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ANALYSIS OF MECHANICALLY STABILIZED EARTH WALL, REINFORCED EARTH STRUCTURES

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Ing. Eren BALABAN

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ABSTRACT

There is an increasing interest on environmental concern all around the world. Waste management or storage of wastes takes attention of civil engineers to design environment friendly structures. Developing world increased mobility of people all around world and transportation of goods. Tires are used on vehicles which are used transportation of goods and people. When tires come to end of their life cycle, storage of them becomes huge problem. They are cut into small pieces to use in civil engineering applications such as production of asphalt concrete, concrete. Another usage area of scrap tire is a fill material in geotechnical engineering structures, such as retaining walls and embankment. In this study, tire chips are mixed with sand and clay and their mixtures at a range of 10%, 20% and 30% by weight in order to produce lightweight backfill. In order to determine strength parameters of mixed soils, direct shear tests are performed. Results of direct shear test is modelled on finite element code. Reinforced earth walls are designed using federal highway administration (FHWA) method using direct shear test results for sand, clay, sand tire crumb mixture and clay tire crumb mixture backfills. Designed walls are constructed at laboratory and tested with a loading plate. Another aspect of design of reinforced earth structures consist of effect of foundation layers, because design codes do not consider foundation layers' effect into consideration. Finite element analysis is conducted for different foundation layer properties for reinforced earth wall with different backfills. Results of this study showed that, tire crumbs can be considered as a backfill material and performance of reinforced earth wall depends on properties of foundation soils.

KEYWORDS

Sand, clay, tire crumb, reinforced earth wall, geosynthetic, foundation, direct shear test

ABSTRAKT

Na celém světě roste zájem o životní prostředí. Stavební inženýři navrhují konstrukce, které jsou šetrné k životnímu prostředí s využitím druhotného materiálu z odpadového hospodářství. Z důvodu rychlého rozvoje technologií dochází ke zvýšení přepravy zboží a mobility lidí na celém světě. Pneumatiky, které se využívají v automobilovém průmyslu se stávají na konci své životnosti velkým problémem. Při nevhodné formě likvidace mají nepříznivý dopad na přírodu a životní prostředí. V současné době jsou dnes při recyklaci pneumatiky ve stavebnictví využity v podobě pryžového granulátu, který se přidává jako příměs do asfaltových směsí. Další využití je možné definovat v oblasti lehkých zásypů geotechnických konstrukcí při výstavbě opěrných zdí a zemních násypových těles. Ve své práci se zabývám využitím pryžového granulátu, který je v kombinaci s pískem a jílem míchán v poměrech 10%, 20% a 30%, pro vytvoření lehkého zásypu. Pro stanovení nutných parametrů smykové pevnosti vytvořeného lehkého zásypu byly provedeny a vyhodnoceny krabicové smykové zkoušky a stanoveny základní fyzikálně mechanické vlastnosti testovaného materiálu. Získané výsledky přímého měření smykové pevnosti byly porovnány s modely vytvořenými metodou konečných prvků. Pro návrh zemních konstrukcí byly využity předpisy (FHWA), které definují užití pryžového granulátu se zeminou pro oblasti vyztužených zemních těles v návaznosti na parametrech smykové pevnosti zeminy. V laboratoři Výukového a výzkumného centra v dopravě (VVCD), Dopravní fakulty Jana Pernera byly testovány fyzikální modely navržených zemních těles, kde pro zatěžování a stanovení modulu přetvárnosti byla využita metoda statické zatěžovací zkoušky. Pro rozdílné hodnoty poměru vyztužení granulátu a zeminy byly výsledky získané z fyzikálních modelů analyzovány a porovnány s výsledky modelů vytvořených pomocí metody konečných prvků. Dosažené výsledky svědčí o tom, že lze pryžový granulát mísený se zeminou využít při stavbě vyztužených zemních konstrukcí v oblasti dopravního i pozemního stavitelství.

Klíčová slova

Písek, jíl, drť a granulát z pneumatik, vyztužená zemní konstrukce, geosyntetika, zakládání staveb, krabicová smyková zkouška

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1 INTRODUCTION

Reinforcement of concrete by steel rods has been well known by civil engineers. Therefore, strengthening structures with other materials is not a new idea for civil engineers. This idea was adopted to geotechnical engineering by French engineers nearly five decades ago. Their idea was simple enough, could we strengthen the soil by using steel rods like in concrete structures. They performed some experiments and showed that the idea of reinforcing the soil could be applied in the design step of geotechnical structures. Since that time, a lot of research has been done to understand behaviour of reinforced earth structures.

Reinforced earth could be used under the foundations where bearing capacity of the soil is under desired value. Another application of reinforcing soil is retaining walls. Reinforced earth walls can be used to retain railway and road embankments, bridge abutments. They are also used to retain contaminated wastes in valleys under some special conditions.

Reinforced earth walls are constructed by inserting reinforcement material into backfill soil, placing facing elements (example: concrete blocks, steel facings, wooden facings), adding another backfill soil again. Construction of reinforced soil can be considered as staged construction because, first of all levelling pad is laid through foundation soil and then backfill soil must be placed, compacted, after that, reinforcement rods must be placed. This process continues until the desired height of the wall is reached.

Since the day that reinforced earth walls are introduced, they are widely used in practical engineering. It is easier to construct reinforced earth wall than conventional retaining wall. Reinforced earth walls also have economical advantage than conventional retaining walls because it is cheaper to construct. Another advantage of reinforced earth wall is their aesthetic appearance. Reinforced earth walls are considered as flexible walls because they tolerate lateral and vertical deformation more than conventional retaining walls. They provide faster construction speed than traditional retaining walls.

2 LITERATURE REVIEW

Published studies in the literature can be divided into four groups. Those groups can be counted as experimental studies, finite element/difference studies, case studies and developing/improving design methods like limit equilibrium method or working stress method. Generally, experiments are followed by finite element/difference method to conduct parametric studies.

2.1 Experimental Studies

E. Bourgeois, L. Soyez, A. Le Kouby [1] studied the behaviour of reinforced earth wall subjected to strip load. Experiments are conducted to understand effect of railroad loading to reinforced earth wall. In order to do that 90kN and 850kN loads are applied through railroad to reinforced earth wall. After experiments wall and the loading is modelled in plane – strain and 3D conditions. After the analysis, it is seen that problem is obviously 3D, however by proper defining the properties of wall, reinforcement and load distribution in plain strain condition, tensile forces on reinforcement and deformation of the wall can be reproduced accurately. Using more complex soil models and introducing compaction to FEM programme can produce more accurate results.

Suliman B.A. Mohamed, Kuo-Hsin Yang, Wen-Yi Hung [2] conducted experimental and numerical studies to investigate behaviour of two-tier reinforced wall. Their research revealed that, maximum tensile forces of the lower reinforcements decrease as the upper wall move away from the lower wall. However, this behaviour continues till threshold value. After that threshold value maximum tensile force on the lower reinforcements remains constant. This because that, after that threshold distance upper tier doesn't have any influence over the lower tier. Writers are also studied horizontal deformation of the walls. They observed highest horizontal deformations on the upper tier. As the offset distance of upper tier increases horizontal deformations get lowers. They also concluded that, FHWA design code causes unnecessarily long reinforcements.

Myoung-Soo Won, You-Seong Kim [3] investigated proper measurement of strains for different type of reinforcements. They used different methods of attaching strain gauges to reinforcements. They used geogrid, woven and non – woven types of reinforcements. They used three different strain gauges too. After that, writers constructed earth wall with CL backfill. They concluded that strain gauges can be used to measure strains over reinforcements. They also concluded that, heavy rainfall did not cause additional pore water pressure after heavy rainfall. This may be due to writers used permeable reinforcements. They observed higher earth pressures than Rankine's earth pressure at rest. Due to the heavy rainfall, earth pressure showed fluctuations.

Guangqing Yang, Baojian Zhang, Peng Lv, Qiaoyong Zhou [4] investigated behaviour of 12m height reinforced earth wall. This wall constructed with rigid concrete facing with steel bars inside. Steel rods with 2.5m length are placed from facing towards reinforced soil zone. After that, geosynthetic bags filled with sand placed back of the facing. Then, geogrid layers were placed by wrapping them around those geosynthetic bags. They observed non – uniform pressure over the foundation soil, which has a peak value at the middle. Geogrid layers show two strain peaks. Second strain peak is observed at the end of the geogrid. This is explained as, unreinforced soil zone settled more than reinforced soil zone which caused higher strains at the end of reinforcements.

Ching-Chuan Huang, Woei-Ming Luo [5] is compared performance of cantilever retaining wall and reinforced earth wall constructed over yielding foundation. Walls are constructed 1/4 scale. Yielding of foundation is provided by spring placed under steel plate. Stiffness of spring is changed to observe the behaviour of walls for different foundation stiffness's. After this study, it is seen that, GRS wall is less affected from settlement, showed lower displacements and developed lower lateral earth pressures than cantilever retaining wall.

Eder C.G. Santos, Ennio M. Palmeira, Richard J. Bathurst [6] investigated usage of demolition waste as a backfill. They constructed 3.6m high wrapped around face geosynthetically reinforced wall over collapsible soil. They installed strain gauges to reinforcements. They observed settlement of foundation soil due to weight of wall and rain. Due to settlements upper part of the wall moved horizontally inward at the top. Settlements also caused relatively high horizontal displacements. Those displacements can be acceptable for temporarily walls. Strains measured over the reinforcements are around 0.3% which is acceptable. Strains tend to increase with the rain. Due to rotation of the wall, vertical pressure near toe is higher than it should be.

Chengzhi Xiao, Jie Han b, Zhen Zhang [7] studied performance of geosynthetically reinforced wall under static footing loading over rigid foundation. They constructed walls with 1/5 scale and measured effect of connection type, width of foundation, length of foundation and offset distance of footing. They found out that offset distance of footing is important for the bearing capacity of footing. It is said that, bearing capacity of footing increases until ratio of offset distance and wall height (D/H) equals to 0.3. Then it decreases until the ratio 0.6. After that ratio it becomes constant with offset distance. This means that after D/H ratio 0.6, bearing capacity is controlled completely by backfill sand. Higher foundation width caused higher bearing capacity, however increase in bearing capacity is smaller when D/H ratio is high. Length of reinforcement does not increase bearing capacity, however if smaller reinforcement is lower, bearing capacity of foundation is determined by backfill soil with smaller D/H ratio. Mechanical connection produced lower horizontal deformation. Researchers observed three different types of failure wedges during experiments. Those wedges are similar to Spencer's two-part wedge (bi-linear).

M. Ehrlich, S.H. Mirmoradi, R.P. Saramago [8] investigated effect of compaction to strains over the reinforcement and lateral displacement of the facing. They found out that, well compacted soil showed over – consolidated behaviour. This yields to higher strains on reinforcements but lower horizontal displacement of wall facing.

Chungsik Yoo, Hyuck-Sang Jung [9] investigated behaviour of the two-tiered walls. Two-tiered wall is constructed and during parametric study, effect of densifying reinforcement is investigated. It is seen that; horizontal displacement of lower tier affects the behaviour. Construction of upper tier increased strains in the upper reinforcements of lower tier. In order to minimize the effect of upper tier to lower tier, reinforcements should be densified either by lengthening or decreasing vertical distance between upper layers of the lower wall.

Taesoon Parka and Siew Ann Tanb [10] investigated behaviour of reinforced earth walls by including polypropylene fibers to backfill soil. Tests are undertaken with the 0.2% polypropylene and under 45kN/m static train load. During study, researchers compared behaviour of wall with only polypropylene fibers, only geogrid and geogrid + polypropylene. They found out that best results are obtained by reinforcing sand with geogrid and polypropylene. This implies that economical design can be achieved by using polypropylene fibers.

Mario Riccio, Mauricio Ehrlich, Daniel Dias [11] investigated effect of cohesion into performance of reinforced earth wall. Writers have also studied effect of compaction and vertical loads between backfill and wall facing. They founded that, cohesion tends to increase the strains over the reinforcements. They concluded that, vertical movements stop after construction of each layer, because compaction stress is much higher than the geostatic stress. When the reinforcement stiffness is lower and high compaction stress is applied, a more uniform reinforcement strains are observed.

Chungsik Yoo, Sun-Bin Kim [12] studied 5m height two tier wall by experiments and calibrated it with numerical study. They investigated deformation of wall facing, deformation of reinforced soil zone, deformation of retained soil and strains in reinforcements. When the surcharge loads applied during experiment, it is seen that, upper reinforcements immediately experienced strain increase. However, after some time later, strain decreases gradually with time to constant value. For the lower tier, it is seen that horizontal deformation of the wall is related to, horizontal deformation of reinforced soil zone. However, for the upper tier, horizontal deformation of facing is composed of both reinforced soil zone and retained soil zone. From this, it can be deduced that, for upper tier, surcharge load is important for both internal and external stability, but for lower tier it affects only internal stability. In this study, effect of footing is also studied. It can be said that, larger strains observed just under the footing. In order to avoid stress concentration at this location, additional layer of reinforcement can be placed. For the large surcharge forces, tensile resistance of reinforcement against biaxial or oblique loading can be important.

Carina Maia Lins Costa et all [13] studied time – dependent behaviour of reinforced wall by using centrifuge setup. During experiments, they conducted short term tests to find out maximum g level that the wall can stand without failure. From the short-term test, constant force that will be used to conduct long-term test. After long term tests, results are compared with the ASTM standard test values. It is seen that ASTM test produces good results. Although, the sand is expected to reduce creep effect, ruptures are observed at high g levels.

A. L. Shinde, Æ J. N. Mandal [14] studied behaviour of reinforced earth wall with limited space. They conducted experiments and finite element study. Strip surcharge loads are applied at different magnitudes and locations. It is seen that, when magnitude increases horizontal deformation of facing increases. Horizontal deformation of facing also increases when the strip surcharge loading gets closer to the facing. If the reinforcements are anchored to rigid area, forces over reinforcements and horizontal deformation of the facing decrease.

Sutapa Hazra Æ Nihar Ranjan Patra [15] studied using sand and fly ash as backfill and geogrid as reinforcement for the use in counterfort retaining wall. It is found out that, using fly ash with geogrid reinforcement in counterfort retaining wall is more stable than sand backfill with geogrid reinforcement.

M. Pinho-Lopes, D. M. Carlos, M. L. Lopes [16] undertook flume tests over the reinforced earth wall with fine backfill. In the Aveiro region of Portugal, local people used to collect clay and reinforce with reed to protect saltpans from tidal effects of ocean. However, this process takes much time and work. Therefore, using reinforcements are preferred nowadays. In this study, test results of reinforced walls are compared with results of traditional wall results. Test results of traditional walls are gathered from literatures. It is seen that; traditional wall is more stable than reinforced wall. Traditional wall has lower permeability also. This could also be inferred to using different soils in this study. It is recommended that performing flume tests with fiber reinforced soil is recommended.

H. Ahmadi, M. Hajialilue-Bonab [17] conducted experiments and finite element analysis to clarify the bearing capacity of strip footing in reinforced earth wall. Number of reinforcements, depth of reinforcement, vertical spacing of reinforcement and distance of footing are considered as variables. According to reinforcement depth and footing distance from facing, two types of failure are observed. If footing is close to wall and reinforcement is deep, failure occurs due to wall fails. If the reinforcement is close to footing, failure occurs as general shear failure of soil. If distance of footing increase from the wall, bearing capacity of footing increase up to some point. After that point, bearing capacity decreases.

S. Bali Reddy and A. Murali Krishna [18] studied recycled tyre chips mixed with sand as lightweight backfill. Void ratios and strength parameters are determined for different mixture ratios of tyre chips and sands. Tyre chips are used up to 50% by 10% increments. From the experiments, wall performed best with the 30% chip tyre.

Richard J. Bathurst, Sebastian Althoff, and Peter Linnenbaum [19] studied influence of test method on direct shear behaviour of segmental retaining wall units. Direct shear tests are conducted over different type of shear interface and loading systems. Friction only, pins, shear keys, shear tailings and friction with geosynthetics are used as shear interfaces and piston with fixed pressure, piston with adjustable pressure and airbag are used as loading systems. When airbag is used as loading mechanism, experiment can be implemented better into numerical studies. If only friction surface is used, only shear angle is observed. When shear keys are used, both cohesion and friction is observed during test. It should be remembered that, cohesion is the apparent cohesion which is independent from normal force. Some cohesion is observed for other shear interfaces too.

Guangqing Yang, Huabei Liu b, Peng Lv, Baojian Zhang [20] investigated the behaviour of lime treated cohesive soil backfilled soil. Researchers built 6m height reinforced wall. Observations showed that, just at the end of the construction, vertical pressure equals to theoretical value. However, pressure decreased with time. This can be addressed to arching effect and differential settlements. Lateral earth pressure decreased with time, because lime treated soil gains strength with time. Horizontal deformation of facing can contribute to this behaviour. Strains are stayed constant after compaction due to strength of soil. It is also found that lateral earth pressure is carried by both connection and facing.

Guang-Qing Yang, Huabei Liu b, Yi-Tao Zhou, Bao-Lin Xiong [21] investigated long term behaviour of two-tier wall which contains sand – rock mixture. It is observed that; lateral earth pressure increased at the lower tier and decrease at the upper tier. This is because of the rotation of upper tier. Strains observed over the upper reinforcement layers decreased slightly or remained constant. Strains over lower reinforcements increased with time. It is also concluded that, using rock as backfill can increase heterogeneity which can cause higher differential settlements and variability of density. Existence of rock can decrease bonding between geogrid and soil. Installation damage to strain gauges can increase.

Guangqing Yang et. al. [41] measured the behaviour of wall having 12m height and reinforced by 0.5m interval. Two types of backfills are used during construction. In the lower part of the backfill sand is used as backfill, in the upper part of the backfill, clay is used as backfill. Due to the settlements, maximum foundation pressure is observed in the middle of the foundation. Location of the maximum strains on the reinforcements depends on their location. Locations of maximum strains follows slip surface of the backfill. Reinforcements under the sand backfill show two peaks. It is also found out that measured lateral earth pressure is smaller than the lateral earth pressure on active state.

2.2 Finite Element/Difference Method Studies

Sompote Youwai and Dennes T. Bergado [22] studied behaviour of chip – tyre sand mixed backfilled reinforced earth wall by utilizing finite difference method. In order to calibrate finite difference method, pull-out tests are performed. Then, triaxial testing is done in FLAC and compared with literature. Soil is modelled as elastic – plastic material. Writers concluded that lateral movement of the wall increases with increased tyre chip ratio. Tensile force in the reinforcement increased with increasing shredded tyre ratio in the mixture. If the reinforcement stiffness increases, horizontal deformation of the wall decreases. It is concluded that if the interface stiffness of backfill and soil increases, tensile force over reinforcement increases and lateral movement of the wall decreases.

Chia-Cheng Fan [23] studied behaviour of reinforced wall in a valley by using Plaxis 3D. Reinforced earth wall in a valley has different cross sections throughout the wall. This causes different normal forces along the face of the wall. Different forces yield different horizontal deformations, which disturbs plain strain condition of the wall. Therefore, research was conducted in 3D. Finite element analysis showed that, valley has restriction effect to wall and lateral bending observed at the wall facing. Due to the V shaped valley, considerable strain increase observed in minor direction.

Jian-Feng Chen, Jun-Xiu Liu, Jian-Feng Xue, Zhen-Ming Shi [24] studied behaviour of reinforced earth wall constructed over soft soil. In the study, reinforcement length, stiffness of reinforcement and ultimate strength of reinforcement are chosen as parameters. It is seen that computed curves and finite element method results are in good agreement. It is observed that, pore pressure increases, where slip line passes. It is also better to prolong construction duration in order to construct more stable wall by letting dissipation of pore water pressure. Extension of reinforcement length increased factor of safety against pull-out and rupture. However, after threshold value, extension of reinforcement does not contribute to safety of the wall due to change of failure mechanism. If the stiffness of reinforcement is increased overall factor of safety is increased up to thresh hold value of stiffness. Increased stiffness can yield to less extended reinforcement layers. Research is concluded as; weaker soil causes more layers to be extended.

M. Ehrlich, S.H. Mirmoradi [25] investigated the effect of facing stiffness and toe resistance to behaviour of reinforced earth wall. In order to compare different stiffness block facing and wrap around facing is used. It is seen that, in case of wrap around face shows higher horizontal and vertical deformations. It is seen that tensile forces of the 2nd, 3rd, and 4th layers of reinforcements are almost same for each case. Results are also similar to each other when there is no toe resistance and there is toe resistance. It is also found that, shear forces over the reinforcement is also dependent shear stress developed between facing column and foundation soil. It should be remembered that, magnitudes of lateral earth pressure are closer to Rankine active state in case of wrap around face.

Dov Leshchinsky, Farshid Vahedifard, Ben A. Leshchinsky [26] aimed to explain difference between theoretical failure and actual failure of reinforced earth wall due to bearing capacity. During foundation design, generally Meyerhoff's method is used. In this method, due to overturning and resistive moments, an eccentricity is assumed. After this study it is found out that, failure mechanism is one sided and friction angle between reinforced soil and retained soil has effect in the failure mechanism. It can be concluded that, failure mechanism is different than Meyerhoff's method.

Huabei Liu [27] investigated short and long-term behaviour of reinforced earth walls with different backfills. During study three different wall heights, reinforcement length, reinforcement spacing, reinforcement stiffness, soil strength and soil stiffness are chosen to conduct parametric study. It is found out that, lower stiffness reinforcement led to smaller displacement ratio when spacing distance is lower. After study, it is revealed that, backfill soil, reinforcement spacing and reinforcement stiffness are important factors contributing to lateral stiffness. Longer reinforcement caused higher end of construction lateral deformation while, caused lower creep deformations. Longer reinforcement length also provides lower lateral deformation of retained soil zone. Backfill soil influenced the stiffness of reinforced soil zone, as well as magnitude of the lateral earth pressure behind the reinforced soil zone. Both strength and stiffness of should be considered during the analyzing the lateral displacement at the back of reinforced soil zone.

Yan Yu, Ivan P. Damians, Richard J. Bathurst [28] studied modelling differences between finite element method and finite difference method. Plaxis and FLAC are used as finite element method and finite difference method respectively. They also compared the results. In order to compare results between two methods, results of normal and shear stresses between facing and backfill, reinforcement and backfill panel's axial loads are chosen as comparison parameters. During study, it is told that, different out plane dimensions can be assigned to facing and reinforcement only in FLAC. Both methods produced acceptable and comparable results. However, in FLAC, strains over the reinforcement are lower due to reason explained above.

Huabei Liu, Xiangyu Wang, Erxiang Song [29] studied long term behaviour of reinforced earth wall backfilled with marginal soil. 8m height wall is modelled on Abaqus. During the study, different creep rates are used for backfill and reinforcement. Effect of reinforcement length, stiffness and different spacing of reinforcement are investigated. It is found out that, with constant creep rate of reinforcement, increasing creep rate of backfill resulted, increased lateral displacement of facing and reinforcement loads. If soil creep is lower than reinforcement creep, load is transferred to the soil which yield stress relaxation of reinforcement. While keeping soil creep rate constant, increasing reinforcement creep yields increased wall deformation. It also yields higher stress over the soil.

Jie Han, Dov Leshchinsky [30] analyzed back to back mechanically stabilized walls with different weight to height ratios. Critical failure surface, tension loads over reinforcements and development of lateral earth pressure are studied during study. It is found out that, critical surfaces could interact if the weight/height ratio is smaller than 2. If the weight/height ratio is higher than 2, critical failure surface does not enter reinforced soil zone. Active earth forces are observed in each case; however, earth pressures are more likely to closer to Rankine lateral earth pressure when weight/height ratio is higher. If interaction distance is equal to zero, load on upper reinforcement increased, however load in lower reinforcement decreases on lower reinforcement. This prevents lower reinforcement to failure due to pull – out.

Abdelkader Abdelouhab, Daniel Dias, Nicolas Freitag [31] studied behaviour of reinforced earth wall with different type of reinforcement, different soil strength, different friction and different compaction. FLAC is used to model the wall. In order to calibrate FLAC, pull out tests are performed. It is seen that; increased cohesion causes lower deformation of the wall. As the friction width of the reinforcement increases, safety level of the wall also increases. Extensible synthetic reinforcements provide better adhesion with soil. However, metallic reinforcements show better settlement performance. Elasticity modulus of reinforcement highly affects the deformation of the wall. In order to capture the tensile loads of reinforcements non – linear soil models should be used. It is seen that; interface parameters depend on the confinement pressure. Confinement pressure also affects shear stiffness too. Importance of compaction is revealed in this study.

Hoe I. Ling Æ Huabei Liu [32] analyzed modelling of reinforced earth wall. In the first part of the study soil is modelled as non – linear hyperbolic elastic, reinforcement is modelled as non – linear elastic, facing blocks are modelled as linear elastic. Interfaces are modelled also elastic materials. This is called simplistic method. In the second part, interface elements are modelled as elastic – plastic interface element, generalized plasticity soil model applied, reinforcements and blocks are assumed linear elastic materials. After study it is seen that similar results are acquired for facing horizontal displacements, facing lateral stress, vertical stress over foundation and strain in reinforcements. The results from both methods are similar to measured data.

Yonggui Xie, Ben Leshchinsky [33] undertook a study to find out optimum reinforcement density for bridge abutments. The present results are densifying reinforcements where it is needed. Reinforcements are densified from top to down or down to top. Locations of surcharge, strength of reinforcement are also changed for parametric study. Densifying reinforcements have positive effect; however, after some point failure type is changed from sliding. It is also found out that if place of surcharge load is further from distance or high strength of reinforcement used less densifying the reinforcements can produce same results.

I. P. Damians et. al. [34] studied behaviour of reinforced earth wall by changing foundation compressibility and reinforcement stiffness. Soil, soil – block interface and reinforcement – backfill interface are modelled during study. It is seen that as the stiffness of foundation and reinforcement stiffness are reduced, facing deformation increases. Reinforcement with higher stiffness deployed higher stresses than reinforcement with lower stiffness. Trapezoidal stress distribution observed over lower stiffness reinforcement while in higher stiffness reinforcement it was not observed. If reinforcement with lower stiffness is used over the low stiff foundation soil, higher stresses are observed over reinforcement. Highest pressure in the foundation soil is observed near to toe instead of just under the toe due to up – drag forces in the connections. There is not any connection between height of the wall and rigidity of foundation in case of foundation pressure. It should be remembered that highest pressure observed further from toe in case of rigid foundation.

Lazhar Belabed, Hacene Benyaghla and Jarir Yahiaoui [35] studied internal stability of reinforced earth wall. They considered the effect of possible failure wedges and earth pressure distribution into the reinforcement loads and safety of the reinforced earth wall. They considered different internal angle of friction and height of the wall throughout the wall. Finite difference method employed in the study. Gathered results are compared with the mathematical results. It is seen that in all cases, the least preferable condition is mixed failure wedge with elliptical loading.

Ben Leshchinsky [36] conducted parametric study to see the effects of density of reinforcement, strength of reinforcement and setback distance of footing into the behaviour of MSE wall. After studies, it is seen that, increasing density of reinforcements can yield smaller setback distance and higher bearing capacity of the footing. Increasing strength of reinforcement also has positive effect on the behaviour of wall and footing.

XUE Jian-feng, CHEN Jian-feng [37] tried to incorporate the limit equilibrium method into stress reduction method. During calculation of factor of safety, strength of soil and strength of reinforcement are reduced, while in strength reduction method only strength of soil is reduced. This causes differences in calculated factor of safety with these methods. Writers suggested an iterative method in order to incorporate the strength reduction of reinforcement into strength reduction method. They applied that methodology into stronger and weaker reinforcement. Results showed that, after few iterations, factor of safety converges into constant value. In case of stronger reinforcement, convergence of factor safety takes higher number of iteration than weaker reinforcement. It is also seen that, in weaker reinforcement failure type changes to compound failure from global failure.

Graeme D. Skinner and R. Kerry Roweb [38] studied design of reinforced earth wall on a yielding foundation. Effect of bridge abutment and traffic load are also considered during study. N/C yielding, and N/C failure is observed on foundation failure. After consolidation of foundation soil, vertical and horizontal deformations of wall are increased. It should be remembered that, vertical and horizontal deformations are also a function of facing stiffness, stiffness of backfill soil and stiffness of reinforcement. During consolidation, foundation soil in front of the soil level is increased. Vertical stresses around wall toe are higher due to force transfer mechanism between backfill soil and facing. Horizontal stresses are also lower except bottom of the wall and the top of the wall after 95% of consolidation. The reasons are increased vertical stress in the foundation soil for the first one. The reason of the second one is stress redistributions in the backfill and additional loadings due to rotation of wall.

Jie Huang et. al. [39] studied behaviour of reinforced earth wall behaviour with a drilled shaft in the reinforced soil zone. During study, facings are modelled as a rigid facing. The study has 3 parts. Initial analysis of the reinforced wall is the first stage of the study. Measurement of real wall is the second phase of the study. In the last stage, finite element model is revised according to measured data. It is found out that, horizontal deformation of drilled shaft increases horizontal deformation of the wall. Writers revised the study and refined the finite element model [40]. In the new study, strain hardening model is implemented to model soil, facing is modelled discretely. Effect of compaction and stress dependent elasticity modulus of soil are considered. After completion of the refined analysis, results are closer to the experimental results. It is seen that after 225kN lateral load applied to drilled shaft, deformations become non – linear. As drilled shaft deforms, facing of the wall deforms too. As load acting over drilled shaft increases, strains are increased on reinforcement. Strain increase higher in the places where it is closer to the shaft.

Dov Leshchinskya, Yuhui Hua, Jie Hanb [60] studied active earth pressure on reinforced earth wall with limited backfill space. They used ReSSa and Flac programme to model the wall with different backfill spaces. During the study they modelled resultant reinforcement at the (1/3)H. After that, they obtained active earth pressure coefficient for each case and constructed a design chart to determine active earth pressure coefficient with limited space. They also concluded as, installing anchors into bedrocks and then connecting end of reinforcement to those anchors can provide resistance against pull out of reinforcement. Using benching excavated into rock can eliminate a potential weak interface and also provide an anchor length for reinforcement to develop frictional anchorage.

2.3 Case Studies

Krystyna Kazimierowicz-Frankowska [42] studied deformations of wrap – around face and deformation of geosynthetics. Deformations are studied at the end of the construction and 33 months after completion of construction. It is seen that deformations are highest at the end of construction. However, they decreased with time. The major settlements occurred in 6-month period after construction. Creep effects are observed after 6 months. The negative effects of UV are observed.

Jian-Feng Xue et. al. [43] investigated failure of reinforced earth wall constructed on soft clay foundation. During the construction, in order to drain excess pore water pressure from foundation soil pre – fabricated vertical drains (PVT) are installed. However, excess pore water pressure increase and failure of wall are observed. Then, in order to find out possible reasons of failure, Plaxis model is run. It is seen that, during construction PVTs are failed. Failure of PVTs caused excess pore water pressure. Increased pore water pressure on foundation soil caused yielding of foundation soil, which caused failure of wall. After analysis, it seen that, including some PVTs in front of the wall improves the stability.

Abdolhosein Haddad and Gholamali Shafabakhsh [44] investigated possible reasons of failure. Wall is constructed with a considerable fine backfill. In order to determine strength parameters samples are gathered. After that, wall is modelled in FEM programme. It is found out that backfill soil has significant amounts of fines which caused low permeability. FEM analysis showed that reinforcements have low factor of safety against pull – out capacity and rupture. Slope – stability analysis also yielded to low factor of safety for wet case.

