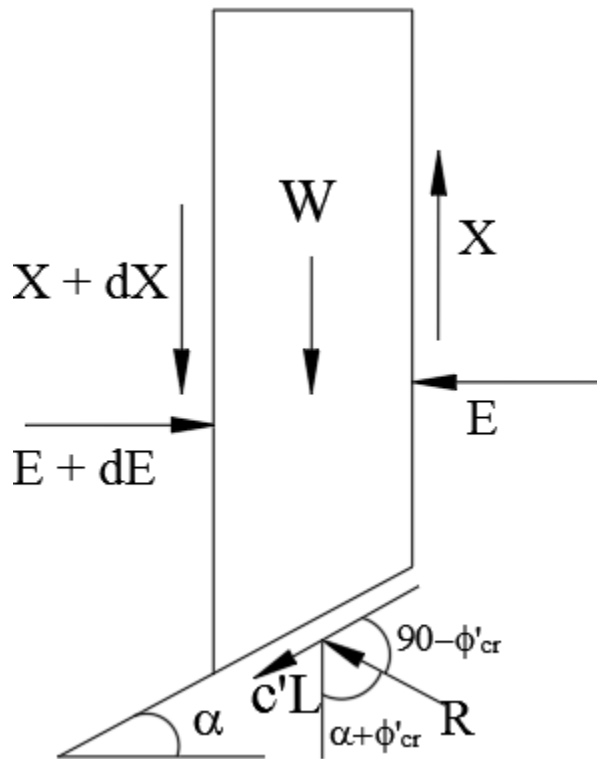


Fig. 5.38. Failure wedge with planar surface



**Fig.5.39.** Forces acting on typical element of failure wedge

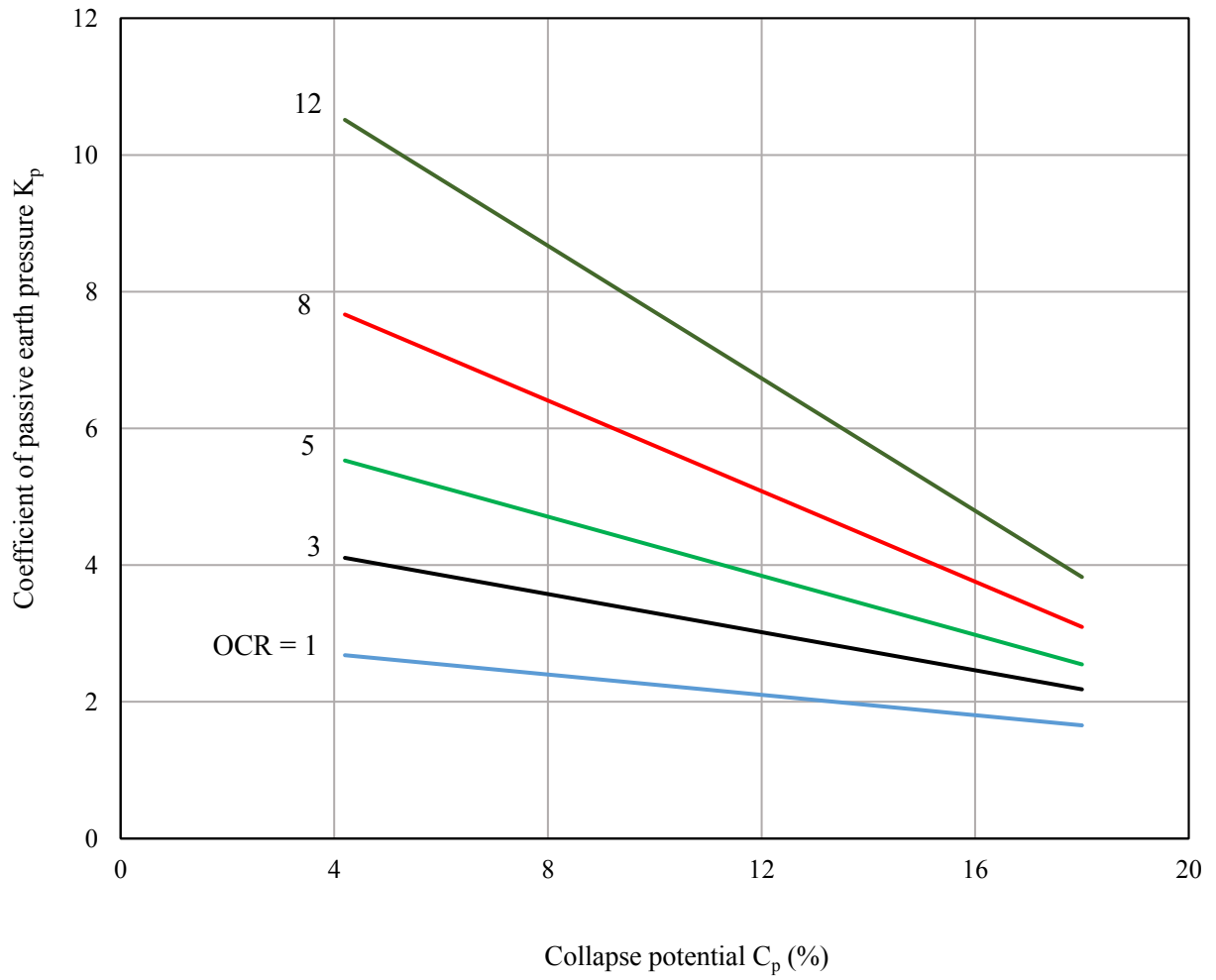
The proposed analytical model [Eq. (5.11) and Eq. (5.13)] was validated using the experimental results for dry collapsible soils. Table 5.6 presents the results of this analysis together with the experimental values of the passive earth force  $P_p$  and the coefficient of passive earth pressure  $K_p$ , where good agreement can be noted.