Chungsik Yoo [45] investigated behaviour of reinforced earth wall constructed 6 years ago. Wall showed great lateral earth deformation. First observation on the field showed vertical spacing of reinforcement does not comply with NCMA specification. It is also seen that significant upper part of the backfill remained unreinforced because of drainage ducts. Construction reports are read as a second part of study. It is written in the repost, some part of wall showed excessive lateral bulge. Due to the rain, lateral deformations increased. Analysis of wall regarding FWHM and NCMA factor of safeties were low. It is concluded that, due to the low factor of safety and heavy rain, wall experienced large lateral deformation. The behaviour of wall also observed in ABAQUS. At the end of the study, remedial treatment is proposed by writer.

R. Kerry Rowe and Allen Lunzhu Li [46] gathered information from literature for reinforced earth wall. Reasons of failure or excessive deformation are listed. In this paper followings are said; strains over reinforcement and creep forces are higher when clay is used as backfill. Post construction strains higher in case of clay backfill. Rankine and Coulomb methods predict failure surface very well however, they overestimate the strains on reinforcement. This behaviour is because of Rankine and Coulomb methods do not estimate loads acting over facings. Increasing facing rigidity yield higher connection forces on reinforcements, however it reduces strains in shear surface. Effect of foundation also referred in this study but concluded as further studies are required to explain effect of foundation to behaviour of reinforced earth wall. Type of reinforcement is also important for the performance of wall after reinforcement.

James G. Collin [47] investigated failure of reinforced earth wall which has a primary and secondary reinforcement. It is found out that, connecting only secondary layer of reinforcement and heavy rain and lack of adequate draining system. While connecting only secondary reinforcement is considered during design and construction, phreatic level was not thought. As a treatment, all facings are removed, primary reinforcements are connected to facings, and more secondary reinforcements are added to wall.

A. Sengupta [48] investigated possible reasons of failure of reinforced earth wall. In order to determine strength and physical properties of backfill and foundation soil, SPT and CPT are conducted. Unconsolidated undrained tests are conducted to specimen gathered from site. Foundation soil is identified as silty clay. It is also found out that, foundation soil is in pre - consolidated state. Unit weight of the backfill soil is underestimated while bearing capacity of foundation soil is overestimated. After that, slope stability analysis is done using Bishop's method. Bishop's method showed that, horizontal forces are not in equilibrium and factor of safety against circular failure is less than 1. After construction, foundation soil consolidated which caused failure of wall. During re – construction, prefabricated vertical drains (PVT) are installed to prevent from excess pore water pressure.

2.4 Improvement\Development of Design Methods

Assaf Klar, Tal Sas [49] developed a new design method. This new method is called kinematic constrains (KC). This method takes into account parts of the wall during design, which means distributing forces between facing of the wall and reinforcement. It provides compatibility between facing and reinforcements. This procedure overcomes limitation of top – down procedure and also requires less input data then finite element or finite difference methods. Method uses iterative process to find out solution of the wall. First of all, tensile strength variations are found over the reinforcements for different failure wedges. Secondly, displacements of reinforcements are evaluated. After that, horizontal support forces are calculated by using Coulomb active earth pressure theory. Then, initial shear forces acting over the wall are guessed to calculate horizontal support forces. In order to find out correct solution, differential evolution algorithm is used. This algorithm minimizes the deformation of wall and reinforcement to find correct solution. Then bending moments over wall can be calculated. Results are compared with FLAC and they are compatible with each other.

Thai Son Quang et. al. [50] developed a method called multiphase by using elastoplastic context. It combines the fast and easy calculation of homogenization method with specific failure condition at the interface soil and reinforcing strips.

Suliman B.A. Mohamed et. al. [51] investigated applicability of limit equilibrium into geo-synthetically reinforced two-tiered walls. Writers suggested using constant tensile stress distribution over reinforcement. Distribution of tensile stress has great effect in determination of long-term strength of reinforcements. Location of circular failure line agrees well with failure lines obtained from experiments. When LE and FHWA failure lines are compared, it is seen that FHWA predicts failure line far away from wall facing which results in overestimation of embedment length of reinforcement.

V. A. Barvashov and I. M. Iovlev [52] established a calculation method for reinforced soil mass, especially for soil nails.

Jonathan T. H. Wu & Jean-Baptiste Payeur [53] investigated forces at interfaces like block to block and block to reinforcement connection. Resisting and driving equations are derived for friction type connection. Then parametric analysis is undertaken regarding block to block friction angle, block to reinforcement friction angle and soil friction angle. It is seen that, as the vertical spacing of reinforcement decreased, net connection force increases. When block to block friction angle increases, connection force also increases. When block to reinforcement friction angle is low, negative connection forces can be observed. It should be remembered that, negative connection force is not desirable in reinforced earth wall. Increasing bulk unit weight of the block and equivalent depth of block contributes to safety of the wall.

Satyendra Mittal et. al. [54] developed new design charts for rigid retaining wall and rigid retaining wall with reinforcements. Then experiments are conducted with walls which are designed according to new design charts. Backfill soil is chosen as sand with 10% fines. Backfill is used in dry state. After that parametric studies are conducted. Percent of cohesive soil, magnitude of surcharge and length of reinforcement is thought as variables.

Richard J. Bathursta et. al. [55] developed a new design method in order to overcome conservative solutions produced by AASHTO and other design methods. New design method is named as Working Stress Method and it is used to capture actual stress level over reinforcements. In order introduce this new method, very extensive literature review is done to understand effect of measurement type of strains, determination of geotextile stiffness, installation damages, facing element and its stiffness, facing batter, soil properties and duration of loading and magnitude and distribution stress over reinforcement. Method produces good results until strain value of reinforcement of 3%. This is due to backfill soil fails after 3% strain and after that point reinforcement strain is out of scope of working stress condition method.

Dov Leshchinsky, Beongjoon Kang, Jie Han [56] developed an ideal design method called general framework. It is basically reverse application of safety map method. In the general frame work, points of constant factor of safeties are identified which means potential failure surfaces have the same theoretical likelihood of passing through any point. This method produces data required for ultimate limit state design. Other limit equilibrium methods can be implemented in this framework.

R. Baker, Y. Klein [57] developed fully integrated limit equilibrium design process for reinforced earth walls. Method explicitly considers strength properties of the wall facing, soil and reinforcement. Design requirements are enlisted as local inequalities and enforced at each relevant point rather than only globally. Interaction between soil and reinforcement is represented by reinforcement response functions. Those functions can be obtained from pull-out tests and can be characterized by reinforcement type and retained soil. The interaction between wall and reinforcements are represented by a system of interaction parameters. These parameters are dependent of relative strength of wall and reinforcement. At the end of the design process, stress distribution over each reinforcement, soil pressures over the wall, shear forces and bending moments acting over the wall can be found out. These functions allow complete and rational design of each property.

D. M. Carlos, Margarida Pinho-Lopes [58] studied behaviour of reinforced walls in the Aveiro Lagoon of Portugal. External stability analysis is done for sand and sand with fine particle mixtures. Length of reinforcement and vertical spacing of reinforcement are changed during study. Two different design methods are used. One of design method only considers pure sand; another one considers sand with fine particles. It is seen that, method which considers pure sands produces more conservative results. It is seen that for effective stress concept (long term behaviour) wall is safe for each case however, for total stress concept (short term behaviour) there might be problem regarding sliding.

R. Baker, Y. Klein [59] applied their method which is explained in [57]. They applied the method into conventional retaining wall, reinforced slope and reinforced wall with rigid facing. Study showed that, sum of maximum stress of each reinforcement layer is not equal to Rankine's active force, which yields to use of higher strength reinforcements. It is also shown that location of maximum tensile forces over reinforcement layers does not coincide with Rankine's failure plane. Maximum reinforcement load is closer to the wall facing; therefore, shorter reinforcements can be used in the design of wall. Connection loads are also lower than the connection loads evaluated in conventional design.

O. Al Hattamleha, B. Muhunthanb [63] adopted membrane methodology to calculate the strains in reinforcement layers. Results are acceptable in lower reinforcements while they are not at upper reinforcements. This might due to membrane analogy is not applicable to upper layers.

2.5 Other Studies

Economic aspects of reinforced earth walls, reliability analysis and comparison of design of reinforced earth wall are introduced basically in this chapter.

B. Munwar Basha, G. L. Sivakumar Babu [61] applied load resistance factor design approach, which is based on reliability analysis of structures, into reinforced earth walls. In this study, method is applied to external stability analysis of reinforced earth wall. All parameters that cause uncertainties in design are identified. Then, probable failure modes of earth wall are created. Finally, load and resistance factors are created to use in the design of reinforced earth wall.

Ömer Bilgin [62] discussed effect of various conditions into length of reinforcement. AASHTO design code is used during the analysis. It is found out that, required reinforcement length ratio decreases as height of the wall increases. When the unit weight of the backfill soil increases, required reinforcement length also increases, while increase in unit weight of reinforced soil causes decrease in reinforcement length. However, if soil friction angle of backfill soil increases, required length of reinforcement decreases. Failure mode also depends on the value of the angle of friction of reinforced soil. If angle is higher, most probable failure mode is pull – out of reinforcement, while when angle of friction is low, then eccentricity is the governing mode of failure. Only sliding from external failure criteria is affected from internal angle of reinforced soil zone.

Robert M. Koerner and George R. Koerner [64] focused on possible applicable drainage systems into reinforced earth walls, which are constructed using fine backfill soil. Several possible solutions for proper drainage of water are explained with their schematic drawings and reasons.

Robert M. Koerner, Te-Yang Soongb [65] compared retaining walls according to their costs, method used in their design. Writers also examined the failure or extremely deformed reinforced earth walls reported in the literature. They found that, the most economical retaining wall is reinforced earth walls. It is also stated as cost of synthetically reinforced earth wall is getting closer to strip reinforced earth wall. Modified Rankine Method is the most conservative design method among the Modified Rankine Method, NCMA and FHWA. FHWA permits to use fines up to 15%, while NCMA permits to use fines up to 35% with PI equal to six. However, it is found out that, even these values cause failure or extreme deformation of wall during the rain. They concluded as, control of facing blocks should be done properly. Cracks on the blocks can be due to extreme distortion or settlement of the wall.

P. K. Basudhar et. al. [66] formulated a way for how to calculate cost of reinforced earth walls. During the study geosynthetic and geogrid type of reinforcement are considered. In order to design the wall effectively, Sequential Unconstrained Minimization Technique is used. Then methodology is programmed in FORTRAN.

F. Tatsuokaa et. al. [67] described the rehabilitation and reconstruction of reinforced earth walls in Japan after serious earthquakes. They mostly concerned about reasons of replacing retaining walls to reinforced earth walls. They concluded that, reinforced earth wall with full height rigid facing is chosen because of its ease of construction, cost efficiency, quick construction and better performance than conventional retaining walls.

Sajna Sayed, G.R. Dodagoudar, K. Rajagopal [68] conducted reliability analysis of reinforced earth walls by using response surface method because response surface method can incorporate reliability analysis with finite element method. In this method, a failure criterion is specified from the beginning and then input parameters are specified with randomness such as standard deviation, coefficient of variation. After that, sampling points are selected from input parameters. With those input variables, finite element analysis is conducted and response surface is created. Then probability analysis is performed by using limiting value and response surface. Sensitivity analysis can be performed in order to find which input parameter has more effect. It should be remembered that, deterministic methods and reliability methods yields to same safety level of structure, however reliability methods reflect site conditions better.

Kalehiwot Nega Manahiloh et. al. [69] introduced a Harmony Search Algorithm (HSA) optimization method to design reinforced earth wall by considering cost optimization. HSA algorithm coded in MATLAB. It is found out that HSA algorithm gives better results than Sequential Unconstrained Minimization Technique (SUMT). It is also given that for 9m height wall, cost saving is less than 7m height and 5m height wall in static condition.

3 AIM OF THE DISSERTATION

Literature review showed that most of the literature is concerned about experimental or numerical study. Those studies are mostly concerned about behaviour of reinforced earth wall constructed over rigid foundation with pure sand backfill. However, it is clear from case studies in the literature, significant amount of fine soil is used as backfill during construction. Fine backfill caused extreme displacement or even failure of reinforced earth wall. Contribution of the cohesion into design, force transfer mechanism between soil and reinforcement is not clear. Even though some papers pointed its effect [11, 31], this effect is not explained properly. Another important aspect in the literature is effect of foundation soil into behaviour of reinforced earth wall. Some papers studied [5,6,34,38,43,46,48] effect of deformable foundation. Some of those researchers considered only elastic settlements while some of them considered consolidation settlement of foundation. They all concluded deformation of foundation effects behaviour of reinforced wall badly, sometimes severely wall. However, change of performance of reinforced earth wall is not clear with respect to foundation soils' properties at literature. Another important part of the literature is the creep or time dependent behaviour of reinforced earth wall [13, 27, 29, 42, 46]. They concluded as if clay is used as backfill, time dependent behaviour is more important criteria to think during design.

Some researchers [10, 18, 22] used some additives to enhance the behaviour of the reinforced earth wall with a sand backfill. Clay backfill is used in Portugal [16,58] as a tradition method of construction of retaining wall. Traditional method in Portugal to construct clayey walls by adding roots of local vegetable into clay. Since good results are obtained for sand, and it worked out in Portugal in the past, some additives can be added into clay.

There is a contradiction in the literature in case of using tyre chips to obtain light weight backfill. It is also unclear to effect of tyre crumbs which have smaller grains to behaviour of reinforced earth wall. In order to clarify its effects reinforced earth walls constructed at laboratory with various tyre crumb contents. Effect of tyre crumbs to clay backfill is also evaluated at the laboratory.

In order to reveal effects of fine particles, mixtures are prepared containing low and high percentages of fine particles. Ratio of each mixture is given in the Table 3.1. Since effect of additives are also under consideration, chip tyre will be used as an additive. Strength parameters of soil are found without and with tyre chip content. Those results are presented in the next section.

4 METHODS AND METHODOLOGY

Backfills are created by mixing sand and clay. In each mixture, clay content is increased by 10%. Grain size distribution of sand and clay is determined. Table 4.1 shows mixture ratios with sand and clay. Maximum dry unit weights and optimum water content are evaluated. Direct shear tests are conducted to determine shear strength parameters of mixtures. Direct shear tests are conducted under 9.81 kPa, 19.62 kPa, 40.81 kPa and 58.86 kPa. After that, direct shear tests are modelled on Abaqus. Then, tyre crumb is added to each mixture to determine effect of tyre crumbs. Tyre crumb is also added 10% of mixture and increased by 10% for increment of tyre crumbs in mixtures. Maximum dry unit weights of all mixture with tyre crumbs are also determined. Direct shear tests are also conducted for soil-tyre crumb mixtures. Then small-scale reinforced earth walls are constructed at laboratory and tested in order to study effect of tyre crumbs to performance of reinforced earth walls. Finite element analyses are conducted in order to determine effect of backfill and foundation. Tested soils' properties in laboratory are used during finite element analysis.

Table 4.1. Sand and clay mixture ratios

Sand [%]	Clay [%]
0	100
10	90
20	80
30	70
40	60
50	50
60	40
70	30
80	20
90	10
100	0

5 RESULTS

5.1 Experiment Results

5.1.1 Particle Size Analysis of Soils

Sieve analysis and hydrometer analysis are conducted in order to determine particle size distribution of soils used in this study. Sieve analysis is conducted to sand and tyre crumbs and hydrometer analysis is conducted to fine soil. Gradition curves of sand and tyre crumbs are given on Figure 5.1 below.

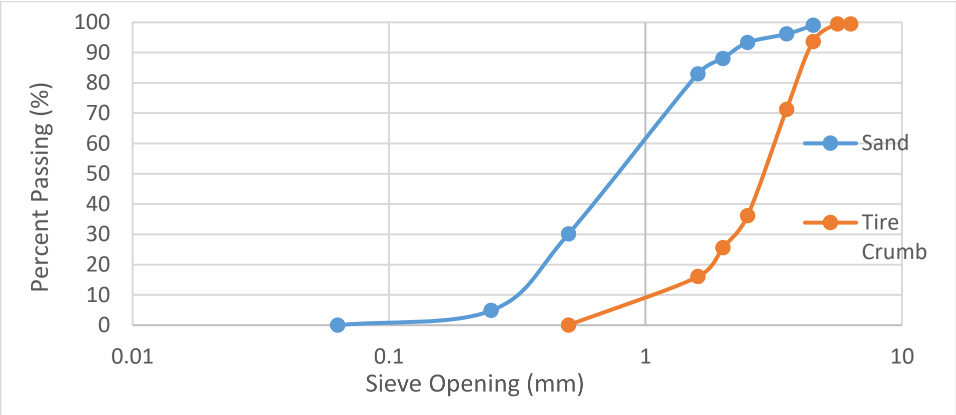


Figure 5.1 Particle Size Analysis of Sand and Tyre Crumb

According to results of sieve analysis, coefficient of uniformity, C_u and coefficient of gradation C_c are found as 3.3 and 0.84 respectively. According to these results, sand is classified as poorly graded sand (SP). The hydrometer analysis results are provided on Figure 5.2 below.

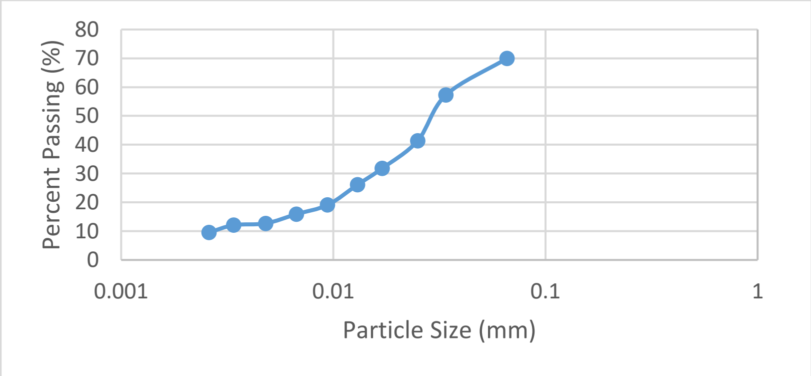


Figure 5.2 Particle Size Analysis of Fine Soil

5.1.2 Specific gravity

Specific gravity of the fine particles is determined in the laboratory VVCD. Fine particles are used smaller than 0.075 mm before beginning of experiment. Specific gravity of the fine soil is found as 2.67 at the end of the experiment. Details of the experiment is given in the table 5.1.

Table 5.1. Specific gravity test results

Pycnometer (g)	53,93
Pycnometer + Water (g)	116,9
Pycnometer + Soil (g)	75,07
Pycnometer + Water + Soil (g)	130,19
Soil (g)	21,14
Water Density(gr/cm^3)	0,99754
Specific Gravity (g/cm^3)	2,69

Sand used in this study is bought from local supplier called as Cemex. Its specific gravity is provided by the supplier. It is given as 2.9.

5.1.3 Atterberg limits

Plastic limit and liquid limit of fine particles are determined in this section of the study. Plastic limit is determined according to ASTM 4318 [72]. In order to see the obtained result is comparable and repeatable, two different experiments are conducted. Plastic limit is calculated as the average of those experiments as 20.18%. Details of the experiment are given in the Table 5.2.

Table 5.2. Plastic limit test results

	P1	P2
Tare (g)	89,1	77,13
Tare + Wet Sample (g)	124,17	94,21
Tare + Dry Sample (g)	118,37	91,3
Water Content (%)	19,82	20,54

There are two methods to calculate liquid limit of fine particles. In the first method, soil is placed inside a brass cup. Half – inch of the soil is removed. After that, cup is dropped from certain height. Number of drops required to close opening between soil is noted. Liquid limit is equal to water content of soil when the opening closes at 25 drops. Second method is called as fall cone method. In the fall cone method, a needle is dropped from the surface of the soil. Penetration of the cone and corresponding water content is noted. In this method, liquid limit is equal to water content corresponding to 20 mm penetration. However, in both methods, it is almost impossible to measure those values at once. Therefore, at least three experiments are required to determine liquid limit. In this study, fall cone method is preferred and three experiments are conducted. Graphic given below is drawn after those experiments.

Details of the experiments are given in the Table 5.3. According to experimental results, liquid limit is calculated as 35.86%. Plasticity index which is described as difference between liquid limit and plastic limit is found as 15.68%.

Table 5.3. Liquid limit test results

Needle Penetration (mm)	9,1	13,2	19,9
Tare (g)	23,62	23,43	21,73
Tare + Wet Sample (g)	35,58	36,73	36,88
Tare + Dry Sample (g)	32,95	33,59	32,9
Water Content (%)	28,19	30,91	35,63

According to results of hydrometer analysis and atterberg limits, the fine soil is classified as lean clay (CL) according to unified classification system.

5.1.4 Standard proctor test

Some of the design codes and most of the literature requires standard proctor test to determine maximum dry unit weight and optimum water content. Standard proctor test is applied to each mixture with different moisture content to determine maximum unit weight and optimum water content. Graphics constructed for clay and sand is given in Figure 5.3 and 5.4 respectively.

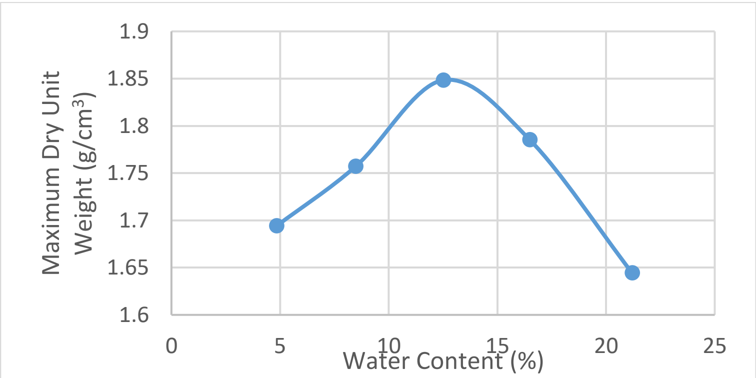


Figure 5.3. Standard proctor test result for clay soil

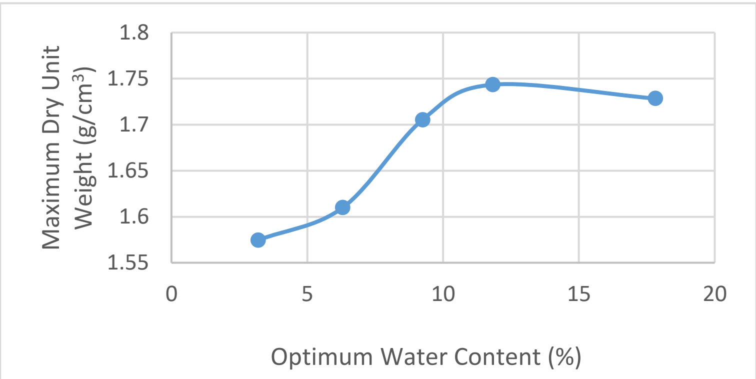


Figure 5.4. Standard proctor test result for sand

According to those graphics, optimum water content of clay and sand is found as 12,5% 11,5% respectively. Maximum dry unit weight is found as 1,85 g/cm^3 and 1,74 g/cm^3 for clay and sandy soils respectively. Maximum dry unit weight and optimum moisture content of mixtures are summarized in the Table 5.4.

Table 5.4. Maximum dry unit weight and optimum water content according to standard proctor test

Mixture	**Maximum Dry Unit Weight (g/cm^3)**	**Optimum Water Content (%)**
100% Sand	1,74	11,5
90% Sand + 10% Clay	1,80	9,5
80% Sand + 20% Clay	2,07	8,0
70% Sand + 30% Clay	2,13	6,8
60% Sand + 40% Clay	2,05	8,0
50% Sand + 50% Clay	2,12	8,6
40% Sand + 60% Clay	2,06	9,1
30% Sand + 70% Clay	2,02	10,0
20% Sand + 80% Clay	2,00	11,0
10% Sand + 90% Clay	1,99	10,0
100% Clay	1,85	12,5

After determination of optimum water content and maximum dry unit weight, change of maximum dry unit weight with respect to tyre chip is determined. In this part, 10%, 20%, 30% and 40% chip tyre content by mass is added into each mixture.

After that, standard proctor test is performed. Optimum water content is added into each ratio with respect to mass of soil. When the 40% chip tyre is added into sand, unit weight decreased to 1,20 g/cm^3. Unit of clay decreased to 1,39 g/cm^3 when 40% chip tyre is added. Unit weights of all mixtures for all the chip tyre content is given in Table 5.5.

Table 5.5. Unit Weight of Soil – Tyre Chip Mixtures

Tyre Chip Content				
Mixture	**10%**	**20%**	**30%**	**40%**
100% Sand (g/cm^3)	1,55	1,47	1,22	1,20
90% Sand + 10% Clay (g/cm^3)	1,75	1,56	1,33	1,27
80% Sand + 20% Clay (g/cm^3)	1,84	1,62	1,42	1,40
70% Sand + 30% Clay (g/cm^3)	1,91	1,69	1,46	1,42
60% Sand + 40% Clay (g/cm^3)	1,78	1,72	1,50	1,44
50% Sand + 50% Clay (g/cm^3)	1,90	1,69	1,50	1,42
40% Sand + 60% Clay (g/cm^3)	1,91	1,70	1,59	1,45
30% Sand + 70% Clay (g/cm^3)	1,85	1,74	1,58	1,44
20% Sand + 80% Clay (g/cm^3)	1,65	1,67	1,55	1,43
10% Sand + 90% Clay (g/cm^3)	1,79	1,64	1,52	1,40
100% Clay (g/cm^3)	1,79	1,64	1,50	1,39

As it can be seen from the table above, as the content of chip tyre is increased, unit weight of the mixture decreased.

5.1.5 Direct shear test

Strength properties of soils can be determined by triaxial test or direct shear test. However, in case of mechanically stabilized walls, failure conditions are more compatible with direct shear test conditions. Therefore, in this study, strength parameters of each mixtures are determined by direct shear test.

Strength parameters determined as drained parameters. Therefore, speed of the test is very important. Two different speed is selected during experiments according to Czech standard for direct shear test (CSN CEN ISO/17892-10 [71]) because, shear force has to be applied slowly to clayey soils in order to dissipate excess pore water pressure. Selected speeds can be given as 0,25 mm/min for sandy soils and 0,065 mm/min for clayey soils. Those speeds are also applied to mixtures with chip tyre content so that results can be comparable.

Confining stress is chosen as 9,81 kPa, 19,62 kPa, 40,81 kPa and 58,86 kPa to find stiffness and strength of soil under low confining stress, because most of the pull – out failure occurs at the top of wall where confining pressure is lower. Those pull – out failures lead to general failure of wall. All the samples are tried to be compacted to its maximum unit weight. Minimum 97% average compaction is accomplished for all soil subjects before direct shear test. All unit weights and average compaction ratio can be seen on the Table 5.6.

Table 5.6. Unit weight and compaction ratio of the direct shear test samples

Mixture	γ_{unsat} (g/cm^3)	9,81kPa γ(g/cm^3)	19,62kPa γ (g/cm^3)	40,81kPa γ (g/cm^3)	58,86kPa γ (g/cm^3)	Average γ (g/cm^3)	Compaction Ratio (%)
100% Sand	1,74	1,71	1,71	1,70	1,72	1,71	98,28
90% Sand + 10% Clay	1,80	1,73	1,78	1,76	1,76	1,76	97,64
80% Sand+ 20% Clay	2,07	2,05	2,05	2,05	2,03	2,05	98,79
70% Sand + 30% Clay	2,13	2,11	2,10	2,10	2,09	2,10	98,59
60% Sand + 40% Clay	2,05	2,04	2,03	2,03	2,03	2,03	99,15
50% Sand + 50% Clay	2,12	2,12	2,12	2,12	2,10	2,12	99,76
40% Sand + 60% Clay	2,06	2,05	2,05	2,04	2,05	2,05	99,39
30% Sand + 70% Clay	2,02	2,01	2,01	2,00	2,00	2,01	99,26
20% Sand + 80% Clay	2,00	1,98	1,97	1,96	1,97	1,97	98,50
10% Sand + 90% Clay	1,99	1,95	1,93	1,94	1,95	1,94	97,61
100% Clay	1,85	1,86	1,81	1,82	1,83	1,83	98,92

After the direct shear test, as the clay content of the mixture increases; cohesion increases and angle of friction decreases.

Angle of friction of sand is found as 47,38 degrees while cohesion is found as 0,456 kPa. Angle of friction of compacted clay is found as 32,44 degrees and its cohesion is found as 37,66 kPa. Cohesion value is not expected on sand samples normally. However, results of this study showed small value of cohesion. The reason of it is called as apparent cohesion. Apparent cohesion is seen due to surface tension of water particles covering sand particles. It should be noted here that; apparent cohesion is acceptable up to 2 kPa. Higher values of cohesion than 2 kPa indicate an error during experiment.

When 10% tyre chip is added in to soil mixture, it is seen that angle of friction decreases for all type of soil. However, cohesion value showed two different behaviour according the content of clay content. When the clay content is lower than 50%, cohesion value increased. However, when the clay content is equal or higher than the sand content, cohesion value decreased compared with virgin samples. When tyre chip content is increased to 20%, internal angle of friction increased for all mixtures when compared both for virgin sample and sample with 10% tyre chip. Cohesion value of sand and 90% sand 10% clay increased when compared with virgin samples. However, all of the cohesion values are smaller than the 10% tyre chip content. When the chip tyre content is increased to 30%, a decrease in angle of friction is seen when compared with 20% chip tyre content. The strength values for virgin sample and samples with different tyre chip content is given in Table 5.7.

Table 5.7. Shear Strength Parameters of soil and soil – tyre chip mixtures

	0 % Tyre Chip		**10% Tyre Chip**		**20% Tyre Chip**		**30% Tyre Chip**	
	ϕ (°)	**c (kPa)**	**ϕ (°)**	**c (kPa)**	**ϕ (°)**	**c (kPa)**	**ϕ (°)**	**c (kPa)**
100% Sand	47,38	0,46	45,39	7,39	47,74	5,86	41,16	6,64
90% Sand + 10% Clay	44,19	3,07	43,84	13,36	46,23	9,73	41,01	7,09
80% Sand + 20% Clay	42,35	11,61	42,17	17,46	46,02	11,56	39,89	8,88
70% Sand + 30% Clay	41,36	21,51	41,04	21,93	44,80	12,50	38,35	11,61
60% Sand + 40% Clay	41,19	24,01	40,70	23,09	43,30	17,11	38,06	17,86
50% Sand + 50% Clay	41,04	26,68	39,86	24,63	42,08	20,26	37,09	19,88
40% Sand + 60% Clay	38,84	25,84	37,78	25,74	41,72	20,63	36,69	20,02
30% Sand + 70% Clay	38,05	26,10	37,45	25,66	41,14	22,58	36,27	23,52
20% Sand + 80% Clay	36,51	34,36	35,61	25,76	40,62	22,64	36,07	26,56
10% Sand + 90% Clay	36,37	37,48	35,54	25,76	38,62	23,69	35,41	26,71
100% Clay	32,44	37,66	33,37	30,19	37,47	28,14	34,60	28,63

The graphics of shear stress – horizontal displacement, horizontal displacement – vertical displacement and maximum shear stress – confining stress are given on attachment 1.

When the failure displacement is investigated of all samples, it is seen that, as the confining pressure is increased, failure displacement also increased. There are only two exceptions on this comment. Those exceptions can be given as samples which are containing 70% and 60% sand without tyre chip content. It can also be said that, as the chip tyre content increases, failure displacement increase becomes stronger. The graphics of failure displacement – confining pressure is given on an attachment 2. As the tyre chip content increases, failure deformation is also increasing. This increase becomes more obvious for the higher confining stress. This behaviour can also be seen on attachment 2.

5.2 Finite Element Model

Direct shear tests without tyre chip content is modelled in a finite element code named Abaqus. Abaqus allows its user to model experiments in 3 dimensions. Sand, 80% Sand, 60% Sand, 40% Sand, 20% Sand and Clay samples are selected to be modelled.

Finite element model is created according to true dimensions of laboratory samples. After that, sample is created, it is meshed so that the finite element analysis can be carried out. Geometry and meshed structure of the model are given on Figure 5.5 below. Mohr – Coulomb material model is chosen to model failure of samples. Mohr – Coulomb parameters are given in Table 5.8 for every soil sample modelled in this study.

Table 5.8. Mohr – Coulomb Material Model Parameters for Finite Element Analysis

							9,81 kPa	**19,2 kPa**	**40,81 kPa**	**58,86 kPa**
	γ_{sat} (kN/m^3)	γ_{unsat} (kN/m^3)	**Φ (º)**	**C (kPa)**	**φ (º)**	**ν**	**G** (MPa)	**G** (MPa)	**G** (MPa)	**G** (MPa)
100% Sand	19,50	17,4	47,38	0,456	17,38	0,3	4.04	7.20	13.5	14.41
80% Sand + 20% Clay	22,31	20,7	42,35	11,61	12,35	0,3	4.74	6.58	11.88	13.22
60% Sand + 40% Clay	22,10	20,5	41,19	24,01	11,19	0,3	7.76	11.04	13.77	24.08
40% Sand + 60% Clay	22,53	20,6	38,84	25,84	8,84	0,3	10.44	12.58	11.36	18.84
20% Sand + 80% Clay	22,10	20,0	36,51	34,36	6,51	0,3	13.30	15.59	22.5	22.93
100% Clay	20,80	18,5	32,44	37,66	2,44	0,3	15.73	17.77	21.05	25.39

Direct shear test is modelled at two steps. Vertical load which equal to either 9.81, 19.2, 40.81 or 58.86 kPa is applied to created model. Deformations of side walls of the model are prevented in x and z axis for both bottom and top part of the sample during step one. Shear force is applied during step 2. In order to apply shear force, displacement on z direction is allowed and shear force is applied by using prescribed displacement. Prescribed displacement is selected as 6 mm.

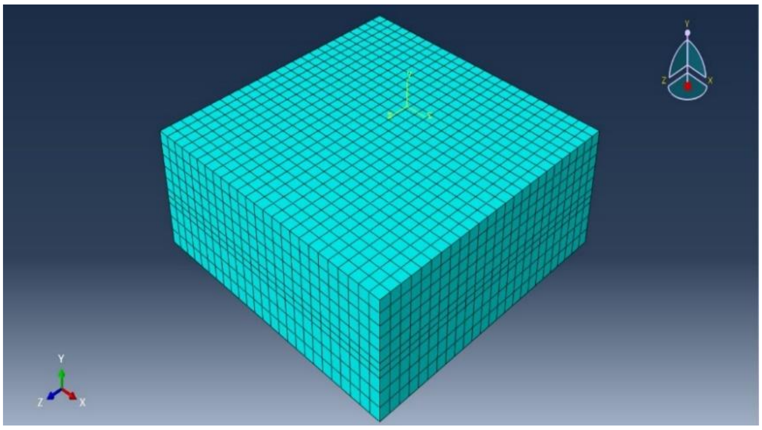


Figure 5.5. Geometry and mesh of finite element model for direct shear test

Mesh sensitivity is checked before performing finite element analysis of the models. In order to find out mesh sensitivity of model, part is seeded in each 12mm, 10mm, 8mm, 6mm, 4mm, 2mm and 1mm. Mesh size refers to dimension of element before application of any type of loading. Shear tests are conducted for each mesh size for sand sample under 58.86 kPa. It is seen that, as size of mesh decreased computed shear stress decreased. However, when mesh size is selected as 1mm, calculated shear stress increased enormously. Mesh sensitivity is given on Figure 5.6 below.

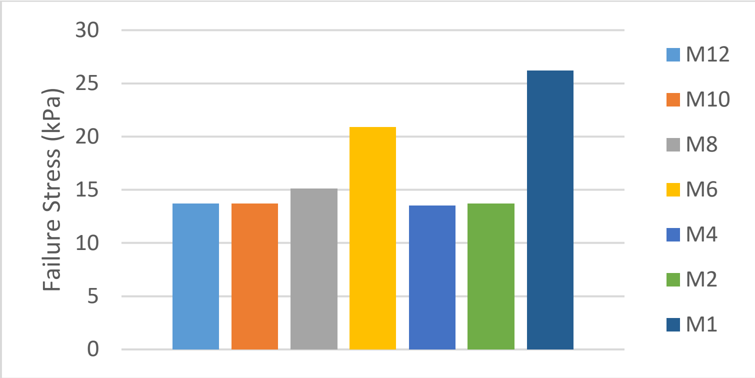


Figure 5.6 Geometry and mesh of finite element model for direct shear test

As it can be seen from Figure 5.6, M4 and M2 produced almost same results. Therefore, M4 is selected during this study to save time. When the results of the Abaqus analysis is investigated, it is seen that Abaqus produced quite good results. As the shear stress – displacement graphics are investigated, finite element results comply with experimental results. Table 5.9 shows measured peak shear stress at laboratory and computed peak shear stress by Abaqus.

Table 5.9. Maximum shear stress values from experiment and finite element model and difference

Content	Confining Pressure (kPa)	Experiment (kPa)	Abaqus (kPa)	Difference (%)
100% Sand	**9,81**	11	10,8	1.82
	19,62	22,7	24,8	9.25
	40,81	43,1	44,9	4.18
	58,86	65,3	64,9	0.61
80% Sand + 20% Clay	**9,81**	19,2	17,7	7.81
	19,62	30,4	27,4	9.87
	40,81	50,5	46,8	7.33
	58,86	64	63,3	1.09
60% Sand + 40% Clay	**9,81**	34,7	34,8	0.29
	19,62	40,3	41	1.74
	40,81	55,9	59,5	6.44
	58,86	78,1	80,2	2.69
40% Sand + 60% Clay	**9,81**	31,6	32,6	3.16
	19,62	43,2	42,1	2.55
	40,81	61,1	60,1	1.64
	58,86	71,4	72,7	1.82
20% Sand + 80% Clay	**9,81**	40,7	39,1	3.93
	19,62	49,1	45,9	6.52
	40,81	66,6	74,1	11.26
	58,86	76,6	73,7	3.76
100% Clay	**9,81**	44,6	45,3	1.57
	19,62	50,4	49,7	1.39
	40,81	61,1	61,2	0.16
	58,86	76,6	79,5	3.79

Table 5.10. Shear strength parameters from experiment and finite element model and difference

	Experiment		Abaqus		Difference (%)	
Content	**φ (º)**	**c (kPa)**	**φ (º)**	**c (kPa)**	**φ (º)**	**c (kPa)**
100% Sand	47.4	0.46	48.2	0	1.7	100,00
80% Sand + 20% Clay	42.4	11.61	43.0	8.80	1.5	24.20
60% Sand + 40% Clay	41.2	24.01	43.0	23.90	4.4	0.46
40% Sand + 60% Clay	38.9	25.84	39.4	25.40	1.5	1.69
20% Sand + 80% Clay	36.1	34.36	37.5	32.60	3.9	5.12
100% Clay	32.4	37.66	34.5	36.80	6.4	2.28

Highest difference is observed for clay sample as 6.35%, however, this difference is below 10% and acceptable. There can also be seen a 100% difference in cohesion of sand sample. This mistake is due to Abaqus analysis resulted in 0 kPa cohesion, however laboratory testing resulted in 0,456 kPa cohesion. Measured and computed shear strength parameters are given on Table 5.10 above.

Stress – horizontal displacement graphics are provided at the attachment 3 for detailed investigations.

6 MODEL TESTS AT LABORATORY

6.1 Design of Models

Design process of the reinforced earth walls will be described in following sub-sections. However, properties of geotextile should be given briefly. 100% polyester (PET) type of geotextile is used during the study. Geotextile is obtained from company BONTEC. The product is called as Bontec HS 110/50. Tensile strength of geotextile is given as 110 kN/m longitudinally and 50 kN/m laterally. Tensile strength is obtained when at 10.5% and 11% strain.

6.1.1 External Design of Models Walls

After determination of mechanical properties of soil and soil tyre crumb mixtures, geosynthetically reinforced soil walls are designed with respect to federal highway administration for sand and clay backfill. Soil properties which are evaluated at the laboratory used during design stage. Height of the walls selected as 450 mm which comply with some of the literature. The length of geotextiles is selected as 0.7H, which is the minimum length of reinforcement under static load. External safety of the wall, such as safety against overturning and safety against sliding are evaluated. Safety against bearing capacity is not evaluated because, steel frame is used at the laboratory. Therefore, bearing capacity failure is not among the possible failure cases. The steel frame at the laboratory is given on Figure 6.1.

Figure 6.1 Steel Frame Used at the laboratory

In order to check internal stability of the wall, geotextiles are placed at every 50 mm which corresponds to 9 layers. The horizontal forces for each layer are computed. The calculations regarding safety against overturning is given on table 6.1 below.

Table 6.1 Results of safety against overturning

	Sand Backfill	Clay Backfill
Resisting Moment (kNm/m)	9.79	9.82
Overturning Moment (kNm/m	0.06	0.06
Limiting Eccentricity (m)	0.079	0.079
Eccentricity (m)	0.01	0.01
Total Vertical Force (kN/m)	63.94	64.1

According to these results, it may be said that, both of designed walls are safe against overturning. After check against overturning, safety check against sliding is conducted. Sliding check is conducted as comparing calculated horizontal force acting on wall and resisting forces against moving. Forces against movement equals to vertical force multiplied by tangent of lower angle of friction angle. Those values are provided on Table 6.2.

Table 6.2 Results of safety against sliding

	Sand Backfill	Clay Backfill
Lateral Load on Wall	0.4	0.4
Sliding Resistance of Wall	2.68	2.85

When table 6.2 is evaluated, it is seen that, designed walls are safe against sliding which is deducted from that limiting value is much higher than driving forces.

6.1.2 Internal Design of Model Walls

Since external design is completed and wall is found to be safe, internal design of model walls can be done in this section. Internal design covers determination of maximum horizontal force acting on each geotextile and computation of pull-out resistance of considered geotextile. It should be remembered that, computed horizontal forces should be lower than the maximum force that a geotextile can bear and pull-out capacity of considered layer. Maximum force on geotextile forms due to active horizontal force from reinforced soil and component of the point load acting over wall. Calculated maximum horizontal stresses are given on table 6.3 for sand.

Table 6.3 Calculation of Maximum Horizontal Force on Geotextile for Sand

Depth of Geotextile (m)	Vertical Stress (kN/m^2/m)	Horizontal Stress-Soil (kN/m^2/m)	Horizontal Stress-Point Load (kN/m^2/m)	Total Horizontal Stress (kN/m^2/m)	Horizontal Force (kN/m)
0.05	0.87	0.13	15.49	15.62	1.56
0.10	1.74	0.26	14.44	14.71	1.47
0.15	2.61	0.40	13.53	13.93	1.39
0.20	3.48	0.53	12.72	13.25	1.33
0.25	4.35	0.66	12.01	12.67	1.27
0.30	5.22	0.79	11.37	12.16	1.22
0.35	6.09	0.93	10.80	11.72	1.17
0.40	6.96	1.06	10.28	11.34	1.13
0.45	7.83	1.19	9.81	11.00	1.10

It is clear from Table 6.3 that, for horizontal load on geotextile layer, the highest load is on the first layer and the lowest is at the deepest layer. This is because of the nature of the stress increase due to point load applied. The highest and the lowest load are computed as 1.56 kN/m and 1.10 kN/m respectively.

Since we know the magnitudes of maximum forces on geotextile, we should compare them with the pull-out capacity of each layer. The pull – out capacity of considered layer may be calculated by following formula.

$$P_r = F^{*}\alpha\sigma_v L_e C$$

where;

P_r = pull-out capacity (kN/m)

F^{*} = Pull – out resistance factor

α = scale effect correction factor

σ_v = The vertical effective stress (kN/m^2)

L_e = Embedment length of the geosynthetic in resisting zone (m)

C = Reinforcement effective unit parameter

The pull – out resistance factor may be determined by experiments or may be computed as following conservatively. There might also be other formulas in literature.

$$F^* = {}^{2}/_{3} \tan \phi$$

Reinforcement effective unit parameter is associated with type of reinforcement and its area which transforms forces to soil. Since geotextile sheet is used in this study, this value may be taken as 2. This means that, whole area of reinforcement transfers load to soil. The part of applied load which is inside the resisting zone is also used to calculate pull – out capacity of geotextile. Calculated pull-out capacities for each geotextile layer is given in Table 6.4 below.

Table 6.4 Pull – out capacity of geotextiles for sand backfill

Depth of Geotextile (m)	L_e (m)	F^*	α	$\sigma_v (kN/m^2)$	P_r (kN/m)
0.05	0.15896609	0.73	0.65	52.24	7.86
0.1	0.17847033	0.73	0.65	55.52	9.38
0.15	0.19797456	0.73	0.65	58.49	10.96
0.2	0.2174788	0.73	0.65	61.21	12.60
0.25	0.23698304	0.73	0.65	63.72	14.29
0.3	0.25648728	0.73	0.65	66.06	16.04
0.35	0.27599152	0.73	0.65	68.25	17.83
0.4	0.29549576	0.73	0.65	70.32	19.67
0.45	0.315	0.73	0.65	72.27	21.55

Pull – out capacity is highly dependent on effective stress, which increases with respect to depth. Therefore, pull out capacity of geotextile layer increases with depth as well. This behaviour is observed on Table 6.4 too. The lowest pull out capacity is calculated as 7.86 kN/m while maximum force at corresponding layer is calculated as 1.56 kN/m. Those values depict that, geosynthetically reinforced wall is safe against pull – out of geotextile layer.

Since internal check for sand backfill is completed, the results for clay backfill can now be given. The geotextile loads and corresponding pull – out capacity are calculated in a similar manner like sand. However, tension cracks are taken into consideration during calculations. The calculated maximum forces acting on geotextile are given on Table 6.5 below.

Table 6.5 Calculation of Maximum Horizontal Force on Geotextile for Clay

Depth of Geotextile (m)	Vertical Stress (kN/m^2/m)	Horizontal Stress-Soil (kN/m^2/m)	Horizontal Stress-Point Load (kN/m^2/m)	Total Horizontal Stress (kN/m^2/m)	Horizontal Force (kN/m)
0.05	0.93	-55.69	30.84	-24.85	-2.48
0.10	1.85	-55.41	28.76	-26.65	-2.67
0.15	2.78	-55.13	26.94	-28.19	-2.82
0.20	3.70	-54.85	25.33	-29.51	-2.95
0.25	4.63	-54.57	23.91	-30.66	-3.07
0.30	5.55	-54.29	22.64	-31.65	-3.16
0.35	6.48	-54.01	21.50	-32.51	-3.25
0.40	7.40	-53.73	20.46	-33.26	-3.33
0.45	8.33	-53.45	19.52	-33.92	-3.39

Calculated forces are found to be negative because cohesion of clay soil is significantly high. After that, pull-out capacity may be calculated for clay backfill. Calculated pull -out capacities for each geotextile layer is given on Table 6.6.

Table 6.6 Calculated pull-out resistance for clay

Depth of Geotextile (m)	L_e (m)	F^*	α	$\sigma_v(kN/m^2)$	P_r (kN/m)
0.05	0.09482532	0.42	0.65	10.21	0.53
0.10	0.12234716	0.42	0.65	13.02	0.88
0.15	0.14986899	0.42	0.65	15.59	1.29
0.20	0.17739083	0.42	0.65	17.97	1.76
0.25	0.20491266	0.42	0.65	20.18	2.28
0.30	0.2324345	0.42	0.65	22.26	2.85
0.35	0.25995633	0.42	0.65	24.22	3.47
0.40	0.28747817	0.42	0.65	26.07	4.13
0.45	0.315	0.42	0.65	27.85	4.84

Since, angle of friction is significantly lower for clay backfill, effective lengths for geotextile layers are lower than those embedded in sand. The vertical stress is also lower due to same reason. Therefore, calculated pull-out resistance is significantly lower than pull-out resistance calculated for sand.

6.1.3 Laboratory Tests

Seven test samples are produced and tested at the laboratory. The samples were constructed with pure sand, 10% tyre crumb sand mixture, 20% tyre crumb sand mixture, 30% tyre crumb sand mixture, pure clay, 10% tyre crumb clay mixture and 20% tyre crumb clay mixture. Therefore, a total number of seven tests were conducted. 5 cm foundation layer is compacted over a steel frame before construction of samples. Sand is used as a foundation layer too. Compaction is carried out with optimum water content to achieve maximum unit weight. Preparation of foundation layer may be seen on Figure 6.2.

Figure 6.2 Preparation of foundation layer

After foundation layer is constructed, geotextiles are cut into required length. Required length of geotextile is computed as 42 cm. It is a sum of height of single layer, embedment length and overlapping length. Height of the single layer is 5 cm as told earlier, embedment length equals to approximately 32 cm and overlapping length is also defined as 5 cm. Overlapping length can be defined as, after each lifting each layer, wall face should be sealed which can be done by embedding that part of the geotextile into next layer. Overlapping length can be seen on Figure 6.3 respectively below.

When the constructions of walls are completed, loading stage of the test is initiated. During this stage, settlement of loading plate and horizontal displacement of wall face are measured. Horizontal displacements are measured by deformation clocks from three different location. Therefore, change of displacement is observed with respect to height. Load system is given on Figure 6.4. Load system consists of reaction beam, hydraulic pump, load cell and loading plate. Loading plate is cylinder in shape which has 300 mm diameter. It is placed 2 cm away from edge of reinforced earth wall's face. Settlement of loading plate is measure by LVDT located right side of the schema. The steel beam placed under load cell, transferred settlement to the other edge, therefore LVDT was able to measure settlement of plate. Loading is performed starting from 0.06 MPa and increased until the wall failed. Load is increased by 0.06 MPa for each consecutive step.

Figure 6.3 Overlapping length of geotextile

Consecutive loading is started after 1 minute spent under current load. This waiting time allows settlements to end. Settlements, horizontal displacements are recorded after this minute. Since different soil types and different content of the tyre crumb, different walls failed at different loads. Settlement of loading plate also varied with backfill type and tyre crumb content.

As it can be seen on the photos, bricks with openings are placed next to the walls when it is necessary. Openings are filled with ballasts. This brick and ballast are used to provide sufficient stiffness so that the reinforced wall will fail from its face.

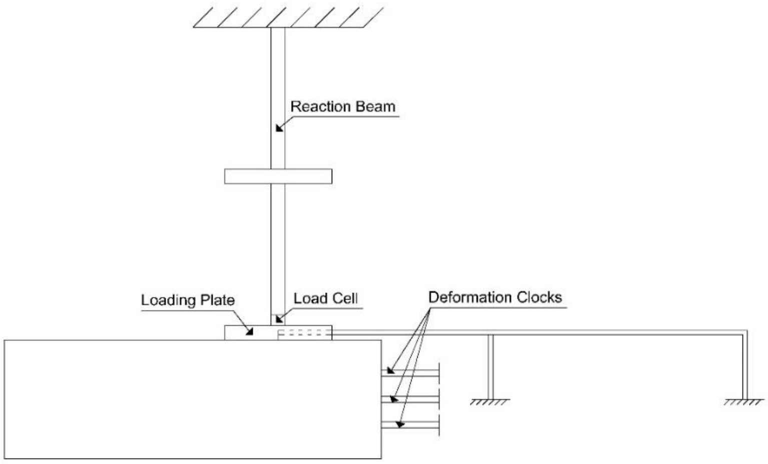


Figure 6.4 Schema of loading system

6.1.3.1 Laboratory Test Results

6.1.3.1.1 Test Results for Sand and Sand – Tyre Crumb Mixtures

The test results for sand and clay backfills will be presented in this section. Sand and sand with tyre crumb mixture results will be given firstly. Secondly, clay and clay with tyre crumb mixtures will be discussed. Finally, results of sand and clay will be compared with each other. Amount of tyre crumb will be taken into consideration too. When reinforced earth wall with sand is tested, maximum load of 0.497 MPa is achieved. Settlement of loading plate is measured as 4.44 mm under 0.497 MPa. The load increment and corresponding settlements measured for sand backfill is given on Table 6.7.

Table 6.7 Load – Settlement values of loading plate

Load (MPa)	Settlement (mm)
0	0
0.055	0.41
0.12	1.09
0.178	1.6
0.236	2.08
0.299	2.65
0.357	3.18
0.425	3.72
0.476	4.19
0.497	4.44

Horizontal displacements of earth wall are measured at three height. The displacement clocks are placed to 32.5 cm, 22.4 cm and 2.5 cm above the ground. Measured horizontal displacement is as high as 1.933 mm, 1.35 mm and 0.669 mm for top, middle and bottom displacement measurement clocks is under 0.497 MPa. Lower displacements are measured under lower loading conditions. Those measured displacements are provided at Table 6.8.

Table 6.8 Horizontal displacement of wall face for sand backfill

Horizontal Displacements (mm)			
Load (MPa)	Top	Middle	Bottom
0	0	0	0
0.055	0.167	0.05	0
0.120	0.444	0.24	0.064
0.178	0.715	0.45	0.139
0.236	0.995	0.65	0.244
0.299	1.275	0.86	0.364
0.357	1.476	1.02	0.459
0.425	1.645	1.16	0.549
0.476	1.86	1.32	0.644
0.497	1.933	1.35	0.669

Displacement starts to increase after second phase of loading for the bottom part of the wall. When reinforced soil is mixed with 10% tyre crumbs, retained wall failed at 0.476 MPa. Same loading sequence is followed as pure sand. Loading plate settled 13.73 mm at this load step which is approximately 3 times higher than pure sand. Settlements of loading plate is given on Table 6.9 below for other loads.

Table 6.9 Settlement of loading plate for sand and 10% tyre crumb backfill

Load (MPa)	Settlement (mm)
0	0
0.069	1.10
0.115	2.24
0.177	4.21
0.236	6.10
0.296	7.98
0.359	9.87
0.416	11.61
0.476	13.73

Horizontal displacement of the wall face is measured from three different location. Displacements are measured from 32.5 cm, 22.5 cm and 2.5 cm above from the ground. Higher displacement is measured at the middle part at first step of the loading. After first loading, horizontal displacement remained constant at the middle, while it increased at the top level. Similarly, as middle point, displacement decreased from 0.098 mm to 0.01 mm when load is increased to 0.115 MPa from 0.069 MPa. Then it remained constant for following load increment. Then it started to increase continuously up to 0.37 mm with respect to increasing load. Measured horizontal displacements for each load steps for each position are given on Table 6.10.

Table 6.10 Measured horizontal displacements for sand and 10% tyre crumb mixture

	Horizontal Displacements (mm)		
Load (MPa)	Top	Middle	Bottom
0	0	0	0
0.069	0.03	0.25	0.09
0.115	0.18	0.23	0.01
0.177	0.48	0.23	0.01
0.236	1.01	0.23	0.07
0.296	1.24	0.24	0.21
0.359	1.24	0.25	0.24
0.416	1.92	0.25	0.31
0.476	2.22	1.89	0.37

Pictures for sand and 10% tyre crumb backfill after failure are provided at Figure 6.5.

Figure 6.5 Pictures of wall with sand backfill and 10% tyre crumb after failure

When sand is mixed with 20% tyre crumb, and used as reinforced soil, wall failed at 0.36 MPa. Loading plate settled 21.81 mm during failure. Measured settlements are provided at Table 6.11 below.

Table 6.11 Measured settlements of loading plate

Load (MPa)	Settlement (mm)
0	0
0.06	4.67
0.12	8.46
0.18	11.86
0.24	15.10
0.30	18.25
0.36	21.81

Horizontal displacements are measured from 3 different points. Those points may be classified as top, middle and bottom because they are placed to 42.5 cm, 22.5 cm and 7.5 cm above the ground. The highest displacements are measured at the top of the wall for each load step. Higher displacement is measured at the bottom than middle for first load step. After that, higher horizontal displacement is measure at the middle than bottom of the wall. The highest measured displacements may be given as 4.22 mm, 3,39 mm and 2.63 mm at the top, middle and bottom respectively under 0.36 MPa. The horizontal displacements for other load steps are provided on Table 6.12.

Table 6.12 Horizontal Displacement for sand 20% tyre crumb mixture

	Horizontal Displacements (mm)		
Load (MPa)	Top	Middle	Bottom
0	0	0	0
0.06	0.96	0.605	0.762
0.12	1.06	1.15	0.79
0.18	1.91	1.23	1.22
0.24	2.085	1.89	1.65
0.3	3.1	2.58	2.16
0.36	4.22	3.39	2.634

Picture of the wall with sand and 20% tyre crumb after failure is shown on Figure 6.6 below.

Figure 6.6 Failed wall after test with sand + 20% tyre crumb mixture as backfill

When sand and 30% tyre crumb mixture as a backfill is tested, settlement of loading plate unfortunately could not be recorded due to technical problems encountered during test. However, horizontal displacements are successfully measured and recorded from three different locations. Horizontal displacements are measured from 37.5 cm, 27.5 cm and 2.5 cm above from the ground. The wall and location of horizontal displacements are shown in Figure 6.7.

Figure 6.7 Location of measurement points of horizontal displacement

The wall failed under 0.24 MPa. The failed wall is shown at Figure 6.8 below.

Figure 6.8 Failed wall for sand and 30% tyre crumb mixture after application of 0.24 MPa load

It is seen that, failure occurred when loading plate undergone excessive settlement. Settlement of loading plate concentrates at the border where sand and tyre chip mixture reinforced soil meets with pure backfill sand. The highest horizontal displacements are recorded under this load. The highest horizontal displacement is always measured at the top. Horizontal displacements are measured under 0.24 MPa may be given as 5.98 mm, 4.335 mm and 2.7 mm from top to bottom respectively. The horizontal displacements are measured for sand and 30% tyre crumb content for every load step is given on Table 6.13 below.

Table 6.13 Horizontal displacements measured during test of sand and 30% tyre crumb

	Horizontal Displacements (mm)		
Load (MPa)	Top	Middle	Bottom
0	0	0	0
0.06	0.81	0.435	0.12
0.12	2.275	0.615	0.475
0.18	4.73	2.395	1.022
0.24	5.98	4.335	2.7

The results are provided for sand and sand with tyre crumb mixture until now. Their results will be compared with each other. The settlement of loading plate is close to each other when sand and 10% tyre crumb sand content is compared at first step two steps of loading. When load equals to 0.06 MPa, measured settlement of loading plate equals to 0.41 mm and 1.1 mm for sand and sand with 10% tyre crumb content respectively. However, as the load increases, difference between measured settlement values increases as well. When 20% tyre crumb content is considered, settlement of loading plate increases more two other cases considered. The change of settlement of loading plate with respect to load is given on Figure 6.9 below.

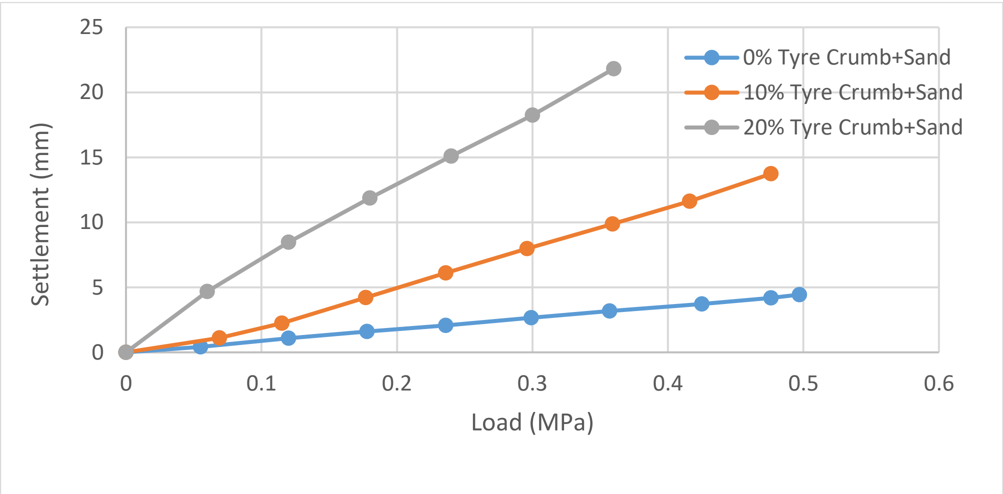


Figure 6.9 Change of settlement of loading plate with respect to load

Horizontal displacement of walls with different tyre crumb content is compared with each other according to measurement point, for example, horizontal displacements of top points are compared with only each other. When horizontal displacement of top point is considered, it seen that displacements almost equal for sand – sand 10% tyre crumb mixture and sand 20% tyre crumb mixture – sand 30% tyre crumb mixture. The change of displacements with respect to depth is given on Figure 6.10.

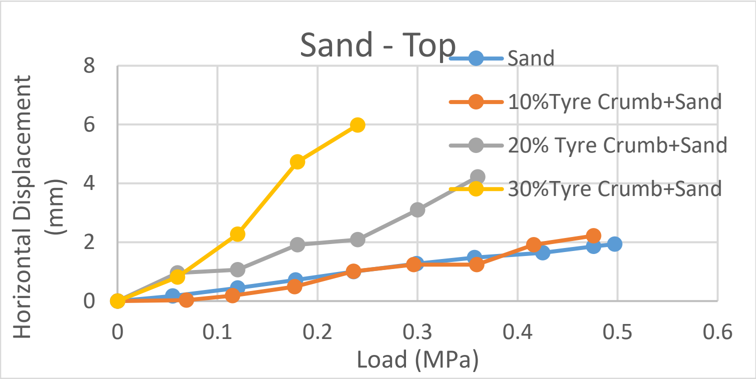


Figure 6.10 Measured horizontal displacements for different tyre crumb content and load

The lowest horizontal displacements are measured for sand – 10% tyre crumb content until 0.42 MPa. After that load step, the lowest horizontal displacement is measured for pure sand. The reason for the change could be that, the reinforced wall with sand 10% tyre crumb content is about to fail or failed at that loading step. Similar behaviour is observed for sand – 20% tyre crumb content and sand – 30% tyre crumb content. Measured horizontal displacement equals to each other up to 0.06 MPa. After that load step, rate of change of horizontal displacement increase for sand – 30% tyre crumb mixture. This change occurs at 0.24 MPa for sand – 20% tyre crumb mixture. However, displacement change remains lower for sand 20% tyre crumb mixture even after 0.24 MPa.