**Table 5.6.** Comparison between the predicted and the experimental values of the passive earth force  $P_p$  and the coefficient of passive earth pressure  $K_p$

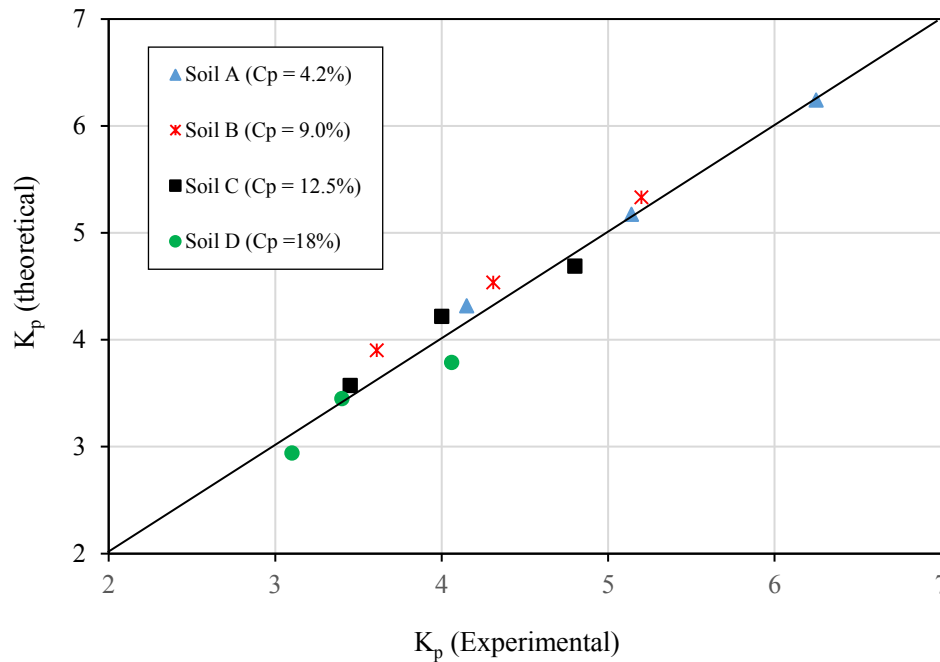
Test no.	Soil type	OCR	Theoretical values			Experimental values	
			$\alpha$ ( $^\circ$ )	$P_p$ (kN) [Eq. (5.11)]	$K_p$ [Eq. (5.13)]	$P_p$ (kN)	$K_p$
1	A ( $C_p = 4.2\%$ )	3.3	26	1.95	4.41	1.84	4.15
2	B ( $C_p = 9.0\%$ )	3.9	25	2.48	4.01	2.26	3.61
3	C ( $C_p = 12.5\%$ )	5.5	24	2.81	3.73	2.66	3.45
4	D ( $C_p = 18\%$ )	7.2	23	3.04	3.24	2.85	3.10
5	A ( $C_p = 4.2\%$ )	4.5	22	1.99	4.77	2.11	5.14
6	B ( $C_p = 9.0\%$ )	5.1	21	2.57	4.42	2.49	4.31
7	C ( $C_p = 12.5\%$ )	6.9	20	2.99	4.10	2.89	4.00
8	D ( $C_p = 18\%$ )	9.9	19	3.32	3.56	3.02	3.40
9	A ( $C_p = 4.2\%$ )	6.0	19	2.10	5.81	2.37	6.25
10	B ( $C_p = 9.0\%$ )	6.6	18.5	2.71	4.95	2.79	5.20
11	C ( $C_p = 12.5\%$ )	8.2	18	3.16	4.52	3.29	4.80
12	D ( $C_p = 18\%$ )	11.7	17	3.55	4.10	3.32	4.06

In this investigation, the analytical model developed in this investigation was used to develop results for a wide range of parameters. These results are presented herein as design charts in Figure 5.40, which will assist designer to predict the coefficient of passive earth pressure  $K_p$  for given OCR and  $C_p$  values for dry collapsible soils.

Figure 5.41 presents comparison between the theoretical and experimental values of  $K_p$  of the present investigation, where good agreement can be observed.



**Fig. 5.40.** Design chart: coefficient of passive earth pressure  $K_p$  of collapsible soil at dry state



**Fig. 5.41.** Comparison between the results obtained from the present theory and the experimental results of the present investigation for the dry state.

For the case of inundated collapsible soil, a reduction factor  $R_F$  was introduced to take into account the effect of the soil collapse on the coefficient of passive earth pressure  $K_p$ . Thus, the coefficient of passive earth pressure of saturated collapsible soils can be determined as follows:

$$K_{P(sat)} = R_F K_{P(dry)} \quad (5.14)$$

It should be reported herein that the results of saturated collapsible soils in the present investigation were divided into two groups: the first group of the data were used to determine the reduction factor  $R_F$  (Figure 5.42), and the second group (the remaining data) were used to validate the results.

Figure 5.43 presents the theoretical values of  $K_p$  determined from Eq. (14) and the remaining experimental values of the present investigation, where a good agreement can be found.