When the displacements are compared according to different tyre crumb content at middle level, the lowest displacements are measured for 10% tyre crumb content. The horizontal displacement increased at the initial load level and then remained constant for sand 10% tyre crumb mixture. After 0.42 MPa, the change of horizontal displacement is more than pure sand backfill, therefore lower horizontal displacement is measured for pure sand backfill content. This behaviour may be seen on Figure 6.11. Horizontal displacements measured for sand – 30% tyre crumb content is higher than pure sand and sand – 10% tyre crumb content. However, it lower when compared with horizontal displacements measured for sand – 20% tyre crumb mixture up to 0.12 MPa. After 0.12 MPa, horizontal displacement measured for sand – 30% tyre crumb content becomes the highest.

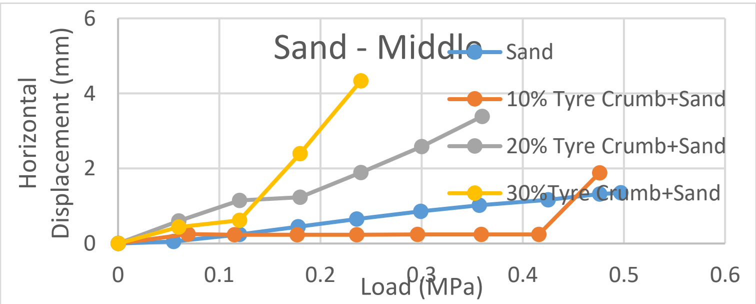


Figure 6.11 Change of horizontal displacement for different tyre crumb content with load

The lowest displacement at the bottom is measured for 10% tyre crumb content. However, there is an exception at 0.06 MPa. The lowest horizontal displacement is measured for pure sand at this load. However, after this point, measured horizontal displacement slightly decreased for sand – 10% tyre crumb mixture. After this load step, increment is lower for sand – 10% tyre crumb content, horizontal displacement remains as the lowest for this mixture. This behaviour may be seen on Figure 6.12.

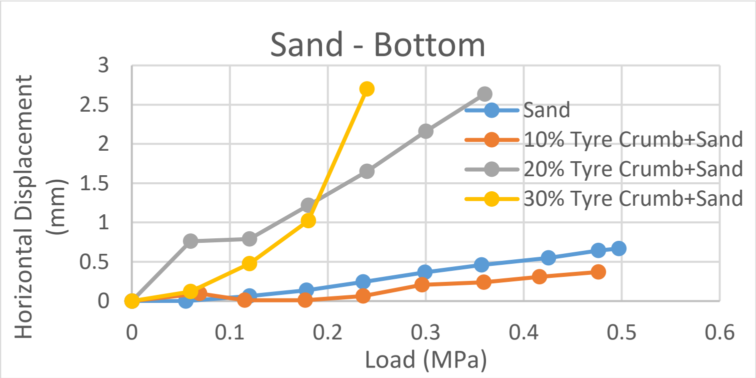


Figure 6.12 Change of horizontal displacement for different tyre crumb content with load

As it can be seen from Figure 5.13, sand – 20% tyre crumb mixture has the highest horizontal displacement at the early load steps. However, after 0.06 MPa, the rate of increase of displacement of sand – 30% tyre crumb mixture becomes higher than sand – 20% tyre crumb content. Therefore after 0.18 MPa, horizontal displacement becomes higher for sand – 30% tyre crumb mixture than sand – 20% tyre crumb mixture at the bottom of the reinforced wall.

6.1.3.1.2 Test Results for Clay and Clay – Tyre Crumb Mixtures

Test results for clay and clay – tyre crumb mixtures will be presented in this chapter. However, it should be noted here that, the part of the wall, where there is not any geotextile reinforcement is constructed with sand. This approach is taken in order to create same driving forces into wall, which are considered during external analysis of the wall. Therefore, the differences on the settlement and horizontal displacements are due to only change of internal design parameters of the reinforced earth wall compared with the sand and sand tyre crumb mixtures. When reinforced earth wall with pure clay backfill is tested, reinforced earth wall failed at 0.188 MPa. Failed wall is shown on Figure 6.13.

Figure 6.13 Failed Reinforced Earth Wall with Clay Backfill

It is seen from the Figure 5.14, failure occurred where clay merges with sand. Loading plate settles 1.62 mm at 0.188 MPa. Settlements measured for other load steps are given at Table 6.14.

Table 6.14 Measured settlements of loading plate for clay backfill

Load (MPa)	Settlement (mm)
0	0
0.06	0.97
0.12	1.28
0.19	1.62

Horizontal deformations are measured from 37.5 cm, 22.5 cm and 12.5 cm above the ground as three points. The highest horizontal displacement is measured as 4.93 mm at the middle point for 0.188 MPa. Measured horizontal displacements for other points and load steps are given on Table .15. It is also clear from Table 5.15; the highest horizontal displacement is measured at the top for 0.06 MPa. The highest horizontal displacement is measure at the middle level for other load steps.

Table 6.15 Measured horizontal displacements for clay backfill

Load (MPa)	Top (mm)	Middle (mm)	Bottom (mm)
0	0	0	0
0.060	0.565	0.29	0.155
0.120	1.775	2.02	1.015
0.188	3.345	4.93	2.635

When clay – 10% tyre crumb content is used as backfill, wall withstand against 0.18 MPa. Loading plate settles 20.83 mm at this load. Settlement values for other load steps are provided on Table 6.16 below.

Table 6.16 Settlement of Loading Plate

Load (MPa)	Settlement (mm)
0	0
0.06	5.79
0.12	13.61
0.18	20.53

Horizontal displacement of wall face is measured from three points such as other tests. Measurement points are located 37.5 cm, 22.5 cm and 12.5 cm above the ground. Horizontal displacements are measure as 11.395 mm, 8.642 mm and 5.905 mm at top, middle and bottom respectively for 0.18 MPa. Measured horizontal displacements for other load levels are provided on table 6.17.

Table 6.17 Horizontal Displacements of Wall face with Clay + 10% Tyre Crumb Mixture

Load (MPa)	Top (mm)	Middle (mm)	Bottom (mm)
0	0	0	0
0.06	0.672	0.752	0.468
0.12	5.44	4.222	2.285
0.18	11.395	8.642	5.905

The wall failed at merging point between clay + 10% tyre crumb mixture backfill and retained sand. Failure cracks may be seen on Figure 6.14.

Figure 6.14 Failure Cracks of Clay + 10% Tyre Crumb Mixture

When clay is mixed with 20% tyre crumb content, wall failed when 0.06 MPa load is applied. The failed wall and cracks may be seen on Figure 6.15. Failure occurred where clay + 20% tyre crumb mixture merges with retained sand soil.

Figure 6.15 Failure cracks of Clay + 20% Tyre Crumb Mixture

Loading plate settled 8.64 mm under 0.06 MPa. Horizontal displacements are measured as 2.89 mm, 1.51 mm and 0.9 mm respectively. Those values are measured from 37.5 cm, 22.5 cm and 12.5 cm above the ground. When the results of clay backfill with different tyre crumb are compared, it is seen that, better performance is evaluated for pure clay reinforced soil. When settlement of plate is compared, huge difference between settlement values are observed between clay and clay + 10% tyre crumb content. The settlement values are compared on Figure 6.16 for all clay and clay – tyre crumb mixtures. Slope of settlement – load graph for pure clay seems to be equal to zero when compared with the clay + 10%. The rate of increase of settlement for 20% tyre crumb content is even higher, but not visible for higher load steps.

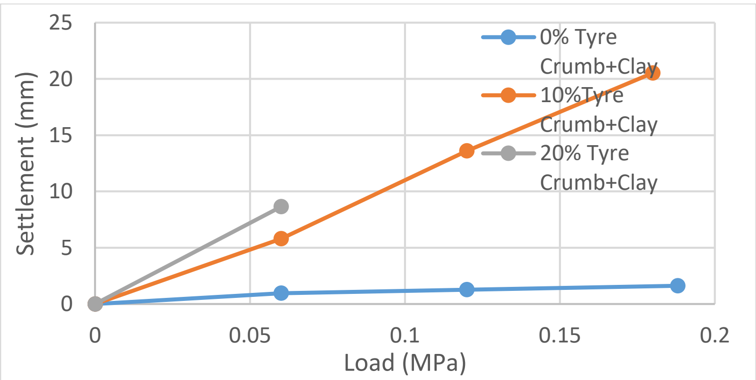


Figure 6.16 Loading plate settlement with respect to different loadings.

When horizontal displacements are compared, it seen that, measured horizontal displacement is almost equal to clay and clay – 10% tyre crumb mixture for 0.06 MPa. However, as the applied load increases, higher horizontal displacement is measured for clay – 10% tyre crumb mixture. The highest horizontal displacement is measured for clay – 20% tyre crumb mixture. However, this could be visible only up to 0.06 MPa. Horizontal displacements for top of the reinforced earth wall are given on Figure 6.17.

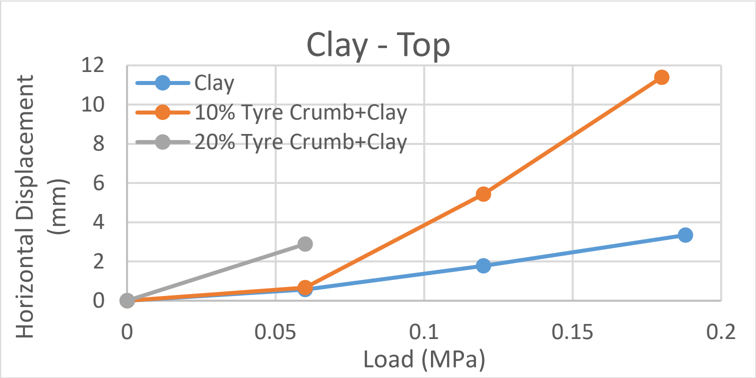


Figure 6.17 Measured horizontal displacements for clay tyre crumb mixture at top of wall

Where horizontal displacements at the middle part, similar behaviour are observed like mentioned above. The lowest horizontal displacement is measured for pure clay backfill. It is should be noted that difference between measured displacements of clay – 10% tyre crumb mixture and clay – 20% tyre crumb mixture is lower. Measured horizontal displacements are shown on Figure 6.18 below.

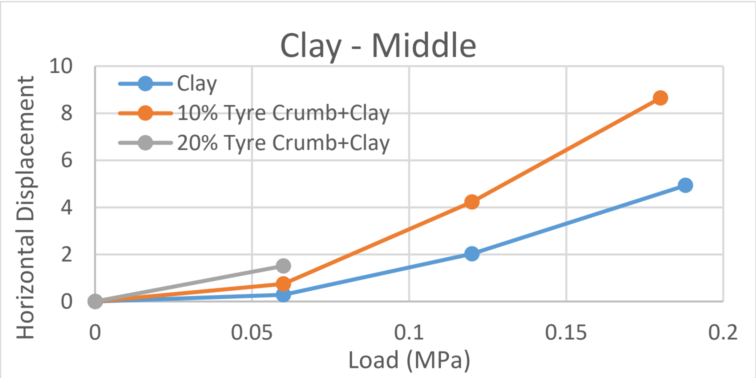


Figure 6.18 Measured horizontal displacements for clay tyre crumb mixture at middle of wall

The horizontal displacement of bottom part is shown on Figure 6.19. Similar behaviour is observed such as displacements at the middle part of the reinforced earth wall. The lowest horizontal displacements are measured for pure clay sample.

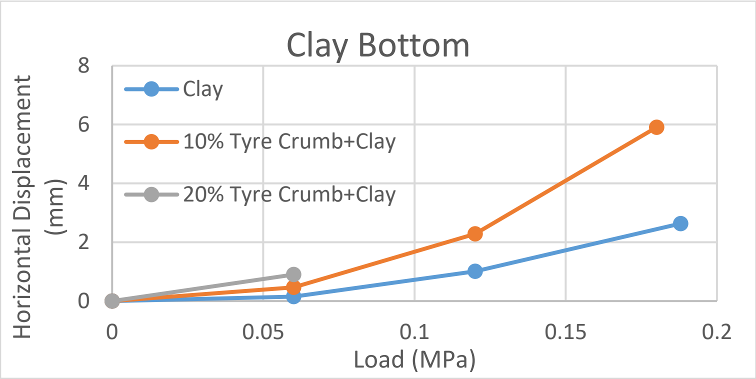


Figure 6.19 Measured horizontal displacements for clay tyre crumb mixture at bottom of wall

6.1.3.1.3 Comparison of Results Measured for Sand and Clay

The results of tests with sand and clay backfill and their mixture with tyre crumbs at same ratio are compared in this section. When behaviour reinforced earth walls with pure sand and pure clay backfills are compared, it seen that loading plate settled 0.41 mm and 0.97 mm for sand and clay backfill respectively for 0.06 MPa. This means that, plate settled more than two times for clay backfill when compared with sand backfill. It can be seen on Figure 6.20 below.

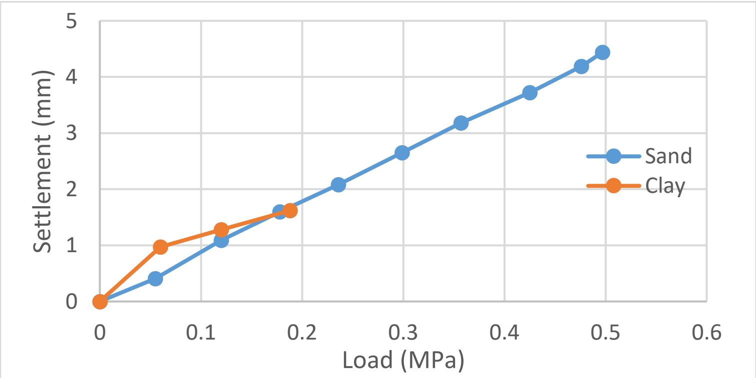


Figure 6.20 Comparison of settlements of loading plate for pure sand and pure clay

However, this ratio decreases for the further loading. The ratio of difference decreases to 17% when load increases to 0.12 MPa. Further increase in load resulted in equivalence of settlements measured for pure sand and clay backfills. However, reinforced earth wall with clay backfill failed at 0.18 MPa, therefore comparison can not be made for further loading steps.

If the settlement of loading plate for backfills mixed with 10% tyre crumb are compared, settlement of loading plate with clay – 10% tyre crumb backfill found to be as high as 426% than sand – 10% tyre crumb content at the 0.06 MPa. The change of settlement with respect to load step is given on Figure 6.21 below. It is clear from the figure that, as the magnitude of load increases, the difference between measured settlement increases. When load reaches to 0.18 MPa, the calculated difference decreases to 388% as a ratio, but in magnitude it becomes 16.32 mm.

Plate settlement is comparable only for 0.06 MPa when sand – 20% tyre crumb mixture and clay – 20% tyre crumb mixture is taken into consideration, because reinforced earth wall with clay – 20% tyre crumb content failed at that load level. Loading plate settled 4.67 mm and 8.67 mm when sand – 20% tyre crumb mixture and clay – 20% tyre crumb mixture is considered respectively. When sand is mixed with 20% tyre crumb, reinforced earth wall was able to withstand to forces up to 0.36 MPa.

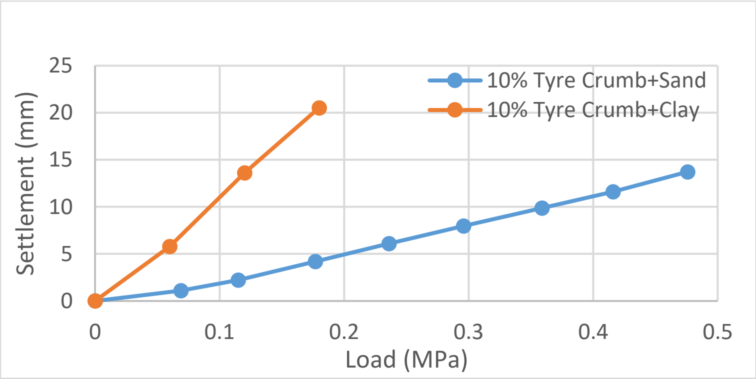
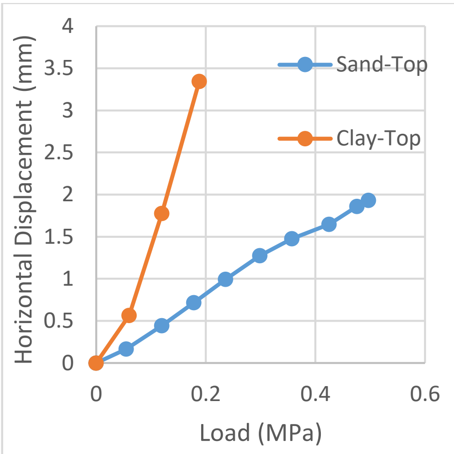
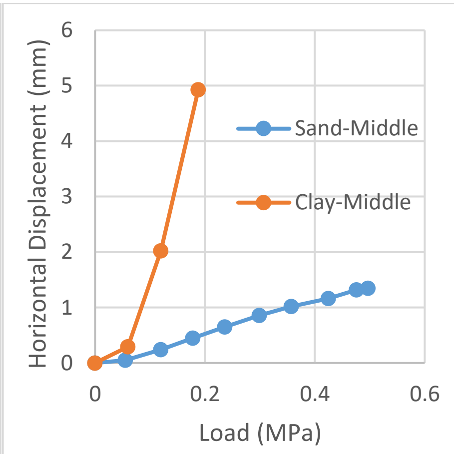


Figure 6.21 Comparison of settlements of loading plate for sand -10% tyre crumb and clay – 10% tyre crumb

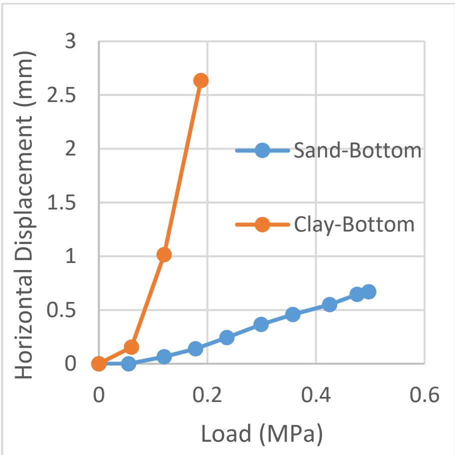
Unlike to settlement of loading plate, there always great difference is observed at level of loading and every section where measurements are conducted. When pure sand backfill and pure clay backfill are considered, it is seen that horizontal displacements increases tremendously after 0.06 MPa, while linearity of horizontal displacement – load curve remains constant. Comparison of horizontal displacements for top, middle and bottom sections are given on Figure 6.22. The difference of horizontal displacements at the top found to be as 0.398 mm at 0.06 MPa and it increased to 2.63 mm at 0.18 MPa. When middle level is considered, horizontal displacements are measured to be 0.29 mm and 0.05 mm for clay and sand backfill. The difference between them is found as 0.24 mm. When load is increased to 0.18 MPa, horizontal displacements increased to 4.93 mm and 0.447 mm for clay and sand respectively. Therefore, computed horizontal displacement difference increased to 4.483 mm. The horizontal displacement remains zero at the bottom of the wall when load is increased to 0.06 MPa for sand backfill, while it increases to 0.155 mm for clay backfill. When load is increased to 0.18 MPa, horizontal displacement of wall at the bottom with clay backfill reaches to 2.635 mm. Horizontal displacement is measured as 0.139 mm for sand backfill. The difference between horizontal deformations for different backfill soil types becomes 2.496 mm.

When horizontal displacement of sand – 10% tyre crumb and clay – 10% tyre crumb mixtures are investigated, it is seen that, measurements of horizontal displacements are higher for clay – 10% tyre crumb mixture than 10% sand – tyre crumb mixture. 11.395 mm horizontal displacement is measured for clay – 10% tyre crumb while 0.484 mm horizontal displacement is measured for sand – 10% tyre crumb mixture under 0.18 MPa at top point. When middle point is considered similar behaviour is observed for clay – 10% tyre crumb mixture. However, when compared with sand – 10% tyre crumb, the rate of increase of horizontal displacement is higher in case of clay – 10% tyre crumb. 0.23 mm and 8.642 mm horizontal displacement is measured under 0.18 MPa at middle point. When bottom point is considered, zero horizontal displacement is measured for sand – 10% tyre crumb content while 5.905 mm horizontal displacement is measured for clay – 10% tyre crumb mixture backfill.





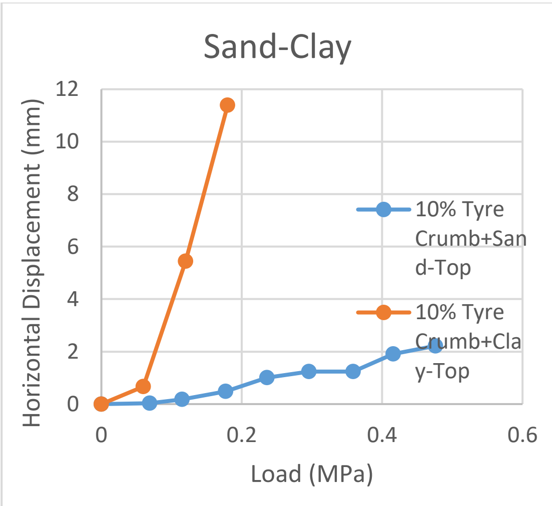
(a) (b)

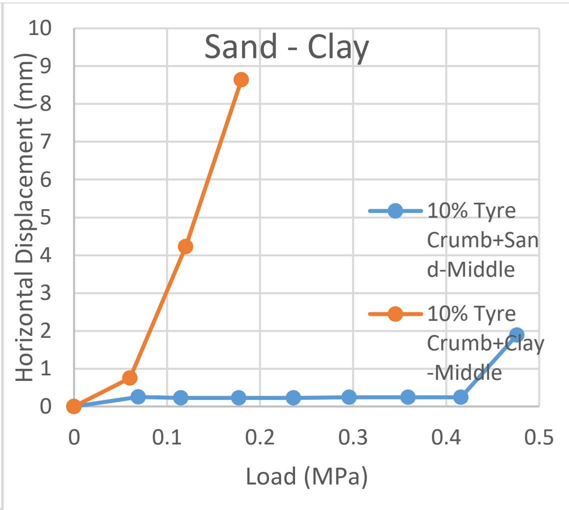


(c)

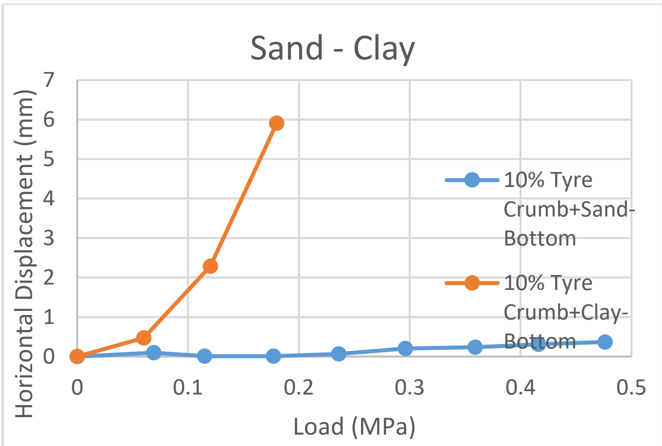
Figure 6.22 Comparison of horizontal displacements of pure sand and pure clay backfills (a) Top (b) Middle (c) Bottom

The graphics of sand – 10% tyre crumb and clay – 10% tyre crumb mixture is given on Figure 6.23 below. When behaviour of retained earth wall with sand – 20% tyre crumb mixture and clay – 20% tyre crumb content is compared, it is seen that, 3 mm horizontal displacement is measured for sand under 0.3 MPa while, same amount of horizontal displacement is measured under 0.06 MPa for clay – 20% tyre crumb content at the top point of reinforced earth wall. When middle measurement point is considered, 1.5 mm horizontal displacement is measured for clay – 20% tyre crumb mixture and 0.605 mm horizontal displacement is measured for sand – 20% tyre crumb mixture. When bottom point is taken into consideration, 0.9 mm horizontal displacement is measured for clay – 20% tyre crumb mixture under 0.06 MPa. 0.12 mm horizontal displacement is measured for sand – 20% tyre crumb mixture under same load.





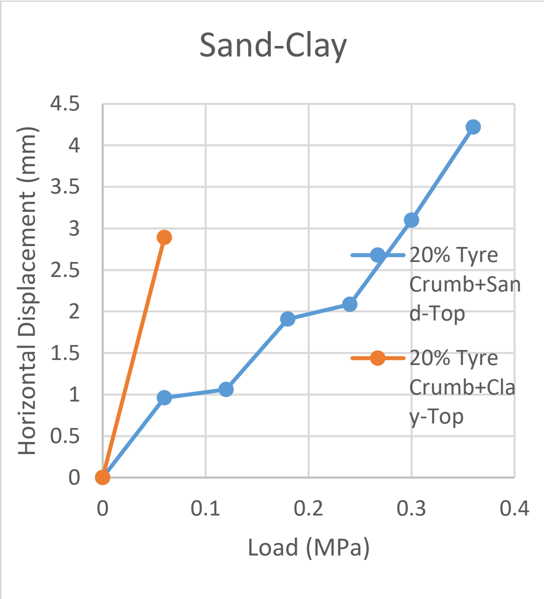
(a) (b)



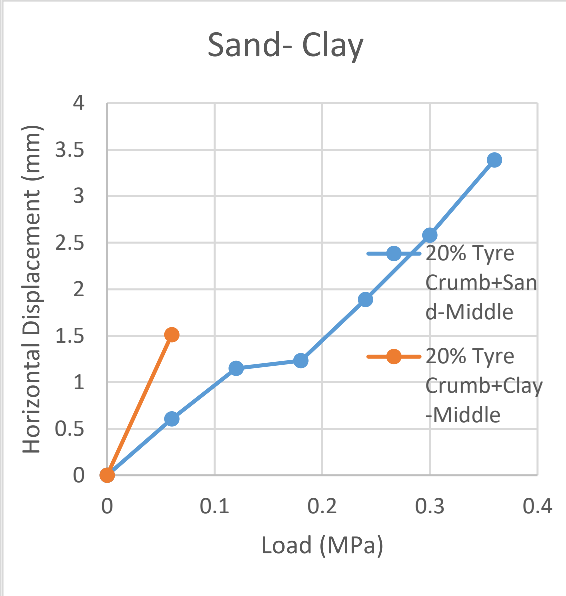
(c)

Figure 6.23 Comparison of horizontal displacements of sand + 10% tyre crumb mixture and clay – 10% tyre crumb mixture backfills
(a) Top (b) Middle (c) Bottom

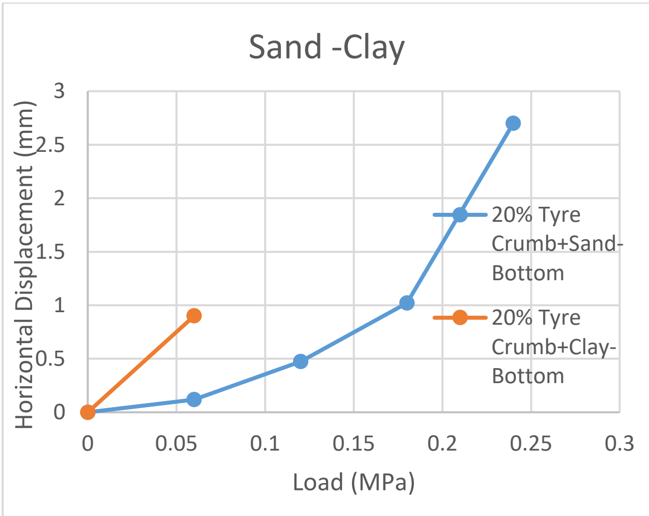
In order to measure 1 mm horizontal deformation in case of sand – 20% tyre crumb mixture, 0.18 MPa load should be exerted to reinforced earth wall. The change of horizontal displacement with respect to load and different backfills may be seen on Figure 6.24.



(a)



(b)



(c)

Figure 6.24 Comparison of horizontal displacements of sand + 20% tyre crumb mixture and clay – 20% tyre crumb mixture backfills
(a) Top (b) Middle (c) Bottom

6.2 Finite Element Modelling of Experiments

Laboratory tests will be modelled in this section. However, since addition of tyre crumbs into soil changes stress – strain behaviour of the soil, tests containing tyre crumbs will not be modelled. Reinforced earth walls without tyre crumb content are modelled in a finite element code named Plaxis. Plaxis is initially developed at University of Delft in Netherlands, however, it is updated by worldwide research programmer's results. Plaxis offer two types of modelling technique which are called as plane strain and axisymmetric models. While plane strain modelling is more appropriate for such a structure like retaining walls, embankments, where length of the structure is much higher than width of structure, axisymmetric model allows user to analyses structures like circular footing or bearing capacity of single pile. Therefore, plane – strain model is preferred during this study. First of all, geometry of the model is created in Plaxis. One of created model is shown on Figure 6.25 below as an example.

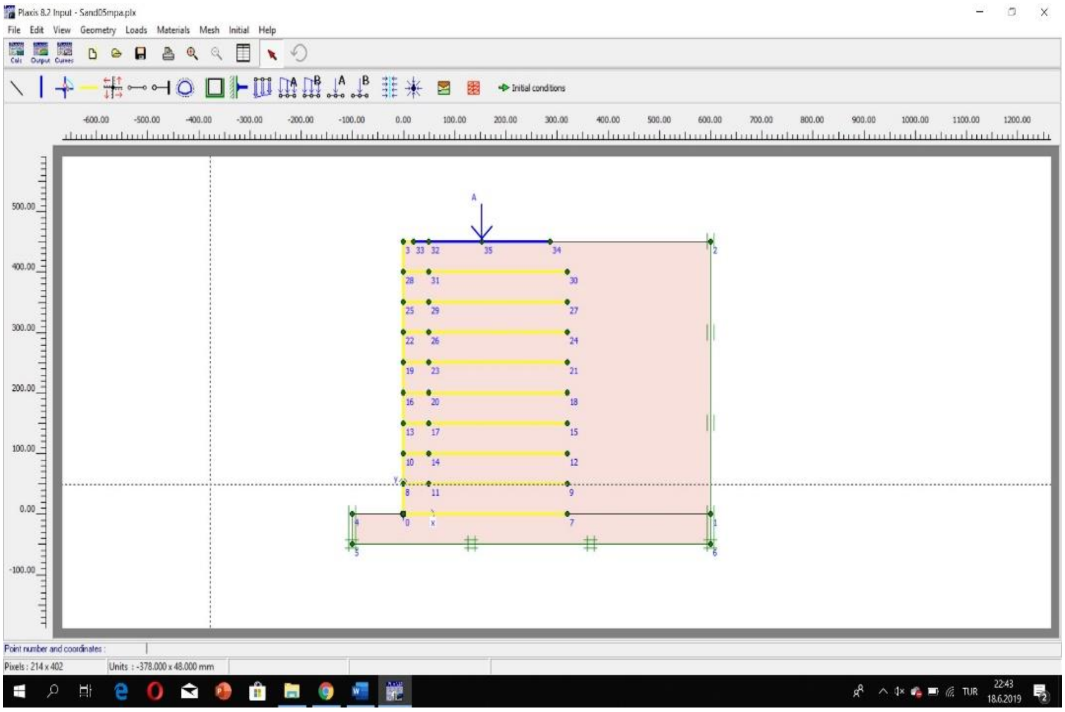


Figure 6.25 Geometry of the created model

Geometry of model also consists of boundary conditions. Boundary conditions are defined as following. Displacements are restrained in x and y directions at bottom, while displacements are restrained at only in x direction. Geotextile is applied into geometry by geogrid element of software. Overlapping geotextile layer is also defined at the geometry by increasing stiffness two times. The loading plate is also introduced into geometry. Then load is applied to plate. Point load type is used to simulate loading stage. Mohr-Coulomb material model is used to model soil. Loading plate and geotextile are modelled as an elastic material. Material parameters of soils are provided on Table 6.18.