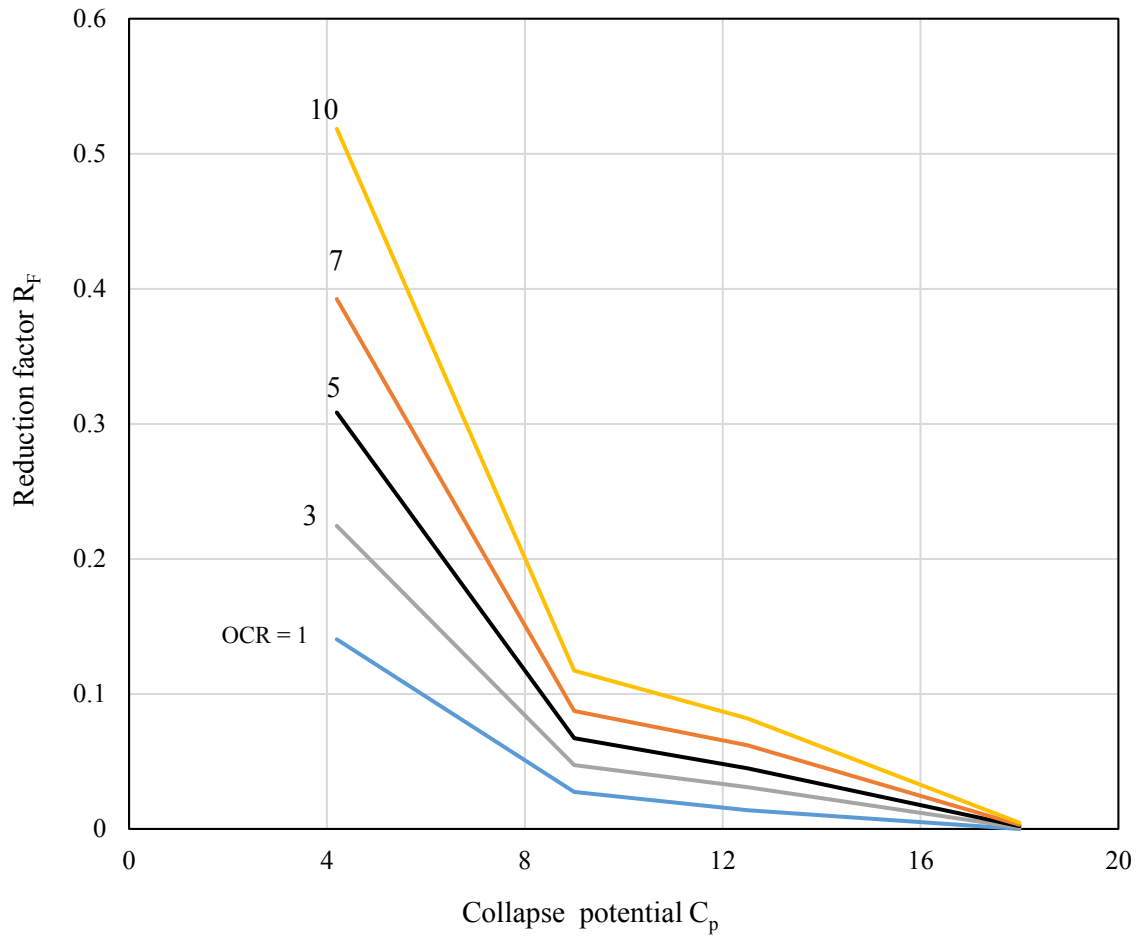
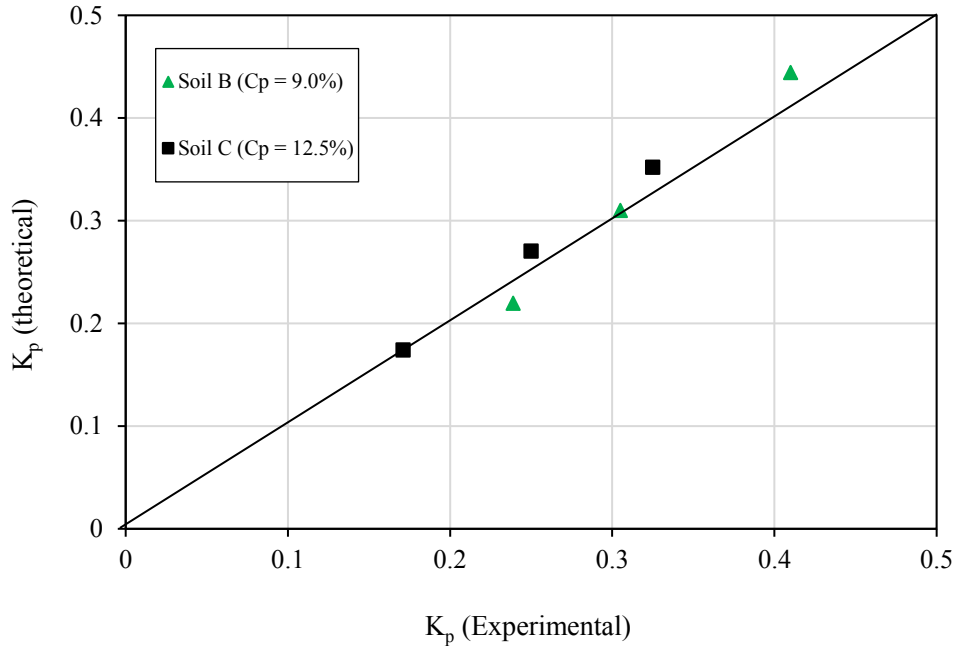