Table 6.18 Material parameters for soil's used during analysis

Soil Type	γ (kN/m^3)	E (kPa)	ν	φ (°)	c (kPa)	ψ (°)
Sand	17.4	64020	0.3	47.4	1	17.4
Clay	18.5	85670	0.3	32.4	37.7	2.4

Plate type elements are used in this study in order to model loading plate and facing elements of reinforced earth wall during parametric study. Material properties of these two plates are given on Table 6.19 below.

Table 6.19 Material Properties of Plate Elements

Plate Element	EA (kN/m)	EI (kNm^2/m)
Loading Plate	1000000000	100000000
Rigid Facing	5760000	15600

Since Plaxis is a finite element code, model should be discretized. This process is also called as meshing. Plaxis offers triangle elements for meshing. However, one type of this triangle element has 6 nodes and 3 stress points, while the other one has 15 nodes and 12 stress points. Using higher amounts of nodes and stress points yields more accurate results. Therefore, triangle element with 15 nodes and 12 stress points are used in this study. Meshed structure is shown in Figure 6.26 below.

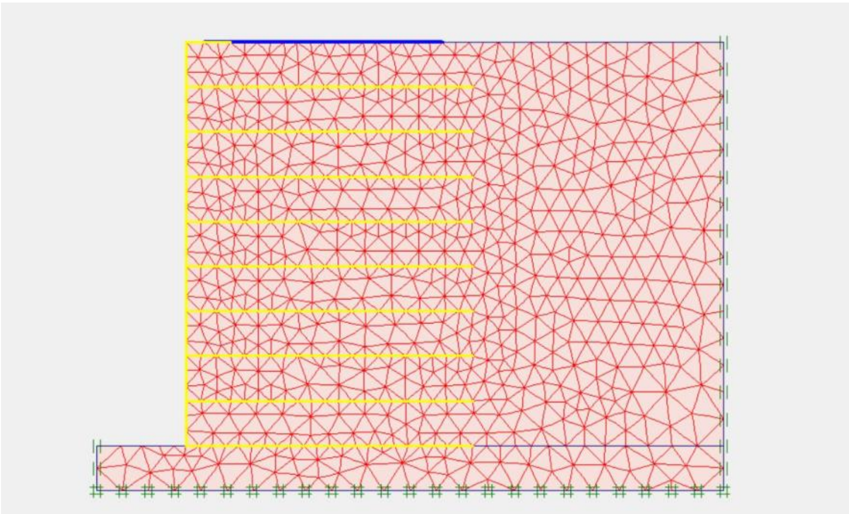


Figure 6.26 Meshed Structured of the wall

1519 elements, 12371 nodes and 18228 stress points are created in model after meshing stage. After meshing the model, calculation process is initiated. Two types of calculation are applied during this part of the study. One type is called as Plastic analysis, which calculates deformation and stresses on the structure. The second type is called as phi-c reduction analysis. Slope stability analysis is conducted and safety against slope failure is evaluated.

6.2.1 Results of Finite Element Analysis

Results of finite element analysis will be presented in this section. The results will be presented in two sub-sections. The finite element results will be compared with test results in first sub-section. The results of parametric study will be presented in the second sub-section.

6.2.1.1 Comparison of Finite Element Results with Test Results

Settlement of loading plate for sand backfill is compared with results of finite element model. It seen that, finite element analysis result is slightly higher than test result under 0.06 MPa. As applied load increases, test results become higher then finite element analysis. When applied load becomes 0.497 MPa, computed settlement increases to 3.6 mm. The settlement values are provided on Table 6.20 from test and finite element analysis.

Table 6.20 Settlement values of loading plate from test and Plaxis analysis

Load (MPa)	Test Settlement (mm)	Plaxis Settlement (mm)
0	0	0
0.055	0.41	0.52
0.12	1.09	0.96
0.178	1.6	1.37
0.236	2.08	1.77
0.299	2.65	2.21
0.357	3.18	2.63
0.425	3.72	3.1
0.476	4.19	3.46
0.497	4.44	3.6

When increment of settlement is investigated, it is seen that settlement increases linearly with respect to applied load. However, slope of the results obtained from laboratory tests are higher than results computed by finite element method. This behaviour may be seen on Figure 6.27.

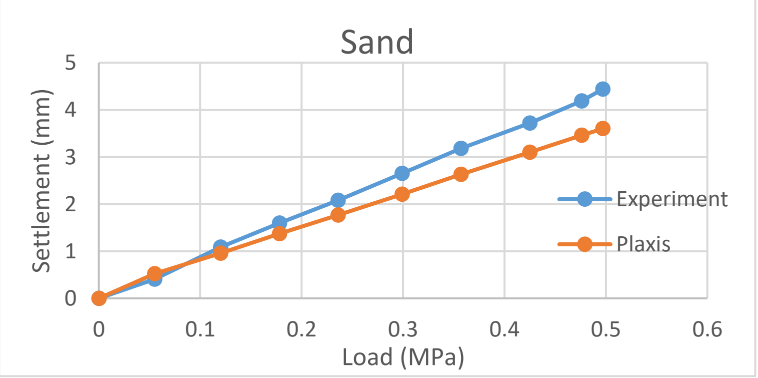


Figure 6.27 Change of settlement of loading plate with respect to load

When test with clay backfill is modelled, finite element software underestimated the settlement of the plate. The settlement values obtained from test and finite element model is given on Table 6.21.

Table 6.21 Settlement values of loading plate from test and finite element method

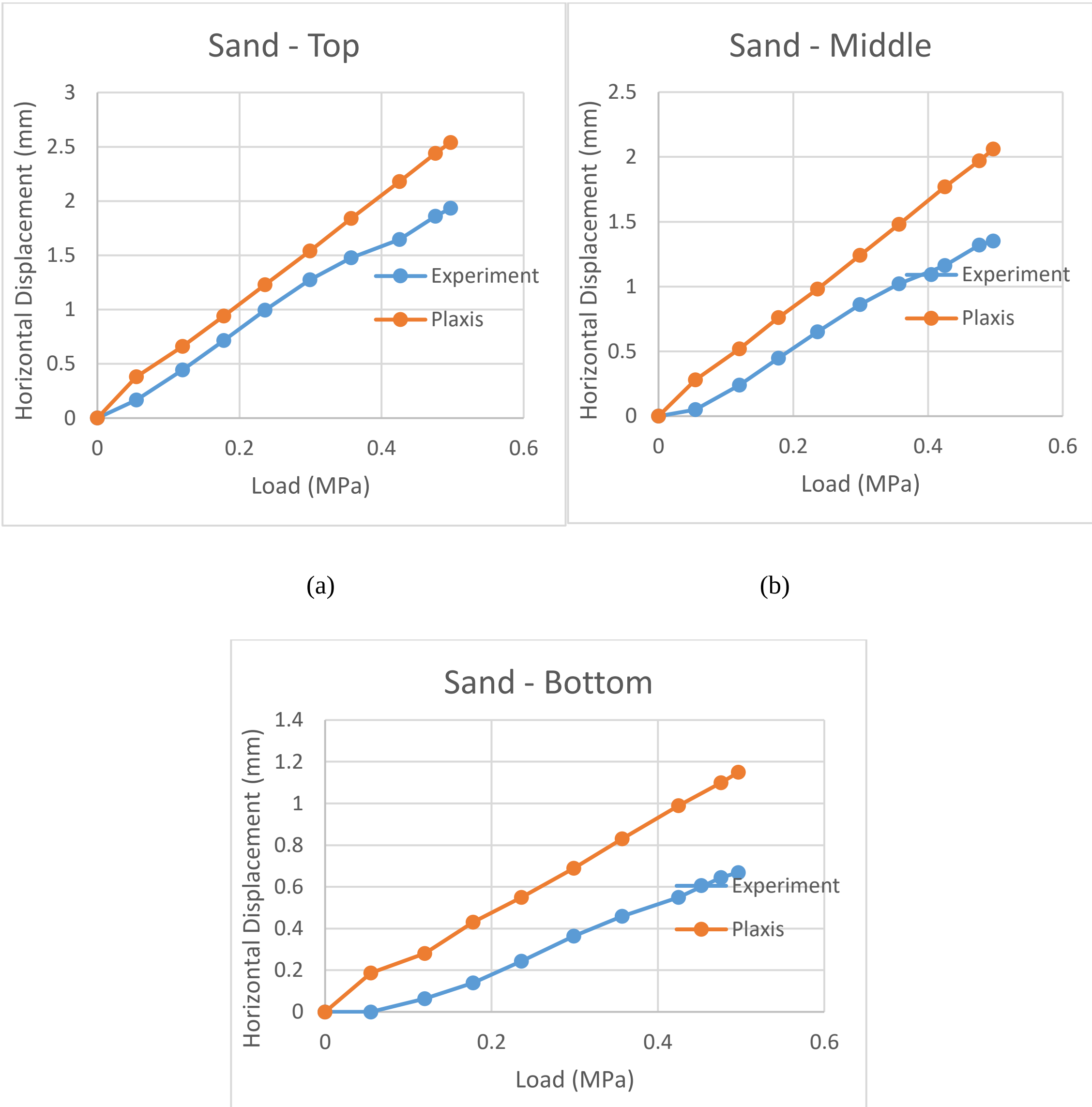
Load (MPa)	Test Settlement (mm)	Plaxis Settlement (mm)
0	0	0
0.06	0.97	0.36
0.12	1.28	0.63
0.188	1.62	0.99

When horizontal displacements of sand backfill is compared with horizontal displacements computed are comply with each other at top, middle and bottom of the wall. However, unlike the loading plate settlement, horizontal displacements computed by finite element model are higher than horizontal displacements measured at laboratory. Horizontal displacement computed as 2.54 mm under 0.497 MPa, while it is measure as 1.933 mm at top measurement point. Computed horizontal displacement decreased to 2.06 mm at the middle layer, while it is measured as 1.35 mm during test under same load. The change of horizontal displacements with respect to different loads are given on Figure 6.28 and values are provided on Table 6.22.

Table 6.22 Measured and Computed Horizontal Displacements for sand backfill

Horizontal Displacements (mm)						
Load (MPa)	Top		Middle		Bottom	
	Test	Plaxis	Test	Plaxis	Test	Plaxis
0	0	0	0	0	0	0
0.055	0.167	0.38	0.05	0.28	0	0.19
0.120	0.444	0.66	0.24	0.52	0.064	0.28
0.178	0.715	0.94	0.45	0.76	0.139	0.43
0.236	0.995	1.23	0.65	0.98	0.244	0.55
0.299	1.275	1.54	0.86	1.24	0.364	0.69
0.357	1.476	1.84	1.02	1.48	0.459	0.83
0.425	1.645	2.18	1.16	1.77	0.549	0.99
0.476	1.86	2.44	1.32	1.97	0.644	1.10
0.497	1.933	2.54	1.35	2.06	0.669	1.15

When settlement of loading plate is investigated, it is seen that, settlement measured at laboratory is higher than the computed settlement values. However, difference decreases from 62.6% to 38.9% as the load increases from 0.06 MPa to 0.188 MPa. Calculated and measure settlement of loading plate is given on Table 6.23 below.



(a) (b)

(c)

Figure 6.28 Comparison of horizontal displacements obtained from test and finite element model for sand backfill
(a) Top (b) Middle (c) Bottom

Table 6.23 Measured and Computed Settlements of Loading Plate for clay backfill

Load (MPa)	Test Settlement (mm)	Plaxis Settlement (mm)	Difference (%)
0	0	0	0
0.06	0.97	0.363	62.58
0.12	1.28	0.63	50.78
0.188	1.62	0.99	38.89

When measured and calculated horizontal displacements are compared, horizontal displacements are almost equal to each other under 0.06 MPa. However, as load is increased, difference between measured and computed horizontal displacements increases. The difference is as low as 78% at top, however it increases to 91% for middle and bottom of the reinforced earth wall under 0.188 MPa. Measurement and computed horizontal displacements are provided on Table 6.24.

Table 6.24 Measured and Calculated horizontal displacements for clay backfill

Horizontal Displacements (mm)									
	Top			Middle			Bottom		
Load (MPa)	Test	Plaxis	Difference (%)	Test	Plaxis	Difference (%)	Test	Plaxis	Difference (%)
0	0	0	0	0	0	0	0	0	0
0.06	0.565	0.27	53.10	0.29	0.15	49.31	0.155	0.08	48.39
0.12	1.775	0.43	75.77	2.02	0.25	87.62	1.015	0.13	86.90
0.19	3.345	0.72	78.48	4.93	0.42	91.48	2.635	0.23	91.27

The difference is also dependent from applied load at the same measurement level. When top layer is taken into consideration, the difference increases from 53.10% to 78.48% when load is increased from 0.06 to 0.188 MPa. Similar behaviour is valid for middle and bottom measurement points. Those values may be seen on Table 6.24.

The difference between laboratory measurement and finite element results is too much in case of clay backfill. This can be due to very low drainage condition of clay backfill which results excess pore water pressure during loading. However, it was not possible to model this behaviour in software's version used this study.

6.2.2 Parametric Study

Parametric study covers different geotextile stiffness, using rigid facing during construction and length. Computed results are compared with computed base results in order to clarify the effects of properties.

6.2.2.1 Effect of Reinforcement Stiffness

Reinforcement stiffness is increased to 2096 kN/m, 4000 kN/m, 8000 kN/m and 10000 kN/m in order to determine effect of geotextile stiffness. When stiffness is increased from 1048 kN/m to 10000 kN/m, settlement of loading plate decreases from 3.6 mm to 3.16 mm for sand backfill. If clay backfill is considered loading plate settlement decreases from 0.99 mm to 0.87 mm. Change of settlement with respect to geotextile stiffness can be seen on Figure 6.30. Settlement of loading plate decreases with increasing stiffness. However, rate of decrease is highly dependent on stiffness intervals. This means that, amount of change decreases at higher stiffnesses. It is also clear from Figure 6.29 that, decrease on computed settlement is more pronounced in case of sand backfill.

Horizontal displacements are also affected from geotextile stiffness. Decrement of horizontal displacements are dependent on stiffness interval which means, that decrement of horizontal displacement is lower at higher stiffness levels for both sand and clay backfill. Decrease of horizontal displacement is also dependent on the place of the geotextile. Horizontal displacement decreases more at the top than middle and at the middle than bottom layer. Although this behaviour is valid for both backfills, it is more visible in case of clay backfill.

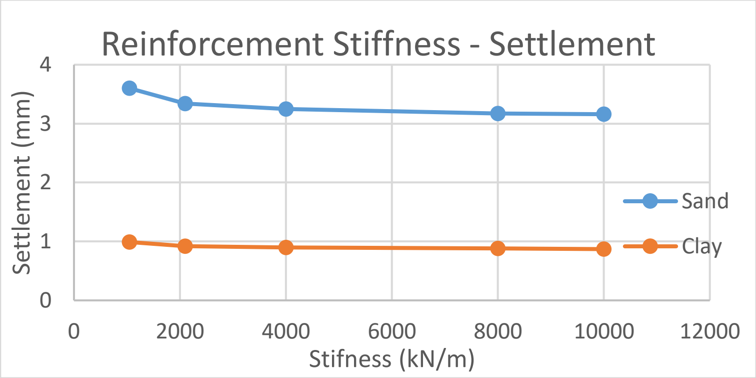


Figure 6.29 Change of plate of settlement with respect to geotextile stiffness

The change of horizontal displacements is provided on Figure 6.30 and Figure 6.31 for sand and clay backfills respectively.

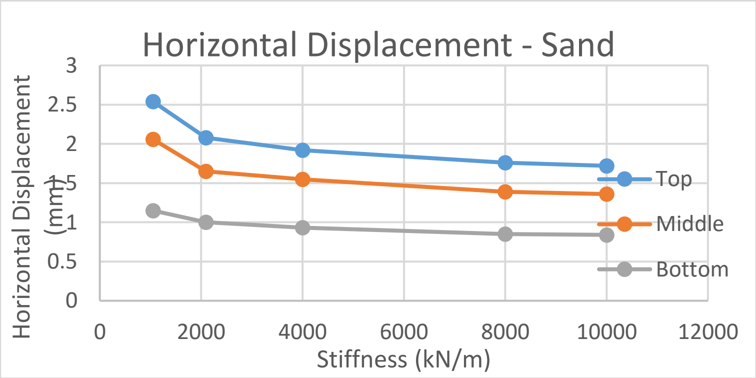


Figure 6.30 Change of horizontal displacements for sand backfill

Computed geotextile maximum forces are also effected from reinforcement stiffness. It is seen that, as the reinforcement stiffness increases, calculated reinforcement forces also increases. Similarly total horizontal force also increases as the reinforcement stiffness increases. In order to make a comparison with forces calculated according to FHWA method, Table 6.25 is given. Horizontal forces computed for the lowest reinforcement stiffness is closer to calculated forces with FHWA method. When FHWA method is considered, the highest maximum horizontal force is computed at the top layer as 6.21 kN/m. As reinforcement is buried deeper maximum horizontal forces decreases. The maximum horizontal force at deepest layer decrease to 1.28 kN/m. However, this behaviour can be seen partially on finite element models. In case of finite element model, as the reinforcement stiffness increases, maximum horizontal forces starts to decrease earlier. Since, maximum horizontal force increases with increasing stiffness on Plaxis, the difference with FHWA method is also decreases at the top while it increases considerably at the bottom.

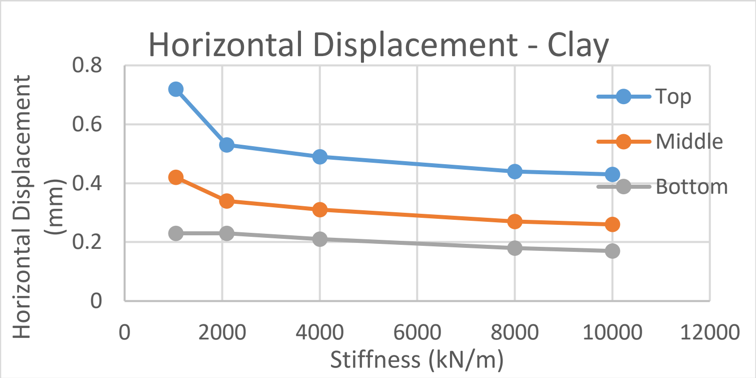


Figure 6.31 Change of horizontal displacements for clay backfill

If maximum horizontal forces under different stiffnesses are investigated with clay backfill, although negative values are computed with FHWA method, finite element results always yielded positive forces. However, those forces are smalled when compared with sand backfill. All calculated values for clay backfill can be seen on Table 6.26.

Table 6.25 Computed reinforcement forces with different stiffness for sand backfill

Sand						
Stiffness	1048 kN/m	2096 kN/m	4000 kN/m	8000 kN/m	10000 kN/m	FHWA
1st Layer (kN/m)	4.58	4.91	5.89	7.23	7.65	6.21
2nd Layer (kN/m)	4.86	5.08	6.69	8.77	9.39	3.76
3rd Layer (kN/m)	4.70	5.22	6.72	8.70	9.26	3.51
4th Layer (kN/m)	4.66	5.37	6.57	8.26	8.75	3.38
5th Layer (kN/m)	4.96	5.60	6.80	8.30	8.77	3.18
6th Layer (kN/m)	5.10	5.72	6.94	8.41	8.88	2.93
7th Layer (kN/m)	5.19	5.83	7.03	8.39	8.84	2.79
8th Layer (kN/m)	5.08	5.79	6.94	8.09	8.48	2.66
9th Layer (kN/m)	3.28	3.91	4.96	6.20	6.62	1.28
Total Horizontal Force (kN/m)	42.41	47.43	58.55	72.34	76.64	29.69

Table 6.26 Computed reinforcement forces with different stiffness for clay backfill

Clay						
	1048 kN/m	2096 kN/m	4000 kN/m	8000 kN/m	10000 kN/m	Theoretical
1st Layer (kN/m)	1.75	1.82	2.32	2.85	3.02	0.49
2nd Layer (kN/m)	1.36	1.64	2.21	2.93	3.17	0.05
3rd Layer (kN/m)	1.24	1.66	2.18	2.83	3.05	-0.13
4th Layer (kN/m)	1.32	1.75	2.27	2.78	2.97	-0.29
5th Layer (kN/m)	1.40	1.78	2.28	2.78	2.93	-0.42
6th Layer (kN/m)	1.49	1.83	2.34	2.84	2.99	-0.54
7th Layer (kN/m)	1.56	1.88	2.40	2.89	3.04	-0.65
8th Layer (kN/m)	1.51	1.91	2.38	2.84	2.97	-0.74
9th Layer (kN/m)	0.73	1.07	1.49	1.98	2.14	-0.40
Total Horizontal Force (kN/m)	12.36	15.36	19.87	24.73	26.28	-2.63

6.2.2.2 Effect of Reinforcement Density

Reinforcement density can be changed by increasing and decreasing distance between consecutive layers. Two additional reinforcement density is considered in this study. When the distance between two consecutive layers are the lowest, it will be named as the highest density. When the distance between two consecutive layers are the highest, it will be named as the lowest density. Therefore, results will be compared for three different reinforcement density including density used during tests. Reinforcement densities will be defined as distances in this study such as 3 cm, 5cm and 9 cm. When reinforcement density increases, settlement of loading plate decreases. When the distance is selected as 3 cm between two consecutive layers, settlement decreases as the distance increases. When distance between reinforcements is 3 cm, settlement of loading plate is found as 3.1 mm. The calculated settlements of loading plate are given on Table 6.27 below for all cases considered.

Table 6.27 Settlement of Loading Plate with respect to reinforcement density

	High Reinforcement Density	Medium Reinforcement Density	Low Reinforcement Density
Settlement (mm)	3.1	3.6	5.34

When horizontal displacements are compared, the lowest displacements are calculated for high reinforcement density. However, decrement is not pronounced. When low reinforcement density is used during modelling, horizontal displacements increase enormously. Profile of horizontal displacements are given on Figure 6.32 below. Calculated lowest horizontal displacement is given as 0.61 mm for high density reinforcement at bottom of wall. It increased to 0.67 mm and 3.17 mm as the reinforcement density decreases.

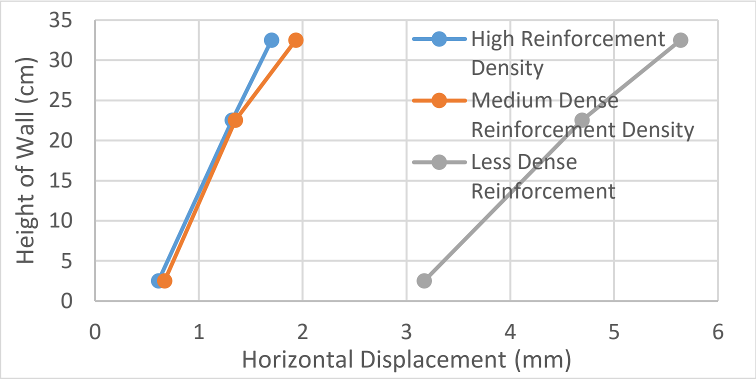


Figure 6.32 Horizontal displacement with respect to reinforcement density

Calculated horizontal displacements are provided on Table 6.28 given below.

Table 6.28 Calculated horizontal displacements with respect to reinforcement density

H (cm)	Dense (mm)	Medium (mm)	Light (mm)
32.5	1.7	1.93	5.64
22.5	1.32	1.35	4.69
2.5	0.61	0.67	3.17

When maximum horizontal forces are considered with respect to reinforcement density, it is seen that, maximum horizontal force on reinforcement increases as reinforcement density decreases. However, when total horizontal forces are compared, same behaviour is not observed. While the highest total maximum force is computed for dense reinforcement case, it is followed by light reinforcement and medium dense reinforcement. Computed horizontal forces for each layer of reinforcement and total horizontal forces are given on Table 6.29.

6.2.2.3 Effect of Reinforcement Length

Length of reinforcement has also some effect to settlement of loading plate, horizontal displacements and maximum force on reinforcement. Reinforcement length is increased to 0.8H and 0.9H from 0.7H in order determine its effect to behaviour of reinforced earth wall. Settlement of loading plate slightly decreases with increase of length of reinforcement. When reinforcement length equals to 0.7H, settlement of loading plate is calculated as 3.6 mm and decreased 3.54 mm when reinforcement length increases to 0.9H.

Horizontal displacements tend to increase as reinforcement length increases. Only exception is seen at the bottom part of the reinforced earth walls. Displacement slightly increases as reinforcement length increases to 0.8H from 0.7H at bottom. Further increase cause decrease of horizontal displacement at the bottom. The change of horizontal displacement with respect to length of reinforcement is given on Figure 6.33 below.

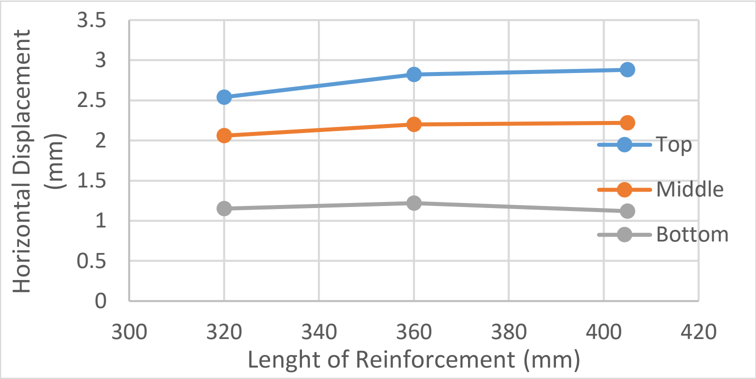


Figure 6.33 Change of horizontal displacements with respect to length of reinforcement

Maximum forces at all reinforcement levels increase as reinforcement length increases. Maximum horizontal force increases to 4.91 kN/m from 4.58 kN/m at the first layer when length increases from 0.7H to 0.9H. Therefore, calculated total horizontal forces slightly increases. The calculated horizontal forces for each reinforcement layer and length are given on Table 6.30 below.

Table 6.29 Calculated horizontal reinforcement maximum loads

	Medium (kN/m)	Low (kN/m)	High (kN/m)
1st Layer	4.58	9.47	3.13
2nd Layer	4.86	11.17	3.03
3rd Layer	4.70	11.81	2.99
4th Layer	4.66	11.09	3.05
5th Layer	4.96	4.28	3.12
6th Layer	5.10	-	3.20
7th Layer	5.19	-	3.26
8th Layer	5.08	-	3.28
9th Layer	3.28	-	3.44
10th Layer	-	-	3.37
11th Layer	-	-	3.48
12th Layer	-	-	3.52
13th Layer	-	-	3.47
14th Layer	-	-	3.46
15th Layer	-	-	2.53
Total Force	42.41	47.83	48.33

Table 6.30 Calculated horizontal forces with respect length of reinforcement

	0.7H (kN/m)	0.8H (kN/m)	0.9H (kN/m)
1st Layer	4.58	4.79	4.91
2nd Layer	4.86	5.08	5.16
3rd Layer	4.70	4.94	5.02
4th Layer	4.66	4.94	5.03
5th Layer	4.96	5.26	5.36
6th Layer	5.10	5.38	5.44
7th Layer	5.19	5.45	5.44
8th Layer	5.08	5.27	5.43
9th Layer	3.28	3.28	3.30
Total Horizontal Force	42.41	44.40	45.10

6.2.3 Factor of Safety Against Slope Stability

Factor of safety against slope stability is evaluated in this sub-section for sand and clay backfills from finite element analysis. Factor of safeties are compared with respect to magnitude of applied load, length of reinforcement, reinforcement stiffness and reinforcement density.

6.2.3.1 Factor of Safety Against Slope Stability with Respect to Load Level

Factor of safety for slope stability is evaluated for different load levels for sand and clay backfills. If sand backfill is considered, factor of safety decreases from 1.379 to 1.212 when applied load is increased from 0.06 Mpa to 0.5 Mpa. If factor of safety of reinforced earth wall with clay backfill is considered, factor of safety decreases from 1.808 to 1.321 as load increases from 0.06 MPa to 0.18 MPa. Change of factor of safeties for sand and clay backfills are given on Figure 6.34 below.

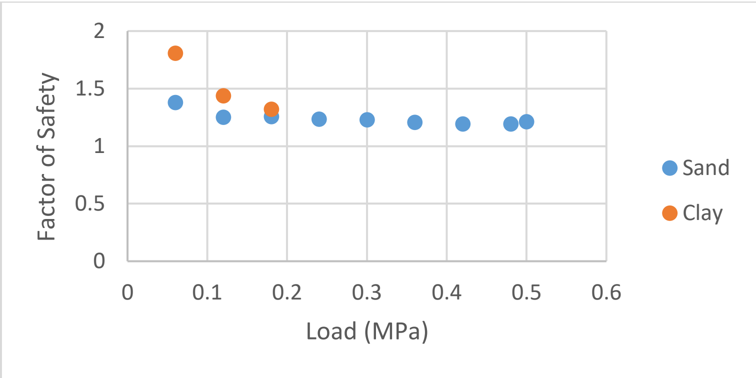


Figure 6.34 Change of Factor of Safety against Slope Stability with Respect to Load

6.2.3.2 Factor of Safety Against Slope Stability with Respect to Reinforcement Length

Factor of safety with respect reinforcement length is computed for sand backfill only and presented in this section. Finite element results show that, factor of safety against slope stability increases as the reinforcement length increases. Factor of safeties are computed as 1.212, 1.233 and 1.268 for 0.7H, 0.8H and 0.9 H reinforcement length respectively.

6.2.3.3 Factor of Safety Against Slope Stability with Respect to Reinforcement Stiffness

Reinforcement stiffness affects computed factor safety. It is seen that, factor of safety increases as reinforcement stiffness increases. Increment behavior is broken down when reinforcement stiffness equals to 4000 kN/m. However, it does not affect general behavior. Change of factor safety is given on Figure 6.35 below.

6.2.3.4 Factor of Safety Against Slope Stability with Respect to Reinforcement Density

Finite element analysis results showed that reinforcement density increases factor of safety considerably. Factor of safety increases from 1.04 to 1.618 as reinforcement density increases from low density to high density.

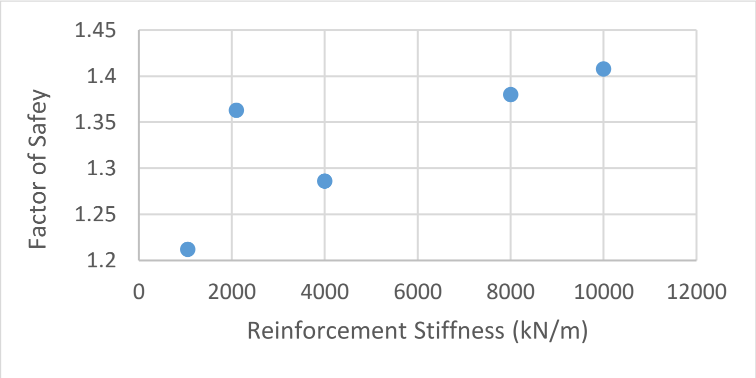


Figure 6.35 Factor of Safety Against Slope Stability with Respect to Reinforcement Stiffness

7 EFFECT OF BACKFILL AND FOUNDATION SOIL TO BEHAVIOUR OF REINFORCED EARTH WALL

There are some case studies on the literature concluding failure of foundation soils of reinforced earth walls. However, effect of foundation soil is not entirely covered. Therefore, in this study, 13 different foundation soil conditions are considered to reveal effect of foundation soil. Two layered sub – soil conditions are considered. Thickness of foundation layers varied as well as their strength parameters and material properties. Behaviour of reinforced earth wall is evaluated for backfills containing sand, 80% sand, 60% sand, 40% sand, 20% sand and clay. Height of the wall is selected as 6 meters, length of reinforcement is selected as 6 meters, stiffness of reinforcement equals to 1048 kN/m. Applied surcharge load is selected as 10.6 kN/m. Reinforced earth walls are designed according to Federal Highway Administration. Design of reinforced earth walls will be given on next sub – section.