Fig. 5.42. Reduction factor  $R_F$

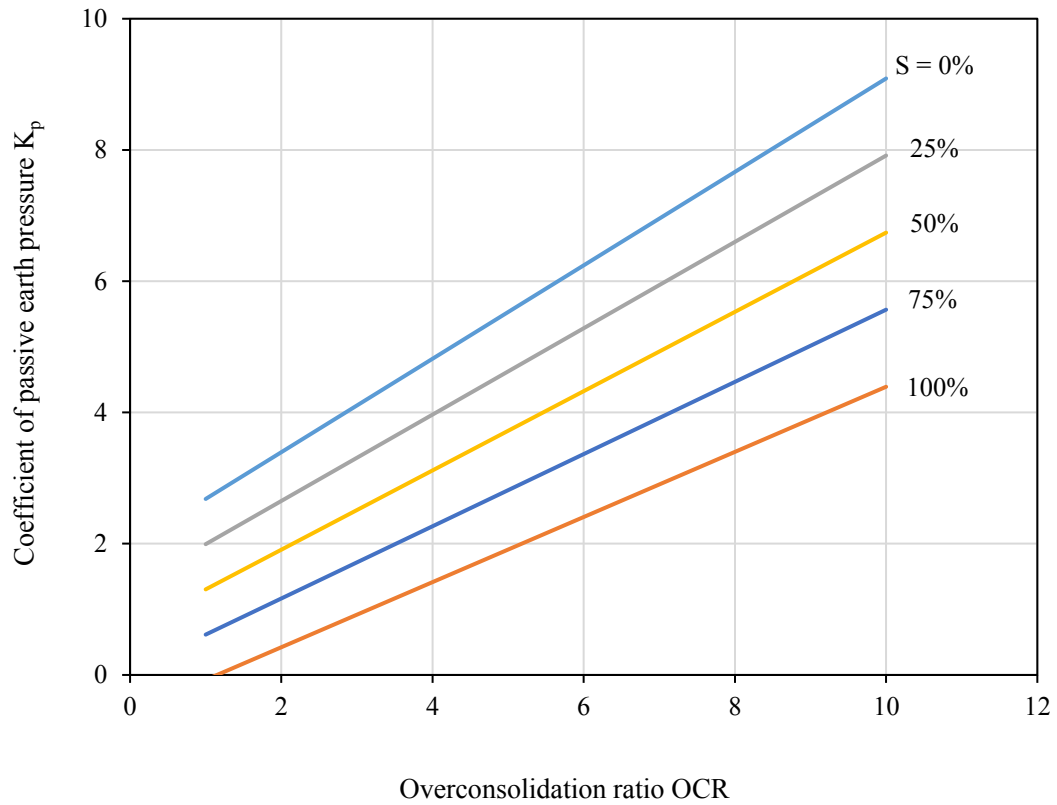


**Fig. 5.43.** Comparison between experimental results and those obtained from the proposed empirical formula [Eq. (5.14)]

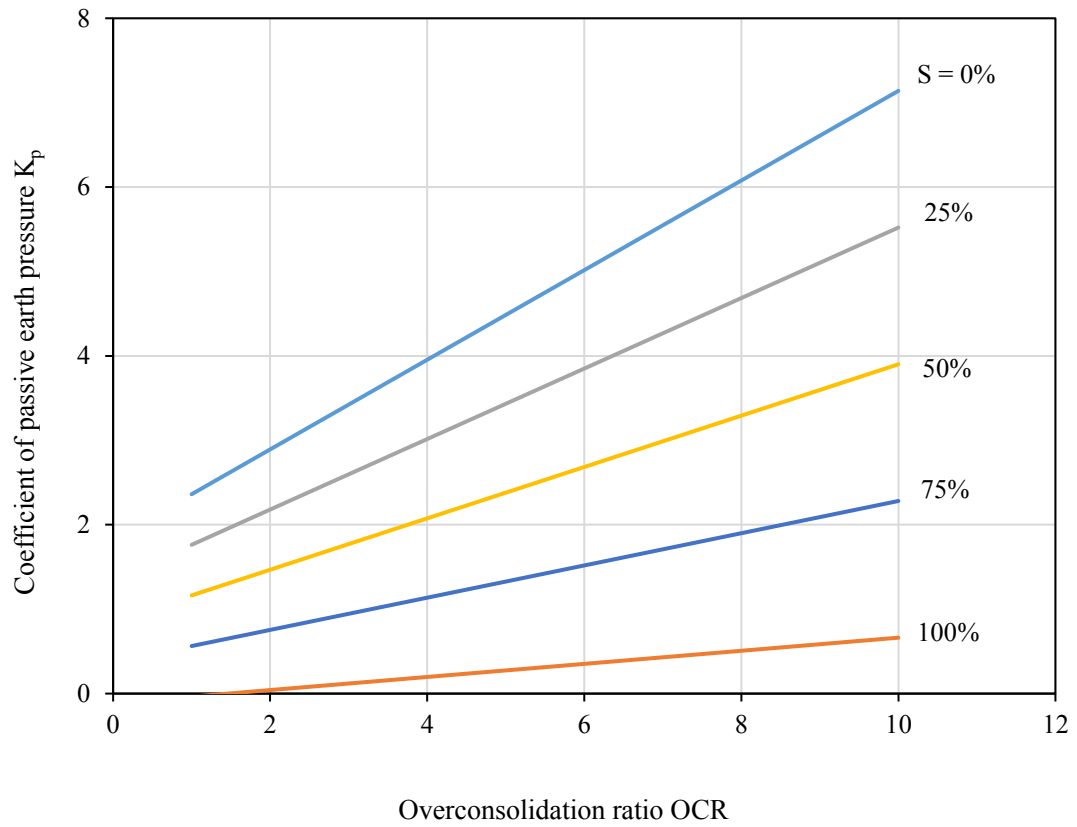
### 5.5. Proposed design charts for predicting coefficient of passive earth pressure $K_p$ for unsaturated collapsible soil at different degree of saturation

Furthermore, in this investigation, with the assumption that the coefficient of passive earth pressure  $K_p$  decreases linearly with the increase of the degree of saturation ( $S$ ), design charts were developed and presented in Figures 5.44 to 5.47 to predict the coefficient of passive earth pressure  $K_p$  of unsaturated collapsible soils as function of the overconsolidation ratio OCR, the collapse potential  $C_p$  and the degree of saturation  $S$  of the soil.

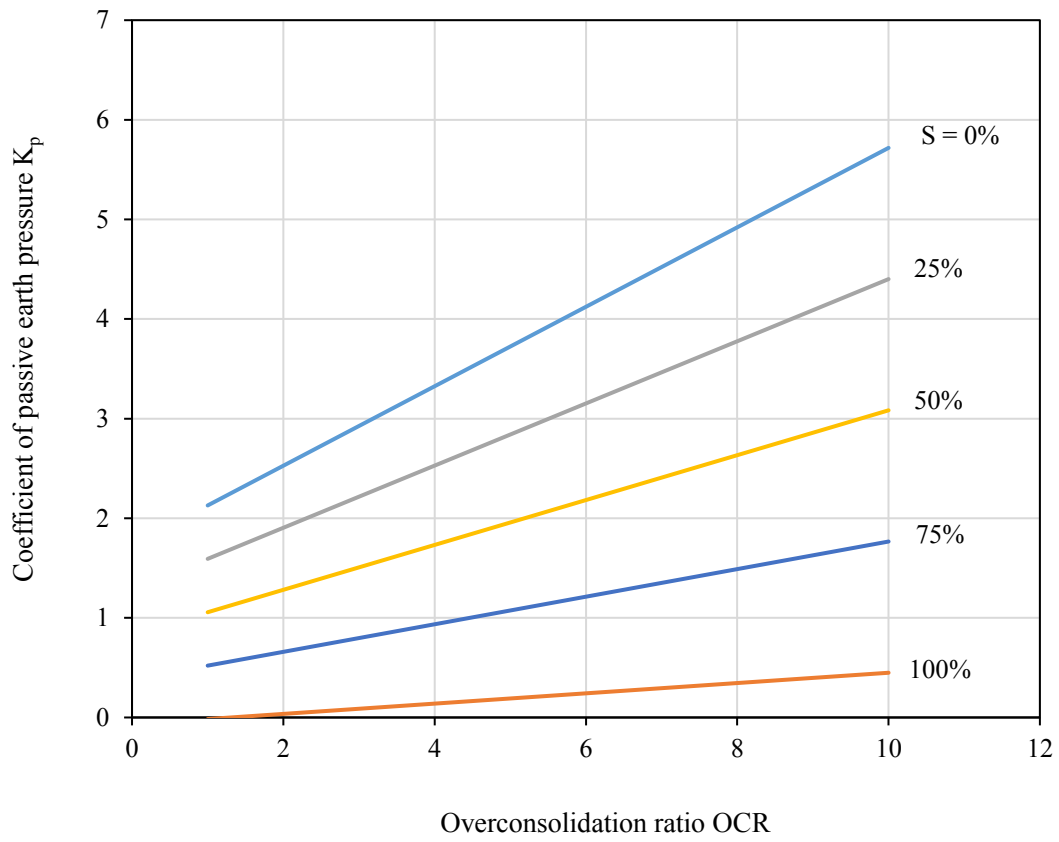