7.1 Design of Reinforced Earth Wall

Design of reinforced earth walls are done in two parts. In the first part, reinforced earth walls externally designed. Internal design of reinforced earth wall designed on second part.

7.1.1 External Design of Reinforced Earth Wall

Reinforced earth wall is accepted as rigid body during this stage. Driving horizontal forces and resisting forces against them are calculated. Then, results are compared to define safety of reinforced earth walls against sliding. Overturning and resisting moments should be calculated so that, reinforced earth wall's safety against overturning can be determined. In order to calculate those forces and moments, material properties of backfill and retained soil should be known. Material parameters of soils used as backfill is provided on Table 7.1, while material properties of retained soils is given on Table 7.2 respectively. Calculated driving forces and resistive forces against sliding is given on Table 7.3. Overturning moments and resistive moments are given on Table 7.4.

Table 7.1 Material parameters of backfill soils

	Reinforced Backfill			
	φ (°)	γ (kN/m^3)	c (kPa)	E (GPa)
100% Sand	47.4	17.4	0.5	64.02
80% Sand + 20% Clay	42.4	20.7	11.6	55.72
60% Sand + 40% Clay	41.2	20.5	24	90.22
40% Sand + 60% Clay	38.8	20.6	25.8	59.48
20% Sand + 80% Clay	36.5	20	34.4	85.54
100% Clay	32.4	18.5	37.7	85.67

Table 7.2 Material properties of retained soil

Retained Backfill			
φ (°)	γ (kN/m^3)	c (kPa)	E (GPa)
20	15	1	15
30	17	20	50

Capacity – Demand ratio defines safety of the structure. FHWA method computes capacity of a structure and demanding forces. Loads are increased or decreased according to their type. Therefore, this ratio is called as capacity demand ratio.

Table 7.3 Horizontal Forces Acting on Reinforced Earth Wall

	Driving Force (kN)	Resistive Force (kN)	Capacity - Demand Ratio
Sand	86.02	361.65	4.20
80% Sand	86.02	430.24	5.00
60% Sand	86.02	426.08	4.95
40% Sand	86.02	428.16	4.98
20% Sand	86.02	415.69	4.83
Clay	86.02	384.52	4.47

Reinforced earth walls can be said as safe structure against sliding according to calculated capacity – demand ratios. The lowest capacity demand ratio is calculated for sand backfill. Overturning moment calculated to be same for all cases because, retained soil has same properties for every type of backfill. The lowest capacity demand ratio is calculated for sand backfill like sliding force analysis. Another important aspect of external design of reinforced earth wall is the highest pressure it will exert to foundation soil and its eccentricity.

Table 7.4 Moments Acting on Reinforced Earth Wall

	Overturning Moment (kNm)	Resistive Moment (kNm)	Capacity - Demand Ratio
Sand	193.85	2070.25	10.68
80% Sand	193.85	2426.65	12.52
60% Sand	193.85	2405.05	12.41
40% Sand	193.85	2415.85	12.46
20% Sand	193.85	2351.05	12.13
Clay	193.85	2189.05	11.29

The highest eccentricity and the lowest pressure are calculated for sand backfill. Eccentricity is found to be equal to each other for 80% sand, 60% sand, 40% sand and 20%. The calculated eccentricity and exerted maximum pressure are given on Table 7.5

below. It should be remembered that, exerted pressure will be carried by foundation layers. Each code defines minimum required foundation soil strength. Therefore, safety against bearing capacity will not be evaluated for each case considered in this section.

Table 7.5 Calculated Eccentricity and Exerted Pressures

	Eccentricity (m)	Limit Eccentricity (m)	Exerted Pressure (kPa)
Sand	0.26	1.5	171.06
80% Sand	0.23	1.5	197.67
60% Sand	0.23	1.5	196.06
40% Sand	0.23	1.5	196.87
20% Sand	0.23	1.5	192.03
Clay	0.25	1.5	179.93

The wall is found to be safe against sliding, overturning and eccentricity according to Table 7.3, Table 7.4 and Table 7.5.

7.1.2 Internal Design of the Reinforced Earth Wall

There are two elements of internal design of reinforced earth wall. Maximum horizontal load on reinforcement is calculated on first step. Pull – out resistance is calculated and compared with maximum horizontal load at the second step. In order to calculate maximum horizontal force on reinforcement, maximum horizontal stress should be calculated and multiplied by average height. Average height means that, height of each reinforcement that is responsible. Calculation of this average height depends on distance between reinforcement layer. Calculated average distances for 0.4 meters reinforcement layer interval is provided on Table 7.6.

Table 7.6 Calculation of average effective distances for each reinforcement

Z (m)	Z- (m)	Z+ (m)	Svt (m)
0.4	0	0.6	0.6
0.8	0.6	1	0.4
1.2	1	1.4	0.4
1.6	1.4	1.8	0.4
2	1.8	2.2	0.4
2.4	2.2	2.6	0.4
2.8	2.6	3	0.4
3.2	3	3.4	0.4
3.6	3.4	3.8	0.4
4	3.8	4.2	0.4
4.4	4.2	4.6	0.4
4.8	4.6	5	0.4
5.2	5	5.4	0.4
5.6	5.4	5.8	0.4
6	5.8	6	0.2

Since load transfer areas of each reinforcement is known, horizontal stresses acting on each reinforcement layer can be calculated. Horizontal stresses should be calculated using Z^- and Z^+ and taking mean value of two results for one reinforcement layer. Calculated maximum horizontal stresses are given on Table 7.7 for each type of backfill and reinforcement level.

Table 7.7 Calculated maximum horizontal stresses

	Sand	80% Sand	60% Sand	40% Sand	20% Sand	Clay
Z	σ_h (kN/m^2)	σ_h (kN/m^2)	σ_h (kN/m^2)	σ_h (kN/m^2)	σ_h (kN/m^2)	σ_h (kN/m^2)
0.4	3.25	-9.41	-42.16	-28.2	-41.06	-49.27
0.8	5.04	-6.69	-39.31	-25.02	-37.63	-45.50
1.2	6.47	-4.51	-37.03	-22.47	-34.88	-42.49
1.6	7.90	-2.33	-34.75	-19.92	-32.14	-39.47
2.0	9.33	-0.15	-32.47	-17.37	-29.4	-36.46
2.4	10.76	2.03	-30.19	-14.82	-26.66	-33.44
2.8	12.19	4.21	-27.91	-12.27	-23.91	-30.43
3.2	13.62	6.39	-25.63	-9.72	-21.17	-27.42
3.6	15.05	8.57	-23.35	-7.17	-18.43	-24.40
4.0	16.48	10.75	-21.07	-4.62	-15.69	-21.39
4.4	17.91	12.93	-18.80	-2.07	-12.94	-18.37
4.8	19.34	15.11	-16.52	0.47	-10.2	-15.36
5.2	20.77	17.29	-14.24	3.02	-7.46	-12.34
5.6	22.2	19.47	-11.96	5.57	-4.72	-9.33
6.0	23.27	21.10	-10.25	7.48	-2.66	-7.07

Maximum horizontal stresses should be multiplied by S_{vt} values from Table 7.6 in order to calculate maximum horizontal forces on reinforcement. Results of this process are given on Table 7.8.

Table 7.8 Calculated maximum horizontal forces for each type of backfill

Z (m)	Sand (kN/m)	80% Sand (kN/m)	60% Sand (kN/m)	40% Sand (kN/m)	20% Sand (kN/m)	Clay (kN/m)
0.4	1.95	-5.65	-25.29	-16.92	-24.63	-29.56
0.8	2.02	-2.68	-15.72	-10.01	-15.05	-18.20
1.2	2.59	-1.80	-14.81	-8.99	-13.95	-17.00
1.6	3.16	-0.93	-13.9	-7.97	-12.86	-15.79
2.0	3.73	-0.06	-12.99	-6.95	-11.76	-14.58
2.4	4.30	0.81	-12.08	-5.93	-10.66	-13.38
2.8	4.88	1.68	-11.16	-4.91	-9.57	-12.17
3.2	5.45	2.56	-10.25	-3.89	-8.47	-10.97
3.6	6.02	3.43	-9.34	-2.87	-7.37	-9.76
4.0	6.59	4.30	-8.43	-1.85	-6.27	-8.56
4.4	7.16	5.17	-7.52	-0.83	-5.18	-7.35
4.8	7.73	6.04	-6.61	0.19	-4.08	-6.14
5.2	8.31	6.91	-5.70	1.21	-2.98	-4.94
5.6	8.88	7.79	-4.78	2.23	-1.89	-3.73
6.0	4.65	4.22	-2.05	1.50	-0.53	-1.41

The highest maximum horizontal forces are calculated for sand backfill for each reinforcement level. Increment of cohesion yielded negative calculated forces depending on amount of cohesion due to tension cracks. Pull – out capacity of each reinforcement layer has to be calculated so that, internal safety of reinforcement earth wall can be assessed. Calculated pull – out capacities of each reinforcement layers are given on Table 7.9 below.

Table 7.9 Pull – out Capacity of Reinforcements for different backfills and reinforcement height

Z (m)	Sand (kN/m)	80% Sand (kN/m)	60% Sand (kN/m)	40% Sand (kN/m)	20% Sand (kN/m)	Clay (kN/m)
0.4	25.14	23.19	21.62	19.18	16.39	11.98
0.8	52.33	48.70	45.51	40.58	34.86	25.76
1.2	81.58	76.53	71.67	64.18	55.41	41.34
1.6	112.88	106.68	100.10	90.00	78.04	58.72
2.0	146.24	139.16	130.79	118.04	102.75	77.89
2.4	181.66	173.96	163.76	148.28	129.54	98.87
2.8	219.13	211.08	198.99	180.74	158.41	121.65
3.2	258.66	250.52	236.49	215.42	189.36	146.23
3.6	300.24	292.29	276.26	252.30	222.38	172.61
4.0	343.88	336.37	318.3	291.40	257.49	200.80
4.4	389.57	382.78	362.61	332.71	294.68	230.78
4.8	437.33	431.51	409.19	376.24	333.94	262.56
5.2	487.13	482.57	458.03	421.97	375.29	296.14
5.6	539.00	535.95	509.15	469.92	418.71	331.52
6.0	592.91	591.64	562.53	520.09	464.21	368.70

Reinforced earth wall with sand backfill has the highest pull – out capacity according to Table 7.8. As the amount of cohesion increased, pull – out capacity decreased respectively. The lowest pull – out capacity calculated for reinforced earth wall with clay backfill.

7.2 Finite Element Analysis of Reinforced Earth Wall

Reinforced earth wall is design is completed according to FHWA. Finite element analysis will be carried out in order to take foundation layers and properties into account. Considered foundation layer thicknesses and material properties are given on Table 7.10.

Table 7.10 Material properties of foundation soils

Foundation Soil 1					Foundation Soil 2				
φ°	γ (kN/m^3)	c (kPa)	E (GPa)	D (m)	φ°	γ (kN/m^3)	c (kPa)	E (GPa)	D (m)
20	15	1	15	2.5	20	18	35	60	5.0
20	15	1	15	2.5	20	18	35	60	5.0
20	15	1	15	2.5	20	18	35	60	5.0
20	15	1	15	2.5	20	18	35	60	5.0
20	15	1	15	2.5	20	18	35	60	5.0
20	15	1	15	2.5	20	18	35	60	5.0
20	15	1	15	2.5	20	18	35	60	5.0
20	15	1	15	2.5	20	18	35	60	5.0
20	15	1	15	2.5	20	18	35	60	5.0
20	15	1	15	2.5	20	18	35	60	5.0
20	15	1	15	2.5	20	18	35	60	5.0
20	15	1	15	2.5	20	18	35	60	5.0
20	15	1	15	5.0	20	18	35	60	5.0
20	15	1	15	5.0	20	18	35	60	5.0
20	15	1	15	5.0	20	18	35	60	5.0
20	15	1	15	5.0	20	18	35	60	5.0
20	15	1	15	5.0	20	18	35	60	5.0
20	15	1	15	5.0	20	18	35	60	5.0
20	15	1	15	5.0	20	18	35	60	2.5
20	15	1	15	5.0	20	18	35	60	2.5
20	15	1	15	5.0	20	18	35	60	2.5
20	15	1	15	5.0	20	18	35	60	2.5
20	15	1	15	5.0	20	18	35	60	2.5
20	15	1	15	5.0	20	18	35	60	2.5
35	17	20	55	2.5	20	18	35	60	5.0
35	17	20	55	2.5	20	18	35	60	5.0
35	17	20	55	2.5	20	18	35	60	5.0
35	17	20	55	2.5	20	18	35	60	5.0
35	17	20	55	2.5	20	18	35	60	5.0
35	17	20	55	2.5	20	18	35	60	5.0
35	17	20	55	5.0	20	18	35	60	5.0
35	17	20	55	5.0	20	18	35	60	5.0

35	17	20	55	5.0	20	18	35	60	5.0
35	17	20	55	5.0	20	18	35	60	5.0
35	17	20	55	5.0	20	18	35	60	5.0
35	17	20	55	5.0	20	18	35	60	5.0
35	17	20	55	5.0	20	18	35	60	2.5
35	17	20	55	5.0	20	18	35	60	2.5
35	17	20	55	5.0	20	18	35	60	2.5
35	17	20	55	5.0	20	18	35	60	2.5
35	17	20	55	5.0	20	18	35	60	2.5
35	17	20	55	5.0	20	18	35	60	2.5
35	17	20	55	5.0	20	16	1	15	2.5
35	17	20	55	5.0	20	16	1	15	2.5
35	17	20	55	5.0	20	16	1	15	2.5
35	17	20	55	5.0	20	16	1	15	2.5
35	17	20	55	5.0	20	16	1	15	2.5
35	17	20	55	5.0	20	16	1	15	2.5
35	17	20	55	5.0	20	16	1	15	5.0
35	17	20	55	5.0	20	16	1	15	5.0
35	17	20	55	5.0	20	16	1	15	5.0
35	17	20	55	5.0	20	16	1	15	5.0
35	17	20	55	5.0	20	16	1	15	5.0
35	17	20	55	5.0	20	16	1	15	5.0
20	15	1	15	2.5	20	16	1	15	2.5
20	15	1	15	2.5	20	16	1	15	2.5
20	15	1	15	2.5	20	16	1	15	2.5
20	15	1	15	2.5	20	16	1	15	2.5
20	15	1	15	2.5	20	16	1	15	2.5
20	15	1	15	2.5	20	16	1	15	2.5
20	15	1	15	2.5	20	16	1	15	5.0
20	15	1	15	2.5	20	16	1	15	5.0
20	15	1	15	2.5	20	16	1	15	5.0
20	15	1	15	2.5	20	16	1	15	5.0
20	15	1	15	2.5	20	16	1	15	5.0
20	15	1	15	2.5	20	16	1	15	5.0
20	15	1	15	5.0	20	16	1	15	2.5
20	15	1	15	5.0	20	16	1	15	2.5
20	15	1	15	5.0	20	16	1	15	2.5
20	15	1	15	5.0	20	16	1	15	2.5
20	15	1	15	5.0	20	16	1	15	2.5
20	15	1	15	5.0	20	16	1	15	2.5
20	15	1	15	5.0	20	16	1	15	5.0
20	15	1	15	5.0	20	16	1	15	5.0
20	15	1	15	5.0	20	16	1	15	5.0
20	15	1	15	5.0	20	16	1	15	5.0
20	15	1	15	5.0	20	16	1	15	5.0
20	15	1	15	5.0	20	16	1	15	5.0

Parametric study is also conducted by finite element method (FEM) using Plaxis v8.2. Models are created by using plane – strain model. One of the considered models is given on Figure 7.1 as an example. In order to eliminate effect of boundary conditions during calculations, boundary conditions are placed 50 meters away from the reinforced earth wall face.

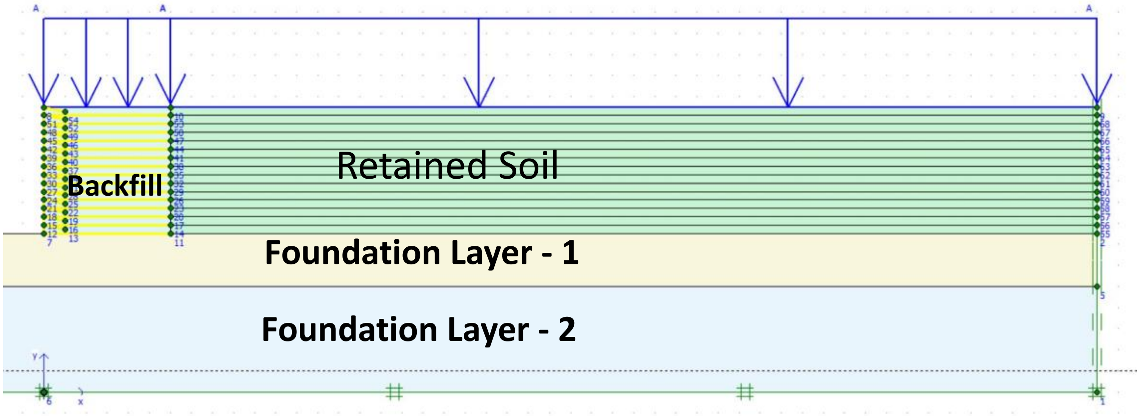


Figure 7.1 Geometry of Modelled Reinforced Earth Wall

15 node elements with 12 stress points are used during meshing. 1749 elements are created with a 0.878 meters element size in average. Those elements consist of 14311 nodes and 20988 stress points. Meshed structure of model is given on Figure 7.2.

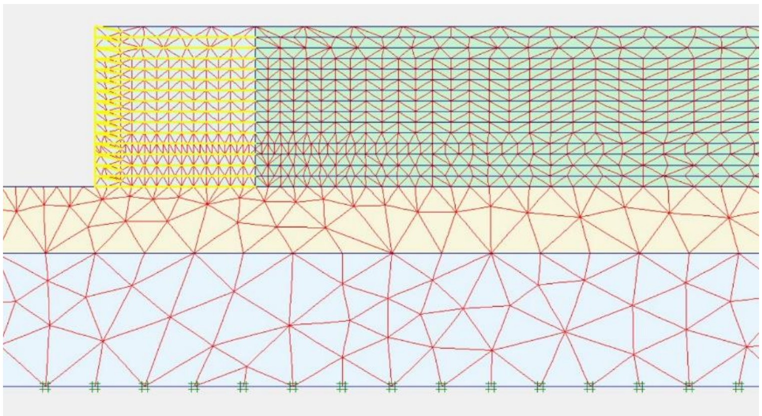


Figure 7.2 Meshed Structure of Finite Element Model

Calculation process consisted of 16 phases. Reinforced earth wall consisted of 15 reinforcement layers. Those reinforcement layers lifted one by one during calculation phase. Surcharge load is applied at the last step.

7.2.1 Finite Element Analysis Results for Different Backfill Types

Results are evaluated according to type of backfill used with considering first case of foundation layer properties. After that, results will be evaluated for each backfill soil type considering all type of foundation layers properties.

7.2.1.1 Horizontal Displacements of Wall Face for Different Backfill

When horizontal displacements of different backfills are considered, the lowest deformations are computed for sand backfill when surcharge load is not applied. When computed horizontal deformations are compared with each other, the horizontal deformations can be put in order as 80% sand, 60% sand, 40% sand, 20% sand, clay and sand from the highest to the lowest. The computed displacements increase linearly with respect to increase of wall height. The highest and the lowest displacements are computed as 144.98mm, 86.92mm respectively at the top of the wall. Change of horizontal displacements without surcharge load is given on Figure 7.3.

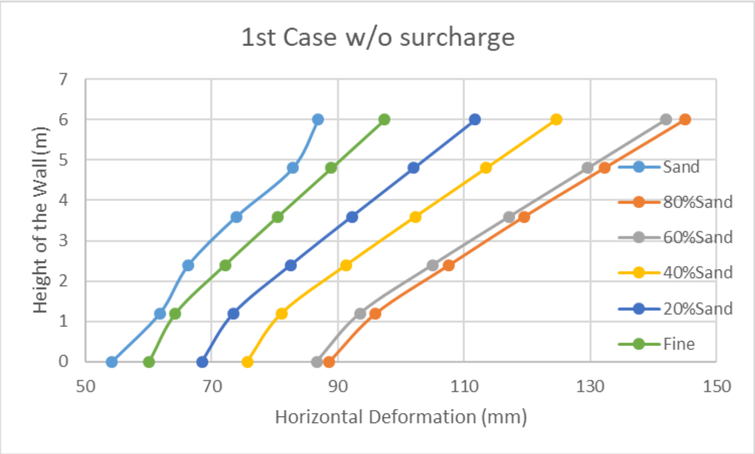


Figure 7.3 Horizontal Displacements of Reinforced Earth Wall for Different Backfills without Surcharge Load

When surcharge load is applied, displacements still increase linearly with respect to height of the wall. However, application of surcharge load changes order of the magnitude of horizontal displacements. The order becomes 80% sand, 60% sand, 40% sand, 20% sand, Sand and clay content from highest to lowest. The highest and lowest horizontal displacements are computed as 472.16 mm, and 366.62 mm respectively. Horizontal displacements under surcharge load is given on Figure 7.4 below.

7.2.1.2 Horizontal Displacement of Retained Soil for Different Backfill

When horizontal displacement of retained soil for different soil type in backfill area is investigated, it is seen that, horizontal displacement increases with height, however decrement is seen at the surface for all type of backfills. The computed horizontal deformations can be given as sand, clay, 20% sand, 40% sand, 80% sand, and 60% sand from the lowest to highest. The computed displacement at the bottom are equal to each other for 80%, 40% and 20% sand contents. Horizontal displacements of retained soil without surcharge load is given on Figure 7.5 below.

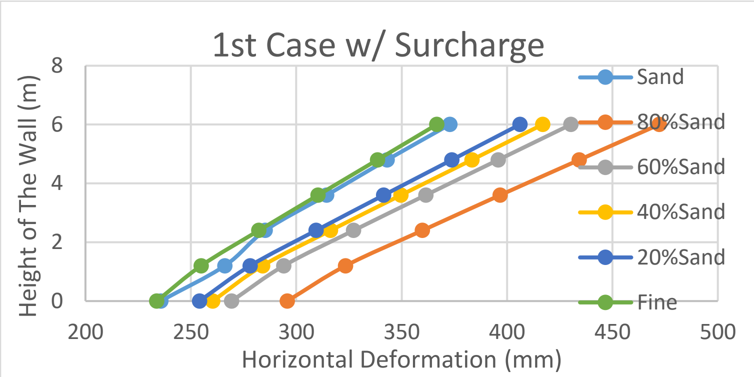


Figure 7.4 Horizontal Displacements of Reinforced Earth Wall for Different Backfills under Surcharge Load

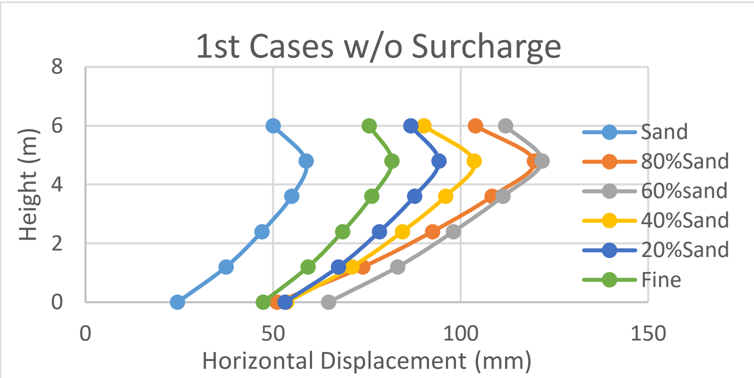


Figure 7.5 Horizontal Displacements of Retained Soil for Different Backfills without Surcharge Load

When surcharge load is applied, the highest displacement is calculated for 80% sand content and 60% sand content. The order remains the same for other contents as mentioned before. The decrease of horizontal deformation is reduced or even vanishes for clay, 20% sand and 60% sand contents at the surface when surcharge load is applied. Computed horizontal displacement of retained wall under surcharge load is given on Table 7.11.

Table 7.11 Computed Horizontal Displacements of Retained Soil under Surcharge Load

H (m)	Sand (mm)	80% Sand (mm)	60% Sand (mm)	40% Sand (mm)	20% Sand (mm)	Clay (mm)
6	286.81	415.47	395.21	372.99	375.44	339.25
4.8	320.85	424.25	390.10	375.26	367.61	332.57
3.6	294.78	387.43	357.43	344.68	337.94	307.34
2.4	263.59	345.08	321.51	309.69	305.18	279.54
1.2	228.81	297.90	282.92	271.20	270.25	249.99
0	184.76	240.23	236.50	223.11	229.27	214.90

7.2.1.3 Settlement of Wall for Different Backfill

When the settlement of wall is investigated for different sand content, it is found out that, settlement is higher at the wall face and decreases linearly as the distance increases with the wall face. The highest settlement is computed for 80% sand content and it is followed by 60%, 40%, 20% sand, clay and sand contents, however at the end of the reinforced soil zone, the settlement computed for 20% sand content is insignificantly higher than 40% sand content. It should also be noted that, computed settlements at the end of reinforced zone are almost equal to each other for different backfill type. Application of surcharge load does not change the behaviour but increases computed settlements. Change of settlements with respect to soil type is given on Figure 7.6.

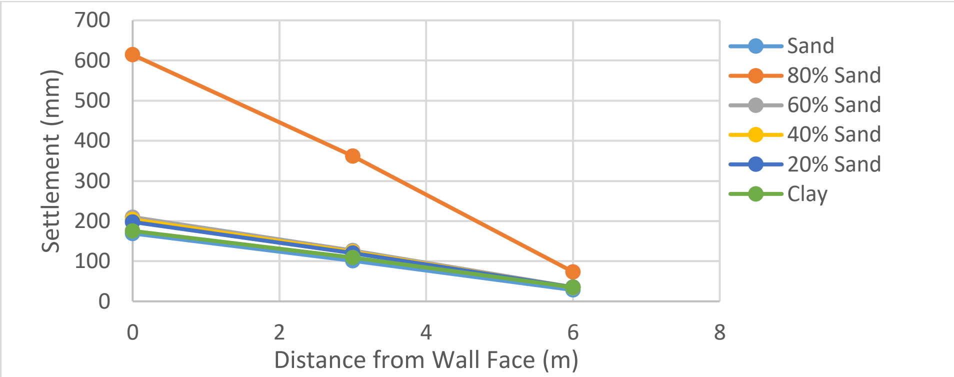


Figure 7.6 Change of Settlement of Foundation for Different Backfill Type under Surcharge Load

7.2.1.4 Maximum Forces on Geotextile for Different Backfill

When the computed maximum forces are compared for different sand content in backfill, it is seen that computed forces generally decreases as sand content decreases. However, in some circumstances, higher force is computed with lower sand content. These circumstances can be named as 40% and 20% sand contents for second layer of geosynthetic, 80% sand content at 7th layer and 40% sand content for the 15th layer geosynthetic. It should be noted here that, computed forces for 80% sand content is higher than computed forces for sand layer for the bottom four layers of geosynthetics. It is also seen that, computed maximum axial forces decrease slightly at 2nd layer geosynthetic for 80% sand, 60% sand and clay content, at third layer geosynthetic for Sand, 40% sand and 20% sand contents. The change of maximum horizontal force with respect to wall height is shown on Figure 7.7.

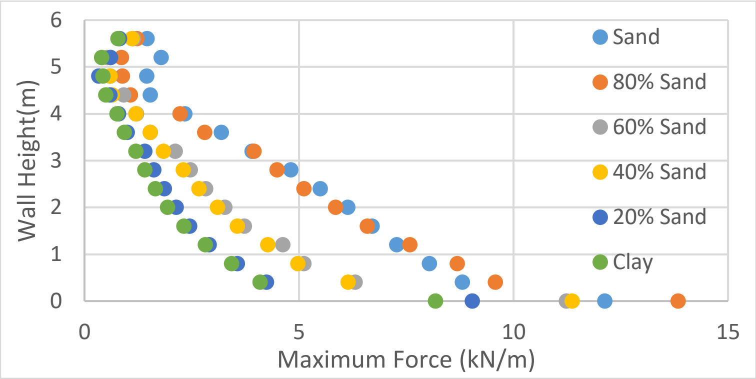


Figure 7.7 Change of maximum horizontal force with respect to height of wall

When the maximum forces of each geosynthetic layer is summed up, the highest value is obtained as 75.04 kN/m for sand content. This value decreases as the sand content decreases. When only clay backfill is considered, the summed horizontal load is found as 30.79 kN/m. Resultant horizontal maximum force is given on Table 7.12.

Table 7.12 Resultant Horizontal Forces for all types of Backfill

	Sand	80% Sand	60% Sand	40% Sand	20% Sand	Clay
Resultant Horizontal Force (kN/m)	75.03588	74.71645	47.57974	45.87042	33.32949	30.78983

When the surcharge load is applied, the computed maximum force increases especially at the first layer geosynthetic. After first layer, computed force decreases for the following 2 layers. After that, computed maximum force increases for following layers. The highest maximum forces are calculated for sand content except for last three layer at the bottom. The highest forces are computed for 80% sand content at last three layers at the bottom. Other than that, higher maximum forces are observed for 40% sand content except for 4th and 7th layer than forces computed for 60% sand content. The decrease of maximum force is continued for 4th and 7th layers. The highest and lowest total resultant force is computed as 98.63 kN/m and 41 kN/m for sand and clay backfill. Higher total force is computed for 40% sand content than 60% sand content, however difference is only about 2 kN/m. The highest increase due to surcharge load is computed as 23.60 kN/m for sand and the lowest increase due to surcharge load is computed as 10.21 kN/m for clay content. Computed maximum horizontal loads and resultant horizontal load is given on Table 7.13 below for all type of backfills.

Table 7.13 Computed reinforcement forces for all backfill types

Depth (m)	Sand (kN/m)	80% Sand (kN/m)	60% Sand (kN/m)	40% Sand (kN/m)	20% Sand (kN/m)	Clay (kN/m)
5.6	4.79	2.86	2.85	3.34	2.57	2.42
5.2	2.64	1.05	0.67	0.74	0.74	0.51
4.8	2.39	0.94	0.62	0.63	0.50	0.45
4.4	2.62	1.13	0.94	0.79	0.88	0.54
4	2.98	2.26	1.25	1.38	1.14	0.81
3.6	3.61	2.81	1.59	1.77	1.41	0.98
3.2	4.55	4.01	2.18	2.14	1.87	1.25
2.8	5.70	4.73	2.56	2.67	2.15	1.45
2.4	6.50	5.46	2.97	3.12	2.45	1.70
2	7.16	6.38	3.46	3.62	2.79	1.98
1.6	7.78	7.41	3.95	4.13	3.19	2.34
1.2	8.51	8.48	4.72	4.85	3.63	2.85
0.8	9.67	9.82	5.48	5.67	4.27	3.52
0.4	11.11	11.21	7.02	7.37	5.54	4.25
0	18.62	23.17	18.80	19.00	17.32	15.95
Resultant	98.63	91.72	59.07	61.22	50.46	41.00

7.2.2 Finite Element Analysis Results for Different Foundation Conditions

Behaviour of reinforced earth wall with respect to different foundation soil will be investigated in this section. Horizontal displacements, settlements and maximum forces on reinforcements will be compared in this section according to backfill type.