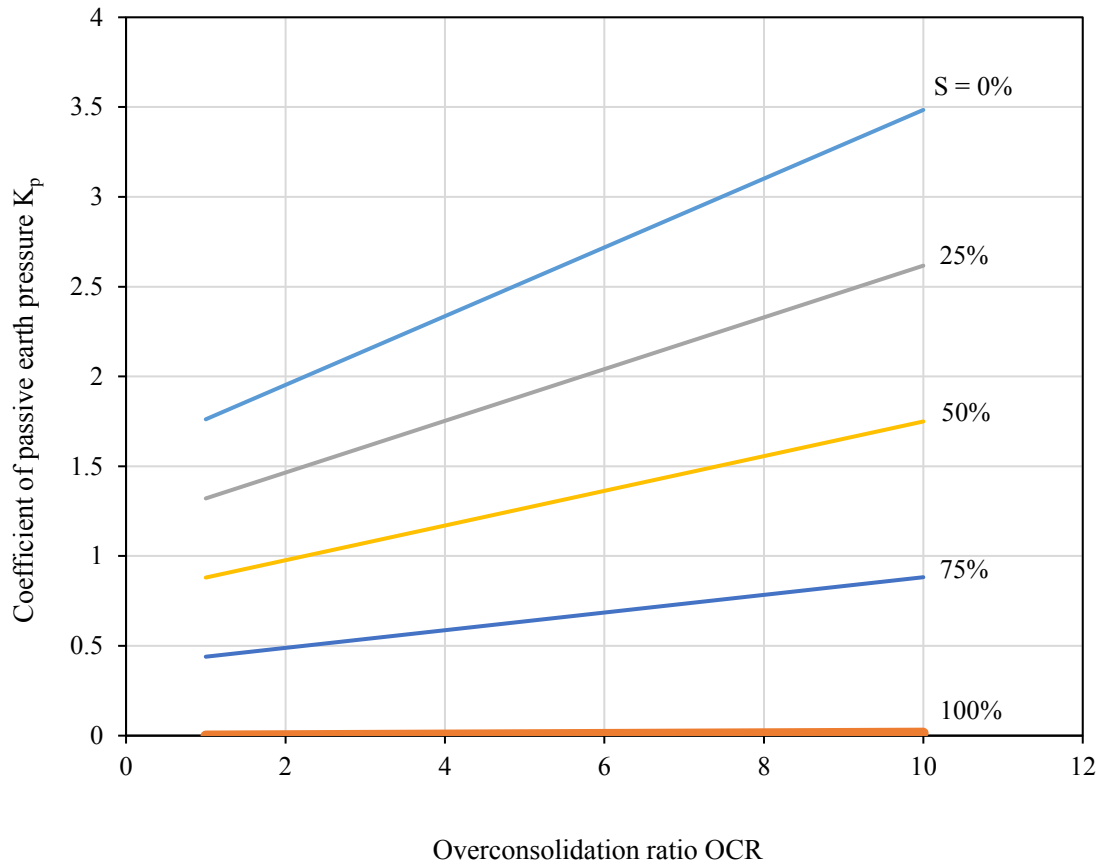
**Fig. 5.44.** Design charts: Values of  $K_p$  for collapsible soils at different degree of saturation “S” ( $C_p = 4.2\%$ ).



**Fig. 5.45.** Design charts: Values of  $K_p$  for collapsible soils at different degree of saturation “S” ( $C_p = 9.0\%$ ).



**Fig. 5.46.** Design charts: Values of  $K_p$  for collapsible soils at different degree of saturation “S” ( $C_p = 12.5\%$ ).



**Fig. 5.47.** Design charts: Values of  $K_p$  for collapsible soils at different degree of saturation “S” ( $C_p = 18\%$ ).

## CHAPTER 6

### CONCLUSIONS AND RECOMMENDATIONS

#### 6.1. Conclusions

An experimental investigation on at-rest and passive earth pressures of overconsolidated collapsible soil at the dry and at full saturation was conducted. The principal objective of the investigation was to provide methods/theories to predict the at-rest and passive earth pressures of overconsolidated collapsible soil at the dry and saturated states. It was also to establish the effect of soil collapse potential  $C_p$  and the overconsolidation ratio OCR on the at-rest and passive earth pressures acting on retaining walls. The following can be concluded:

1. The behavior of collapsible soil was governed by its collapse potential  $C_p$ . The collapse potential  $C_p$  of the soil increased with the increase of the clay content in the soil.
2. Collapse settlement increased due to an increase of the soil collapse potential  $C_p$ . However, the collapse settlement decreased with increasing the overconsolidation ratio OCR of the soil. The collapse settlement increased rapidly after introducing the water up to about 80 % saturation, then continued at a lower rate. Furthermore, collapse settlement took place in relatively short period for higher values of the collapse potential  $C_p$ , while it took longer period for higher overconsolidation ratio OCR. The collapse settlement did not end at 100% saturation but rather continued at a lower rate after, up to 20-40% of the collapse settlement at 100% saturation.
3. The overconsolidation ratio OCR of collapsible soil increased due to the increase of the clay content, which is represented by the collapse potential ( $C_p$ ) in the soils. The overconsolidation ratio OCR also increased when increasing the surcharge “ $P_s$ ” on the soil

mass. However, the overconsolidation ratio OCR decreased considerably during the inundation of the soil. At full saturation, the soil with higher collapse potential  $C_p$  values experienced significant decrease in the overconsolidation ratio OCR value.

4. The earth pressures on retaining walls (both at-rest and passive cases) are function of the soil collapse potential  $C_p$ , the overconsolidation ratio OCR, and the degree of saturation  $S$  of the soil.
5. The following conclusions can be noted for the case of at-rest earth pressure:
  - a. For the case of dry soils: The at-rest earth pressure increased moderately by increasing the clay content in the soil, accordingly with increasing the collapse potential  $C_p$  of the soil.
  - b. For the case of saturated soils: the at-rest earth pressure decreased considerably with the increase of the collapse potential  $C_p$  of the soil.
  - c. At-rest earth pressure increased remarkably with the increase of the overconsolidation ratio OCR for both dry and saturated soils.
  - d. Empirical formulae were developed to estimate the coefficient of at-rest earth pressure of overconsolidated collapsible soils at the dry and saturated states. The proposed formulae compared well with the experimental results for the range of collapse potential  $C_p$  from 4.2% to 18%.
  - e. With the assumption that the coefficient of at-rest earth pressure decreased linearly with the increase of the degree of saturation between the two extreme values for dry and saturated soils, design charts were developed for predicting the coefficient of at-rest earth pressure as function of the collapse potential  $C_p$ , the overconsolidation ratio OCR, and the degree of saturation  $S$  of the soil.

6. The following conclusions can be drawn for the case of passive earth pressure:
- a. For the case of dry soils: the passive earth force and the wall displacement at failure increased with the increase of the clay content, accordingly with the increase of the collapse potential  $C_p$  of the soil. Generally, increasing the clay content in the soil can result in the increase of passive resistance of the soil.
  - b. For the case of saturated soils: the passive earth force and the wall displacement at failure decreased significantly with the increase of the collapse potential  $C_p$  of the soil.
  - c. For the soil having higher values of the collapse potential ( $C_p \geq 12.5\%$ ) the passive earth force of the dry soil was dropped by more than 80% after inundation. Similar observations were noted for the wall displacement.
  - d. Passive earth pressure increased remarkably with an increase of the overconsolidation ratio OCR for both dry and saturated soils.
  - e. Design theory was presented to estimate the coefficient of passive earth pressure of overconsolidated collapsible soils at the dry and saturated conditions. The design theory compared well with the experimental results for the range of collapse potential  $C_p$  from 4.2% to 18%.
  - f. With the assumption that the coefficient of passive earth pressure decreased linearly with the increase of the degree of saturation of the soil, design charts were developed for prediction of the coefficient of passive earth pressure of unsaturated collapsible soil at different degree of saturation.

## **6.2. Recommendations for future work**

The recommendations for future investigations are as follows:

1. Conduct field tests on full-scale retaining wall.
2. Conduct tests on partially saturated collapsible soils.
3. Extend the experimental setup to include inclined walls, sloping backfills.
4. Extend the study to investigate the variation of earth pressure induced by different types of wall movements such as rotation about a point above the top of the wall and rotation about a point below the base of the wall.



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