7.2.2.1 Effect of Foundation Soil Properties to Horizontal Displacements of Reinforced Earth Wall Face

Horizontal displacements with respect to foundation properties are evaluated in this section. Different backfill soils will be discussed in different sub-sections.

7.2.2.1.1 Horizontal Displacements of Reinforced Earth Wall with Sand Backfill

Horizontal displacements measured for sand backfill increases for 3rd, 4th, 10th, 11th, 12th and 13th cases when compared with 1st case. The highest and lowest horizontal displacements are computed for 13th case and 9th case respectively. When thickness of the first foundation layer increased to 5 meters, displacements increase from 86.92 mm to 96.7 mm, however, if the thickness of the second foundation layer decreases, computed horizontal displacement decreases to 92.1 mm at the top of the wall. This value corresponds to the displacement computed for 4th case. When the soil strength parameters and layer thickness is decreased, which corresponds to 10th case, computed horizontal displacements increase to 97.40 mm at the top. If only the soil strength parameters of 2nd foundation layer changed, calculated horizontal displacements increase to 106.35 mm at the top of wall. Computed horizontal deformations without surcharge load is given on Table 7.14.

Table 7.14 Horizontal Displacement for Sand Backfill under Different Foundation Layers Conditions

H (m)	1st Case (mm)	2nd Case (mm)	3rd Case (mm)	4st Case (mm)	5th Case (mm)	6th Case (mm)	7th Case (mm)	8th Case (mm)	9th Case (mm)	10th Case (mm)	11th Case (mm)	12th Case (mm)	13th Case (mm)
6.0	86.92	33.92	96.73	92.10	8.61	7.60	8.52	6.35	2.89	97.40	106.35	97.25	111.65
4.8	82.86	32.79	90.46	85.86	8.80	8.01	8.72	7.04	4.26	90.64	98.35	90.34	103.60
3.6	73.94	30.73	83.59	78.89	8.41	7.87	8.36	7.18	5.13	83.16	89.63	82.73	94.98
2.4	66.34	27.56	75.64	70.93	7.33	7.05	7.30	6.70	5.36	74.64	79.92	74.16	85.37
1.2	61.81	23.28	66.36	62.22	5.47	5.56	5.47	5.47	4.50	65.19	69.32	64.85	74.73
0	54.23	22.42	61.78	60.02	2.38	2.49	2.41	2.84	2.83	60.34	63.59	62.01	68.19

When the strength properties of first foundation layer increase, calculated horizontal displacement decreases to 8.61 mm at the top of the wall. If the layer thickness of the first foundation layer increases to 5 meters, displacement decreases to 7.60 mm. If the layer thickness of the second foundation layer decreases at the same conditions with upper case, horizontal displacement at the top of the wall increases to 8.52 mm. When the thickness and soil properties of first foundation layer increase and soil properties and thickness of second foundation layer decrease yielded lower horizontal displacements such as 6.35 mm at the top of the wall. If the thickness and soil properties of first foundation layer increase and soil properties of second layer foundation layer decrease, computed displacements decrease to 2.89 mm at the top of the wall. Those given values are taken from 5th, 6th, 7th, 8th and 9th cases respectively. In these cases, horizontal displacements increase as the depth increases and then displacements decrease for further increase of depth.

When the surcharge load is applied to reinforced earth wall, the highest and the lowest displacements increase to 520.98mm and 6.12 mm for 10th and 9th cases respectively. When the displacement increase is normalized, the highest increase is computed as averagely as 440% for 10th case. The lowest increase is computed averagely as 35% for 7th case. When surcharge load is applied, magnitude order of the computed displacements changes with respect to considered case. As explained above, computed displacements for 3rd, 4th, 10th, 11th, 12th and 13th cases are higher than the reference 1st case. The highest difference is formed between 1st and 12th cases. When surcharge load is applied, computed horizontal displacements at the top of the wall increases to 372.80 mm from 86.92 mm and to 181.26 mm from 97.25 mm for 1st and 12th cases respectively. Change of horizontal displacements with respect to height is given on Figure 7.8.

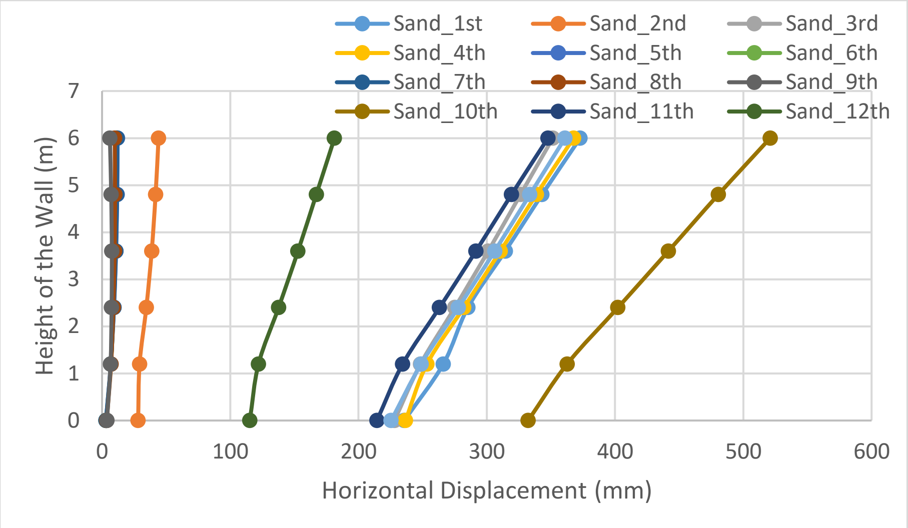


Figure 7.8 Horizontal Displacement of Reinforced Earth Wall with Sand Backfill

7.2.2.1.2 Horizontal Displacements of Reinforced Earth Wall with 80% Sand Backfill

When the horizontal displacements are computed for 80% sand content of backfill, the highest horizontal displacement is computed for 1st case as 144.98 mm at the top of the wall. Since this case is the reference case, behaviour change is observed for 3rd, 4th, 10th, 11th, 12th and 13th case. The lowest horizontal displacement is computed for 9th case as -0.531 mm which means that top of the wall leaned through back. After that, horizontal displacement increased to 4.31 mm for 1.2 meter above the bottom and 2.70 mm at the bottom. Computed horizontal displacements without surcharge load are given on Figure 7.9 with respect to height of the wall.

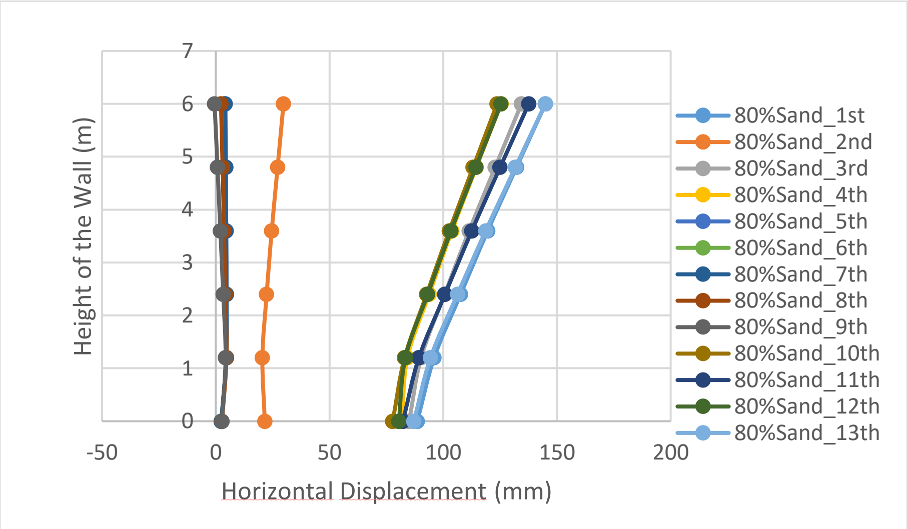


Figure 7.9 Horizontal Displacement of Reinforced Earth Wall with 80% Sand Backfill

When the surcharge load is applied, calculated displacement increases to 472.16 mm for 1st case. Although computed displacements are lower than 1st case when surcharge load is not applied, computed horizontal displacements equal to 524.92 mm for 10th case which is higher than the first case. Computed horizontal displacement almost becomes equal to first case when 3rd case is considered. In this case, horizontal displacement is computed as 471.89 mm at the top of the wall. The highest change is calculated for 10th case which is around 335%. The lowest change is calculated for 5th case as averagely 45%. Calculated negative displacement disappeared when surcharge load is applied to 9th case. Computed horizontal displacements under surcharge load for 80% backfill is given on Table 7.15.

Table 7.15 Computed Horizontal Displacement of Wall Face with 80% Sand Backfill

H (m)	1st Case (mm)	2nd Case (mm)	3rd Case (mm)	4st Case (mm)	5th Case (mm)	6th Case (mm)	7th Case (mm)	8th Case (mm)	9th Case (mm)	10th Case (mm)	11th Case (mm)	12th Case (mm)	13th Case (mm)
6.0	472.16	43.92	471.89	233.54	6.92	6.21	6.89	5.13	2.16	524.92	422.73	396.53	452.91
4.8	434.26	40.23	436.13	214.68	6.85	6.33	6.83	5.54	3.21	485.50	388.55	365.18	416.50
3.6	396.71	36.71	400.74	196.21	6.74	6.42	6.73	5.94	4.29	446.33	354.81	334.25	380.50
2.4	359.84	33.76	365.93	178.47	6.69	6.59	6.70	6.44	5.47	407.70	321.66	303.92	345.07
1.2	323.41	30.82	331.58	161.13	6.13	6.25	6.16	6.44	6.17	369.30	288.86	274.06	310.24
0	295.71	31.60	304.36	153.70	3.07	3.12	3.09	3.67	3.65	338.59	263.83	254.79	282.57

7.2.2.1.3 Horizontal Displacements of Reinforced Earth Wall with 60% Sand Backfill

When backfill consists only 60% sand, similar behaviour is observed with backfill of 80% sand content. Calculated horizontal displacements are smaller than 80% sand content. Calculated horizontal displacements are the highest at 1st case and the lowest at 9th case. Computed displacements equal to 141.96 mm and -2.58 mm respectively. It is seen that, wall face leans back more with backfill 60% sand content than wall with 80% sand backfill. The amount of decrease at the bottom on 9th case is also smaller than the decrease computed for 80% sand content for the same content. Calculated horizontal displacements without surcharge load with 60% sand backfill is given on Table 7.16.

Table 7.16 Computed Horizontal Displacement of Wall Face with 60% Sand Backfill

H (m)	1st Case (mm)	2nd Case (mm)	3rd Case (mm)	4st Case (mm)	5th Case (mm)	6th Case (mm)	7th Case (mm)	8th Case (mm)	9th Case (mm)	10th Case (mm)	11th Case (mm)	12th Case (mm)	13th Case (mm)
6.0	141.96	21.89	103.62	101.21	2.25	1.51	2.20	0.31	-2.58	112.60	117.18	104.55	121.65
4.8	129.60	19.77	94.77	92.47	2.66	2.10	2.62	1.19	-1.10	102.94	106.30	95.19	110.49
3.6	117.20	17.39	85.91	83.72	3.00	2.64	2.98	2.04	0.39	93.15	95.31	85.82	99.28
2.4	105.03	15.03	77.22	75.25	3.29	3.14	3.29	2.87	1.86	83.44	84.43	76.68	88.25
1.2	93.53	13.12	69.27	67.51	3.42	3.47	3.44	3.53	3.17	74.27	74.18	68.27	77.93
0	86.63	14.46	66.61	66.77	2.55	2.66	2.59	3.04	3.05	69.41	68.55	66.70	72.49

When surcharge load is applied, computed horizontal displacements increase more for 3rd and 10th cases then calculated displacement for 1st case. Computed displacements are 434.50 mm, 597.14 mm and 430.26 mm for 3rd, 10th and 1st cases respectively. Application of surcharge load resulted in 203% increase for the first case. The amount of increase is higher than this value for 3rd, 4th, 10th, 11th, 12th and 13th cases. Amount of increase computed as 319%, 270%, 430%, 240%, 245% and 252% respectively. Although amount of increase is higher for 4th, 11th, 12th and 13th cases, computed displacements are still lower than the 1st case. The highest displacement is computed as 597.14mm at 10th case and the lowest displacement is computed as -1.18 mm at 9th case when surcharge load is applied. Change of horizontal displacement of reinforced earth wall with 60% sand content is given on Figure 7.10.

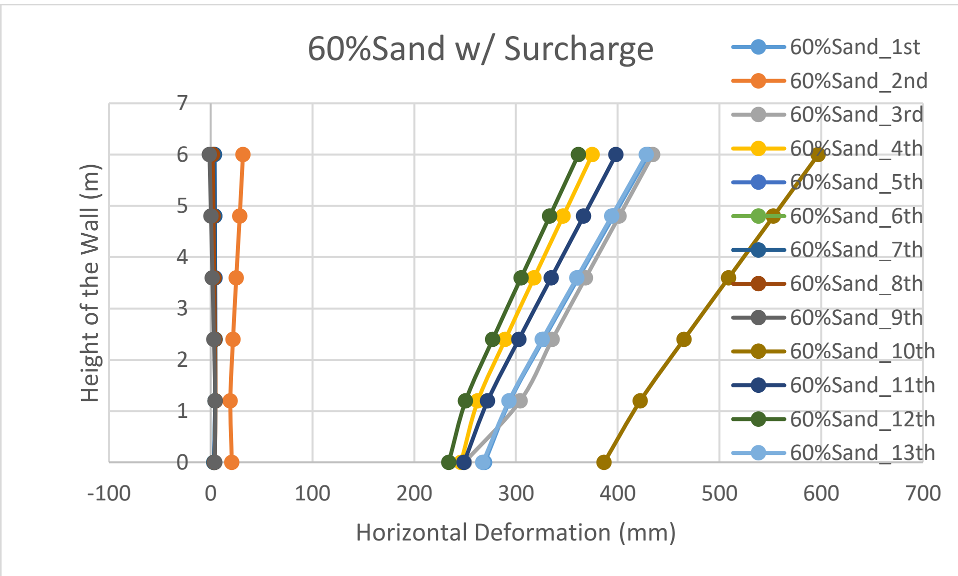


Figure 7.10 Horizontal Displacement of Reinforced Earth Wall with 60% Sand Backfill

7.2.2.1.4 Horizontal Displacements of Reinforced Earth Wall with 40% Sand Backfill

When horizontal displacements of wall constructed with only 40% sand content is investigated, the highest displacement is calculated as 125.80 mm at the top of the wall for 13th case. Computed displacement equals to 124.63 mm for reference 1st case. The lowest displacement is computed as -1.27 for 9th case on top. After that, horizontal displacement increases and decreases again. Change of horizontal displacements with respect to height of wall is given on Figure 7.11.

When surcharge load is applied, horizontal deformation at the top of the wall is calculated as 416.83 mm which is slightly higher than the horizontal displacement for 3rd case which is 416.32 mm. The highest displacement is computed for 10th case which is equal to 452.28 mm. The lowest displacement is computed for 9th case under surcharge load which equals to 0.446 mm. This means that, application of surcharge load removes leaning back of the wall. The highest amount change is also computed for 10th case as 301%. The amount of change calculated as 234% for 1st case. There are some cases where the amount of change is higher than 1st case, but the deformation is lower. Those cases can be given as 3rd, 4th, 11th, 12th and 13th cases. Computed horizontal displacements under surcharge load for different foundation properties are given on Table 7.17.

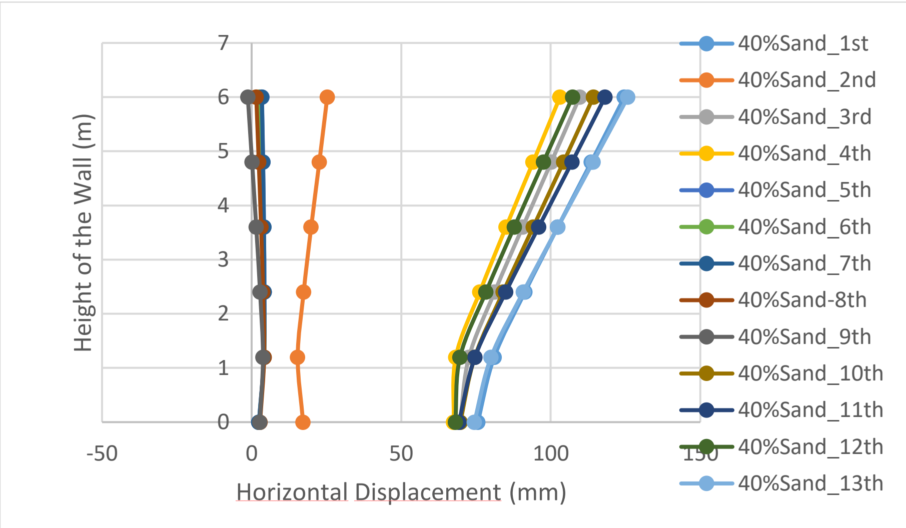


Figure 7.11 Horizontal Displacement of Reinforced Earth Wall with 40% Sand Backfill

Table 7.17 Computed Horizontal Displacement of Wall Face with 40% Sand Backfill

H (m)	1st Case (mm)	2nd Case (mm)	3rd Case (mm)	4st Case (mm)	5th Case (mm)	6th Case (mm)	7th Case (mm)	8th Case (mm)	9th Case (mm)	10th Case (mm)	11th Case (mm)	12th Case (mm)	13th Case (mm)
6.0	416.83	33.26	416.32	393.28	5.31	4.60	5.27	3.47	0.45	458.28	406.85	368.83	450.45
4.8	383.29	29.75	384.79	363.07	5.54	5.03	5.52	4.19	1.83	424.49	374.03	339.84	414.35
3.6	349.62	26.21	353.18	332.79	5.66	5.35	5.65	4.84	3.16	390.50	341.10	310.85	378.14
2.4	316.25	22.99	321.85	302.86	5.60	5.51	5.61	5.34	4.36	356.70	308.42	282.24	342.22
1.2	283.97	20.41	291.72	274.11	5.19	5.30	5.22	5.49	5.18	323.85	276.88	254.88	307.58
0	260.35	21.98	269.20	256.38	3.04	3.12	3.06	3.64	3.65	298.35	253.10	238.07	280.90

7.2.2.1.5 Horizontal Displacements of Reinforced Earth Wall with 20% Sand Backfill

When the horizontal displacements are considered for 20% sand content backfill, the highest displacement is calculated for 13th case as 113.04 mm. This value is slightly higher than the horizontal displacement calculated for 1st case which is equal to 111.79 mm. The lowest displacement is computed as -2.57 mm for 9th case such as 40% and 60% sand contents in backfill. Calculated horizontal displacement of wall face of walls with 20% sand backfill is given on Table 7.18.

Table 7.18 Computed Horizontal Displacement of Wall Face with 20% Sand Backfill

H (m)	1st Case (mm)	2nd Case (mm)	3rd Case (mm)	4st Case (mm)	5th Case (mm)	6th Case (mm)	7th Case (mm)	8th Case (mm)	9th Case (mm)	10th Case (mm)	11th Case (mm)	12th Case (mm)	13th Case (mm)
6.0	111.79	21.10	94.54	88.63	2.32	1.56	2.24	0.32	-2.57	97.09	107.17	98.04	113.04
4.8	102.04	18.98	86.50	81.08	2.73	2.17	2.67	1.22	-1.05	88.84	97.17	89.23	102.73
3.6	92.22	16.70	78.42	73.51	3.08	2.71	3.04	2.08	0.44	80.46	87.11	80.43	92.34
2.4	82.55	14.53	70.47	66.12	3.36	3.20	3.34	2.91	1.91	72.14	77.13	71.80	82.06
1.2	73.49	12.84	63.21	59.39	3.47	3.51	3.47	3.56	3.19	64.33	67.76	63.86	72.44
0	68.46	14.64	60.89	59.02	2.54	2.62	2.55	2.99	2.99	60.44	62.51	62.18	67.33

When surcharge load is applied, horizontal displacements increases for each case. The highest change is computed for 10th case averagely as 450%. The amount of change calculated for 1st case as 270% averagely. This value is smaller than the values computed for 3rd, 4th, 11th cases which are 340%, 310% and 290% respectively. Amount of computed horizontal displacements can be given as 406.05 mm, 522.09 mm, and -1.158 mm for 1st case, the highest and the lowest displacement respectively. It is important to note that, application of surcharge load does not prevent wall from leaning back. When 4th and 11th cases are considered it is seen that although the amount of change is higher in these cases, computed displacement is still lower than 1st case. Change of horizontal displacement with respect to height with 20% sand backfill regarding different foundation conditions are given on Figure 7.12.

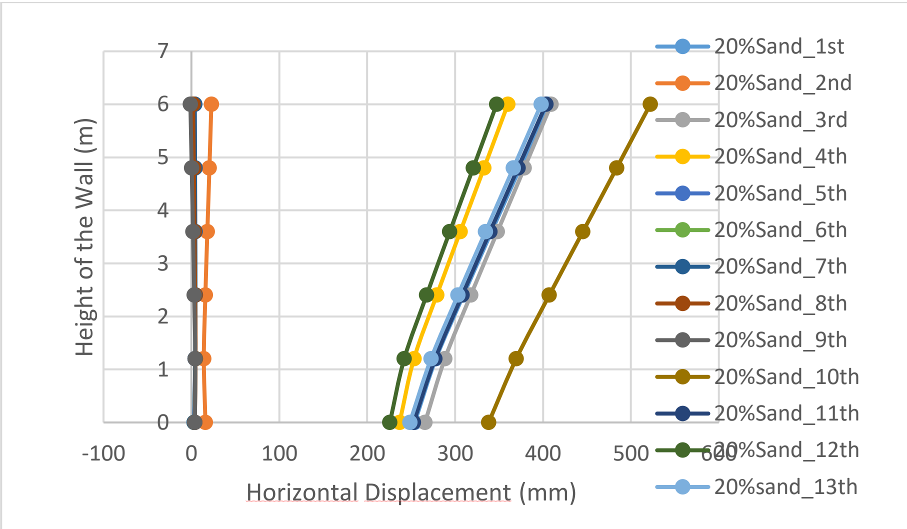


Figure 7.12 Horizontal Displacement of Reinforced Earth Wall with 20% Sand Backfill

7.2.2.1.6 Horizontal Displacements of Reinforced Earth Wall with Clay Backfill

When the horizontal displacement of wall constructed only with compacted clay is investigated, the highest displacement is computed as 97.42 mm for 1st case at the top of the wall. Almost equal horizontal displacement is calculated for 13th case as 96.35 mm. The lowest displacement is computed for 9th case as -2.86 mm. While displacement decreases as depth of the wall increases for 1st, 2nd, 3rd, 4th, 10th, 11th, 12th and 13th cases, it increases for 5th, 6th, 7th, 8th and 9th cases. However, it again decreases at the bottom for latter pronounced cases. Displacement of wall face without surcharge load is given on Figure 7.13 below.

If the surcharge load is applied, the computed horizontal displacement at the top of the wall increases 276% and becomes 366.62 mm for the first case. However, higher displacements are computed such as 383.88 mm and 407.42 mm for 4th and 10th cases respectively. The highest amount of increase is computed for 4th case 430% averagely. The lowest horizontal displacement increased to -1.46 mm after application of surcharge load at 9th case. Magnitudes of horizontal displacement of reinforced earth wall with clay backfill is provided on Table 7.19.

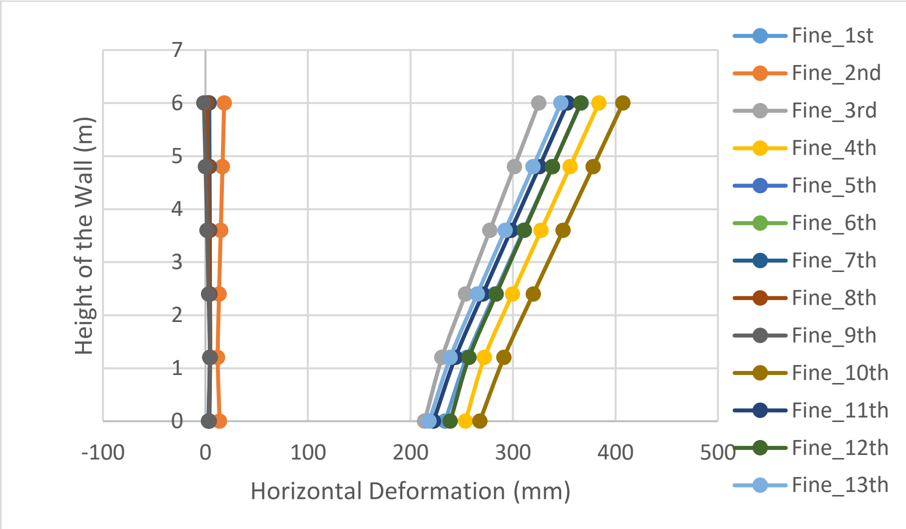


Figure 7.13 Horizontal Displacement of Reinforced Earth Wall with Clay Backfill

Table 7.19 Computed Horizontal Displacement of Wall Face with Clay Backfill

H (m)	1st Case (mm)	2nd Case (mm)	3rd Case (mm)	4st Case (mm)	5th Case (mm)	6th Case (mm)	7th Case (mm)	8th Case (mm)	9th Case (mm)	10th Case (mm)	11th Case (mm)	12th Case (mm)	13th Case (mm)
6.0	366.62	18.27	325.26	383.88	3.72	3.00	3.41	1.69	-1.46	407.42	353.32	366.12	346.87
4.8	338.49	16.67	301.40	355.72	4.04	3.52	3.80	2.55	0.10	378.27	325.76	338.62	319.66
3.6	310.29	14.93	277.51	327.51	4.27	3.96	4.10	3.36	1.63	348.98	298.10	311.09	292.34
2.4	282.26	13.29	253.79	299.51	4.40	4.32	4.30	4.10	3.09	319.78	270.58	283.74	265.24
1.2	254.86	12.01	230.81	272.25	4.32	4.45	4.29	4.61	4.32	291.18	243.73	257.13	238.86
0	233.80	13.79	213.79	253.65	3.06	3.19	3.07	3.75	3.86	267.68	222.47	238.90	218.31

7.2.2.2 Effect of Foundation Soil Properties to Horizontal Displacements of Retained Soil

Displacement of the retained soil with respect to different backfill and foundation layers are investigated in this section. Results are provided in subsections for different backfill.

7.2.2.2.1 Horizontal Displacements of Retained Soil Behind Sand Backfill

When displacement of retained soil is investigated, two different behaviour is observed. The first group may be given as 1st, 3rd, 4th, 10th, 11th, 12th, 13th cases and the second group can be given as 5th, 6th, 7th, 8th and 9th cases.

In the first group, displacement increases from surface to depth at the initial step, after that, displacement decreases as the depth of the wall increases. In the second group, negative displacement is calculated at the top and then increases. Displacement decreases at the bottom again. The highest displacement is calculated for 13th case as 70.83mm at the top and increased to 80.51mm. The displacement computed at the bottom is found as 43.67 mm. The lowest displacement is calculated as -4.60 mm and then increases with respect to depth at 9th case. In the other cases of second group, calculated displacements decrease at the bottom. Computed horizontal displacements without surcharge load is given on Table 7.20.

Table 7.20 Computed horizontal displacements of retained soil for sand backfill

H (m)	1st Case (mm)	2nd Case (mm)	3rd Case (mm)	4th Case (mm)	5th Case (mm)	6th Case (mm)	7th Case (mm)	8th Case (mm)	9th Case (mm)	10th Case (mm)	11th Case (mm)	12th Case (mm)	13th Case (mm)
6.0	50.03	14.71	60.51	55.41	-2.03	-2.26	-2.02	-2.75	-4.60	59.7	65.15	58.81	70.83
4.8	58.82	11.23	67.64	63.17	0.23	0.05	0.26	-0.31	-1.97	67.5	74.12	67.3	80.51
3.6	54.99	7.89	64.13	59.95	1.78	1.69	1.82	1.46	0.20	63.36	70.14	64.23	76.71
2.4	47.04	6.27	56.54	52.93	2.58	2.65	2.62	2.63	1.91	55.47	61.54	56.89	67.81
1.2	37.48	5.90	47.35	44.39	2.88	3.09	2.93	3.37	3.26	46.21	51.12	47.87	56.9
0	24.55	5.28	35.83	33.34	2.57	2.99	2.63	2.84	4.20	35.29	38.32	36.8	43.67

When surcharge load is applied, calculated displacement increases to 286.81 mm and 291.85 mm for 1st and 13th cases. When the latter displacement point is considered, calculated displacement increased to 320.85 mm and 310.40 mm respectively. This means that rate of increase change with respect to geosynthetic layer. If the second group is investigated, the calculated highest displacement is found as -4.10 mm at top and increased to 5.30 mm at the bottom. It should be noted that, application of surcharge load does not change the displacement behaviour of wall. Change of horizontal displacement of retained soil for different foundation conditions are given on Figure 7.14 below.

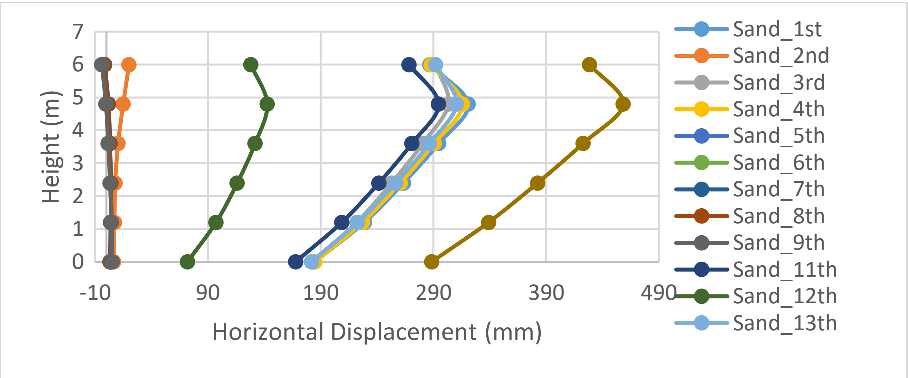


Figure 7.14 Horizontal Displacements Computed for Retained Soil for Reinforced Earth Wall with Sand Backfill

7.2.2.2.2 Horizontal Displacements of Retained Soil Behind 80% Sand Backfill

When the horizontal displacement of retained soil backfill with 80% sand content is investigated, two different behaviour is observed similar to sand backfill. The highest displacement is calculated for 10th case as 294.13 mm. This value is higher that 2 times of displacement calculated for 1st case which is 103.95 mm. Displacement increase at the following geosynthetic layer and then decreases with respect to height of the wall. The lowest displacements are computed at the bottom for all first group and it is found as 45.77 mm for 4th case. If the second group is considered, the highest and lowest displacements computed at the surface is found as -0.52 and -2.27 mm for 7th and 9th cases respectively. Change of horizontal displacement of retained soil with 80% sand backfill without surcharge load is given on Figure 7.15.

When surcharge load is applied, behaviour of some cases changes for the first group. As it is told before, horizontal displacement increases at the surface and then decrease with height. However, on 3rd, 10th and 13th cases, computed displacements start to decrease by increasing depth. In 12th case, there is a very insignificant increase between 1st and 2nd layer geosynthetic layer. Calculated displacements for 12th case at those levels are equal to 353.34 mm and 353.66 mm respectively. The highest amount of change is calculated for 3rd case as 341.93% at the surface. When the second group is considered under surcharge load, the lowest displacement is computed for 10th case as -0.91 mm as same as without surcharge load. Behaviour of displacements are not affected by surcharge load. Computed horizontal displacements of retained soil is given on Table 7.21.

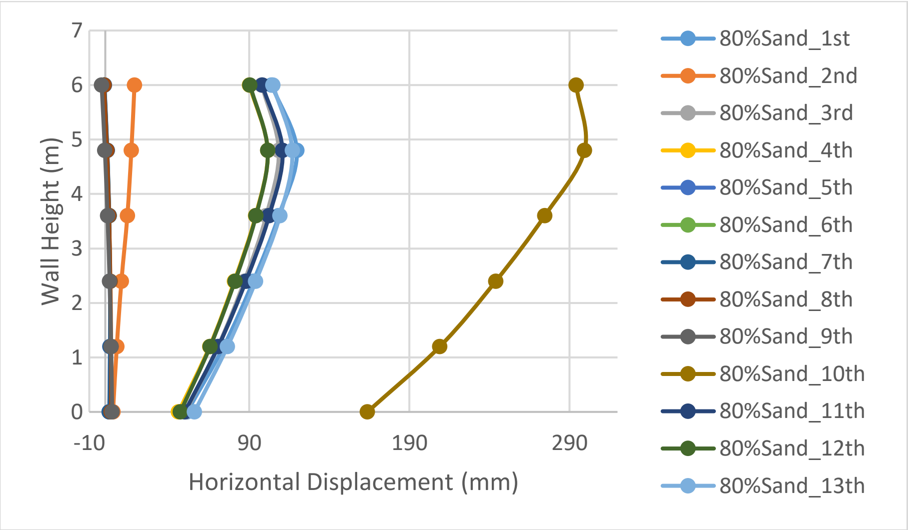


Figure 7.15 Horizontal Displacements Computed for Retained Soil for Reinforced Earth Wall with 80% Sand Backfill

Table 7.21 Computed Horizontal Displacements of Retained Soil under Surcharge Load

H (m)	1st Case (mm)	2nd Case (mm)	3rd Case (mm)	4th Case (mm)	5th Case (mm)	6th Case (mm)	7th Case (mm)	8th Case (mm)	9th Case (mm)	10th Case (mm)	11th Case (mm)	12th Case (mm)	13th Case (mm)
6.0	415.47	29.01	431.87	191.98	0.73	0.68	0.78	0.62	-0.91	480.99	372.57	353.34	409.97
4.8	424.25	25.37	423.67	201.67	3.18	3.11	3.23	3.14	1.71	474.39	376.01	353.66	403.63
3.6	387.43	20.12	390.75	185.46	4.04	4.06	4.1	4.19	3.17	436.58	345.06	324.81	371.65
2.4	345.08	13.45	351.27	162.82	4.19	4.34	4.25	4.67	4.22	393.16	307.1	289.32	331.51
1.2	297.9	9.30	307.32	136.34	3.92	4.25	3.99	4.83	4.98	345.47	264.53	249.73	287.05
0	240.23	5.95	251.14	98.81	3.13	3.69	3.21	4.59	5.5	285.87	211.34	198.9	231.59

7.2.2.2.3 Horizontal Displacements of Retained Soil Behind 60% Sand Backfill

Similar like previous cases, there are two behaviour in this case too. Same behaviour are observed for same cases. However, computed displacements are smaller than the calculated displacements for 80% sand backfill for same cases. The highest displacement is computed for the reference 1st case as 111.94 mm. The computed displacements increase at the second layer geosynthetic level for all first group cases. In case of second group cases, calculated deformations decrease at the bottom for 5th, 6th and 7th cases while it continues to increase for 8th and 9th case. The lowest displacement is computed for -3.91 mm for 9th case. Computed horizontal displacements of retained backfill without surcharge load is given on Table 7.22 below.

Table 7.22 Computed Horizontal Displacements of Retained Soil

H (m)	1st Case (mm)	2nd Case (mm)	3rd Case (mm)	4th Case (mm)	5th Case (mm)	6th Case (mm)	7th Case (mm)	8th Case (mm)	9th Case (mm)	10th Case (mm)	11th Case (mm)	12th Case (mm)	13th Case (mm)
6.0	111.94	12.56	80.07	78.7	-1.52	-1.75	-1.49	-2.1	-3.91	87.88	90.34	81.44	94.79
4.8	121.68	11.47	86.48	84.33	0.33	0.15	0.36	-0.1	-1.62	93.24	96.66	87.18	100.89
3.6	111.29	10.5	80.91	79.17	1.64	1.55	1.68	1.45	0.35	86.89	90.06	81.69	94.76
2.4	98.08	9.17	72	70.51	2.39	2.44	2.77	2.54	1.96	76.99	79.42	72.55	83.93
1.2	83.23	7.39	61.37	60.08	2.72	2.93	2.77	3.28	3.24	65.53	66.81	61.53	71.1
0	64.77	5.19	48.11	46.54	2.46	2.89	2.53	3.55	4.20	52.01	51.34	48.1	56.08

When surcharge load is applied, the highest displacement computed as 571.43 mm which is increased from 87.88 mm. The calculated displacement equals to 395.21 mm for the 1st case under surcharge load. However, behaviour of the displacement changed for 1st, 3rd, 4th, 10th, 11th, 12th and 13th cases. Horizontal displacement starts to decrease as the depth of the wall increases. The lowest displacement is also computed for 9th case as -3.23 mm under surcharge load. However, the lowest displacement at the bottom is computed for 5th case as 3.09 mm for 5th case as same as without surcharge load. The highest change in displacement is computed as 550.21% for 10th case while, it is calculated as 240% for reference 1st case. Horizontal displacements are given on Figure 7.16 for 60% backfill under surcharge load.

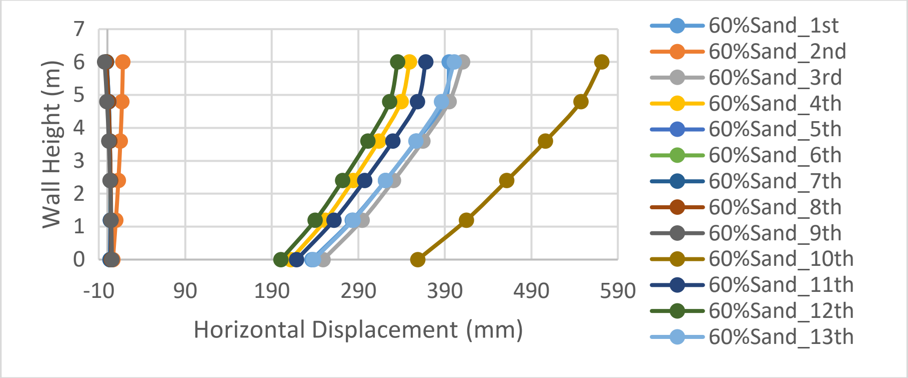


Figure 7.16 Horizontal Displacements Computed for Retained Soil for Reinforced Earth Wall with 60% Sand Backfill

7.2.2.2.4 Horizontal Displacements of Retained Soil Behind 40% Sand Backfill

When horizontal displacement of retained soil with 40% sand content backfill is investigated, the highest displacement is computed for 13th case at the surface and for 1st case at the level of second geosynthetic. Calculated displacements may be given as 91.80 mm and 103.59 mm respectively for locations. After an increase of displacement at the second layer level, displacement decrease as the depth increases for first group. When 2nd group is considered, same behaviour is observed as 60% sand content backfill. The lowest deformation is calculated for 9th case as -2.76 mm at the surface. However, lower displacement at the bottom is computed for 5th case as 2.48 mm. Change of horizontal displacement of retained soil is shown on Figure 7.17 given below.

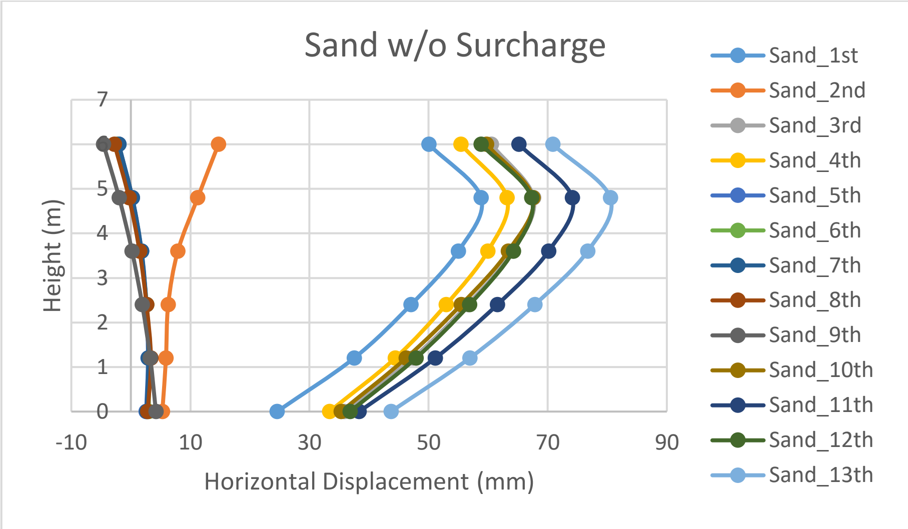


Figure 7.17 Horizontal Displacements Computed for Retained Soil for Reinforced Earth Wall with 40% Sand Backfill without surcharge load

When surcharge load is applied, the highest displacement is computed for 10th case as 422.35 mm. The increase on displacement is only computed for 1st case on the second layer of geosynthetic. The horizontal displacements decrease as the depth of wall increases. The highest amount of change is also calculated for 10th case as 400% averagely. The lowest displacements are computed as -2.03 mm and 3.11 mm at the surface and bottom for 10th and 5th cases respectively. Calculated horizontal displacements of retained soils for different foundation conditions under surcharge load is given on Table 7.23.

7.2.2.2.5 Horizontal Displacements of Retained Soil Behind 20% Sand Backfill

When behaviour of retained soil is considered with backfill of 20% sand content, it is seen that the displacements calculated for 1st and 13th cases insignificantly differ from each other. Calculated displacements are given as 86.68 mm and 86.23 mm for 1st and 13th cases respectively. Those horizontal displacements are also the highest displacements. The lowest displacement is computed as -4.08 mm at the surface for 9th case and 2.49 mm at the bottom of 5th case. Horizontal displacements calculated for retained soil with 20% sand backfill without surcharge load are provided on Table 7.24 below.

Table 7.23 Computed Horizontal Displacements of Retained Soil under Surcharge Load

H (m)	1st Case	2nd Case	3rd Case	4th Case	5th Case	6th Case	7th Case	8th Case	9th Case	10th Case	11th Case	12th Case	13th Case
6.0	372.99	20.98	383.15	360.02	-0.20	-0.29	-0.16	-0.40	-2.03	422.35	367.81	335.3	415.21
4.8	375.26	19.47	376.29	354.27	2.26	2.19	2.32	2.16	0.68	415.46	365.12	331.68	404.49
3.6	344.68	29.02	348.39	328.32	3.40	3.40	3.46	3.49	2.45	385.28	336.49	306.74	374.28
2.4	309.69	13.70	316.04	297.53	3.83	3.97	3.89	4.27	3.8	350.97	303.03	277.02	337.91
1.2	271.2	10.23	280.59	263.92	3.78	4.09	3.84	4.65	4.77	313.86	266.32	244.28	298.03
0	223.11	6.37	234.83	221.46	3.11	3.66	3.19	4.55	5.44	266.28	220.82	202.65	249.26

Table 7.24 Computed Horizontal Displacements of Retained Soil for 20% Sand Backfill

H (m)	1st Case	2nd Case	3rd Case	4th Case	5th Case	6th Case	7th Case	8th Case	9th Case	10th Case	11th Case	12th Case	13th Case
6.0	86.68	12.47	71.82	67.06	-1.64	-1.63	-1.63	-2.26	-4.08	73.53	81.42	75.37	86.23
4.8	94.21	11.54	78.43	73.24	0.25	0.06	0.26	-0.22	-1.75	79.81	88.49	81.46	93.37
3.6	87.70	10.45	74.34	69.72	1.61	1.51	1.62	1.37	0.27	75.47	83.16	77.23	88.54
2.4	78.38	9.21	67.00	62.91	2.39	2.42	2.41	2.5	1.91	67.89	74.03	69.15	79.30
1.2	67.42	7.46	57.89	54.49	2.74	2.93	2.77	3.27	3.23	58.79	63.03	59.30	67.97
0	53.14	5.29	46.24	42.96	2.49	2.90	2.54	3.56	4.20	47.61	49.22	47.10	54.39

When surcharge load is applied, the highest displacement equals to 495.39 mm for 10th case. The behaviour of displacement also changes when surcharge load is applied. Displacement decreases as the depth of wall increases. The lowest displacements are computed for same cases as without surcharge load case. The highest change of the horizontal displacement is calculated 550% for 10th case. The behaviour of horizontal displacements under surcharge load is given on Figure 7.18 below.

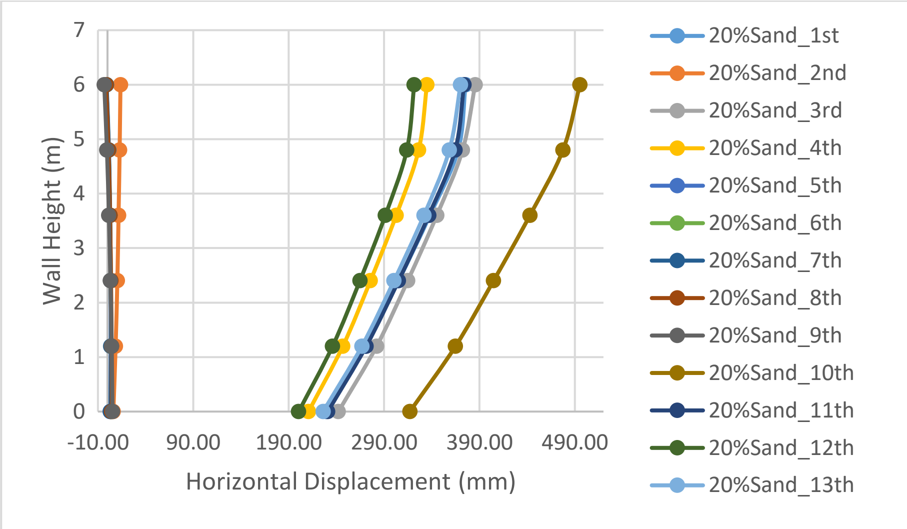


Figure 7.18 Horizontal Displacements Computed for Retained Soil for Reinforced Earth Wall with 20% Sand Backfill under surcharge load

7.2.2.2.6 Horizontal Displacements of Retained Soil Behind Clay Backfill

The highest displacement is calculated for 1st case as 75.61 mm. The deformation increases at the second layer geosynthetic layer for 1st, 3rd, 4th, 10th, 11th, 12th and 13th case. The lowest displacement is computed for 9th case at the surface and 5th case at the bottom as -4.83 mm and 2.50 mm respectively. Calculated horizontal displacements of retained soil is given on Table 7.25 below.

Table 7.25 Computed Horizontal Displacements of Retained Soil for Clay Backfill

H (m)	1st Case (mm)	2nd Case (mm)	3rd Case (mm)	4th Case (mm)	5th Case (mm)	6th Case (mm)	7th Case (mm)	8th Case (mm)	9th Case (mm)	10th Case (mm)	11th Case (mm)	12th Case (mm)	13th Case (mm)
6.0	75.61	8.77	58.59	53.02	-2.12	-2.37	-2.08	-2.90	-4.83	64.14	66.32	57.89	71.29
4.8	81.68	8.33	64.88	59.66	-0.11	-0.30	-0.06	-0.70	-2.32	64.14	73.67	64.25	78.69
3.6	76.33	8.02	62.55	57.51	1.36	1.27	1.42	1.04	-0.14	65.92	70.05	62.17	76.03
2.4	68.57	7.45	57.04	52.53	2.25	2.29	2.31	2.29	1.64	59.78	63.11	56.52	68.53
1.2	59.34	6.53	49.95	45.99	2.68	2.89	2.74	3.17	3.09	52.07	54.31	49.17	59.40
0	47.34	5.23	40.58	37.00	2.50	2.92	2.57	3.54	4.14	42.61	43.23	39.81	48.34

When surcharge load is applied, the highest amount of change occurs at 10th case as 450% averagely. The highest horizontal displacement calculated as 385.7 mm at the surface. The lowest displacements are computed for the same cases with and without surcharge load cases. Computed values become -4.18 mm and 3.13 mm at the top and bottom for 9th and 10th cases respectively. The change of horizontal displacements of retained soil with respect to height of the reinforced earth wall under surcharge load is given on Figure 7.19 below.

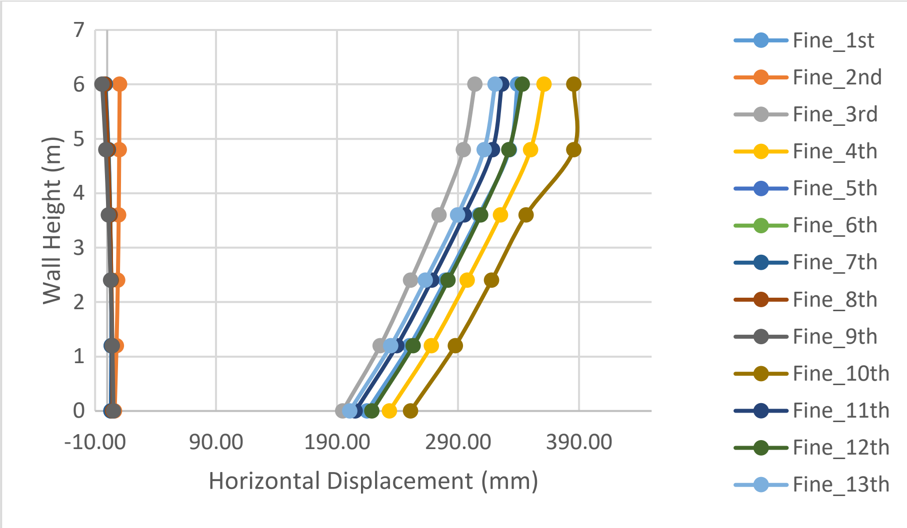


Figure 7.19 Horizontal Displacements Computed for Retained Soil for Reinforced Earth Wall with Clay Backfill under Surcharge Load

7.2.2.3 Effect of Foundation Soil Properties to Settlement of Reinforced Earth Wall

Settlement of reinforced earth walls are also important during design stage and their service life. Settlement of reinforced earth wall is affected directly by foundation conditions. The effect of foundation conditions is evaluated in this section for different backfill types and foundation conditions.

7.2.2.3.1 Settlement Behaviour of Reinforced Earth Wall with Sand Backfill

When settlement of foundation is investigated, three different behaviors are observed. In the first type of behaviour, the highest settlement is computed under the wall face. Settlement decreases toward end of reinforced zone. In case of second type of behaviour, lower settlement is observed under the wall face and settlement increase at the end of reinforced zone. The third type of behaviour is observed only on one case which is 7th case. In 7th case, settlement initially decreases and then increases at the end of reinforced zone. The first group may be listed as 1st, 2nd, 3rd, 4th, 10th, 11th, 12th and 13th cases. The second group maybe given as 5th, 6th, 7th, 8th and 9th cases. The highest settlement is computed as 98.50 mm and 27.66 mm for first and second group respectively. When surcharge load is applied, settlements increases but the behaviors remains the same. The highest settlements become 207.9 mm and 30.46 mm for the same cases when surcharge load is applied. The change of settlement of reinforced earth wall with sand backfill depending on foundation conditions are given on Figure 7.20 below.

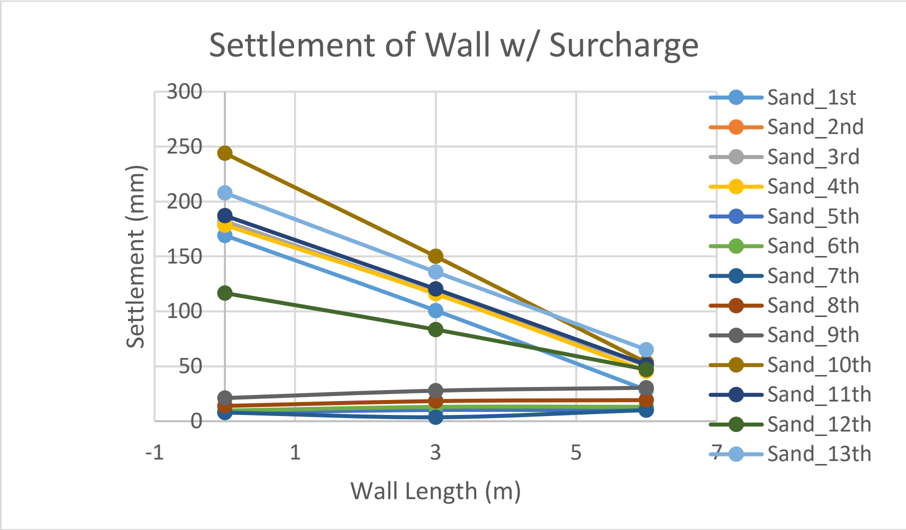


Figure 7.20 Settlement of Reinforced Earth Wall with Sand Backfill Under Surcharge Load

7.2.2.3.2 Settlement Behaviour of Reinforced Earth Wall with 80% Sand Backfill

When the backfill consists of only 80% sand and cohesive content, computed settlements increase in each case compared with sand backfill. However, increase in 1st case is so much higher than the other cases compared with sand backfill. Settlements are computed as 233.39 mm, 139.59 mm and 35.43 mm from beginning of the wall to end of wall for 1st case. Settlements gets even higher when surcharge load is applied. The computed settlements become 614.38 mm, 361.47mm and 73.59mm for the first case. Similar to sand backfill case, there is a second group where settlement tends to increase from wall face to end of reinforced soil zone. However, in 7th case, settlement increases at the first step and then decreases. Change of settlements with respect to different foundation conditions are given on Figure 7.21 below for 80% sand backfill.

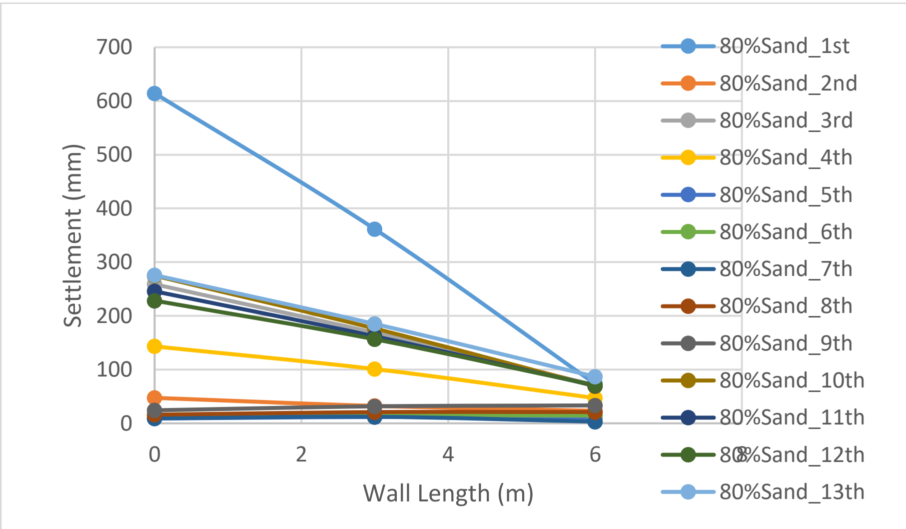


Figure 7.21 Settlement of Reinforced Earth Wall with 80% Sand Backfill Under Surcharge Load

7.2.2.3.3 Settlement Behaviour of Reinforced Earth Wall with 60% Sand Backfill

When the settlement of foundation is investigated for 60% sand backfill, it is seen that, computed settlements are lower than the settlements computed for 80% sand content backfill. In case of 60% sand content, only two different type of behaviour is observed. In first type of behaviour, settlement is higher next to the wall face and lower at the end of reinforced soil zone. In the second group, settlement is higher at the end of reinforced soil zone and lower at the wall face. The first group may be listed as 1st, 2nd, 3rd, 4th, 10th, 11th, 12th and 13th while the second group may be listed as 5th, 6th, 7th, 8th and 9th case. It should also be noted here that; computed settlements decrease insignificantly after middle point of reinforced soil zone for 5th and 7th cases. The highest settlement for first group is computed for 11th case as 106.93 mm and 30.37 mm for second group for 9th case. When the surcharge load is applied, the highest settlements for 1st and 2nd group increase to 240.9 mm and 33.3 mm for 3rd and 9th cases respectively. Change of computed settlements with respect to distance from wall face for different foundation conditions are provided on Figure 7.22 below.

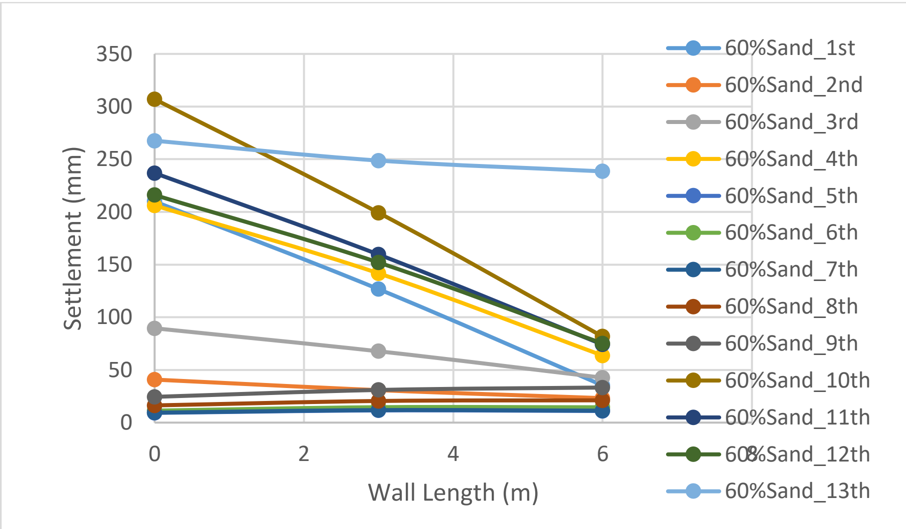


Figure 7.22 Settlement of Reinforced Earth Wall with 60% Sand Backfill Under Surcharge Load

7.2.2.3.4 Settlement Behaviour of Reinforced Earth Wall with 40% Sand Backfill

When settlement of wall with 40% sand backfill is considered, two different behaviour is observed as well. Computed settlements are close to computed settlements for 60% sand backfill case. However, there are two exceptions such as 1st and 13th cases. When 1st case is considered, lower settlements are computed for 40% backfill case than 60% sand backfill. However, amount of difference decreases as the distance with wall face increases. When 13th case is considered, higher settlements are computed for 40% sand backfill case than 60% sand backfill case. The difference between computed settlements decrease as the distance from wall face increases. The highest settlements are computed for 13th case for the first group which are 124.85 mm, 96.11 mm and 64.78 mm. The highest settlement for the second group may be given as 21.88 mm, 28.37 mm and 30.36 mm for 9th case. When surcharge load is applied all calculated settlements increase. The highest settlements are calculated as 274.02 mm, 186.86 mm and 90.34 mm for 13th case and 24.35 mm, 31.30 mm and 33.31 mm for 9th case under surcharge load. Change of settlements with respect to foundation conditions are provided on Figure 7.23 for 40% backfill.

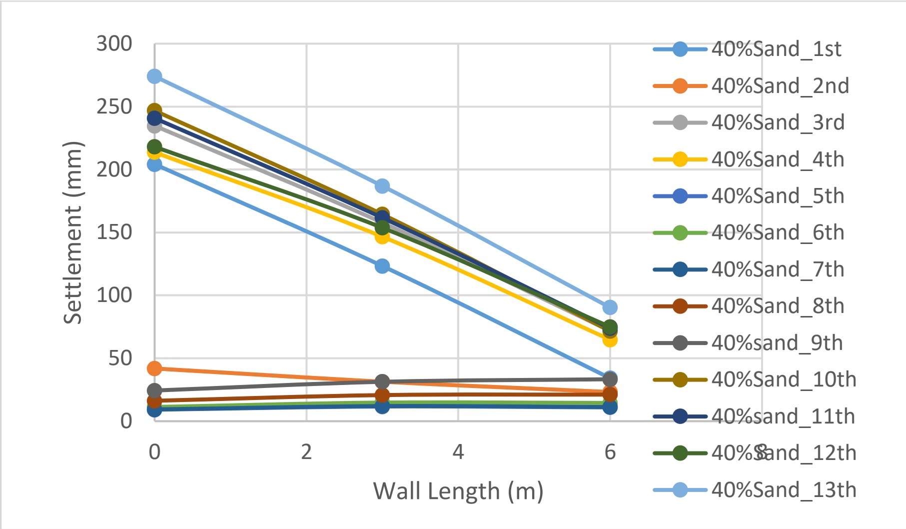


Figure 7.23 Settlement of Reinforced Earth Wall with 40% Sand Backfill Under Surcharge Load

7.2.2.3.5 Settlement Behaviour of Reinforced Earth Wall with 20% Sand Backfill

When settlement of foundation is considered for 20% sand content as backfill, it is seen that the highest settlement is calculated for 13th case as 115.93 mm, 90.97 mm and 63.15 mm from the wall face towards end of reinforced zone. The settlements show two different behaviors as well for this soil content too. The highest settlements are calculated for 9th case as 21.44 mm, 27.59 mm and 29.93 mm from wall face toward end of reinforced zone for second group. When surcharge load is applied, settlements increase for each case. However, rate of increase is higher for 10th case than 13th case. This means that the highest settlement is computed for 10th case as 272.38 mm, 179.04 mm and 76.21 mm from wall face to end of reinforced soil zone under surcharge load. The highest settlement for second group is also calculated for 9th case as 23.90 mm, 30.46 mm and 32.86 mm from wall face towards end of reinforced soil zone under surcharge load. Change of settlements for different foundation conditions under surcharge load is given on Figure 7.24 below.

7.2.2.3.6 Settlement Behaviour of Reinforced Earth Wall with Clay Backfill

In case of clay backfill, two different behaviour is observed as well. First group may be listed as 1st, 2nd, 3rd, 4th, 9th, 10th, 11th, 12th and 13th cases and the second group may be listed as 5th, 6th, 7th and 8th cases. Settlement is the highest under the wall face for the first group while it is the lowest for the second group. The highest settlements may be given as 100.14 mm, 79.55 mm, 57.05mm and 19.93 mm, 26.02 mm, 28.67 mm for 13th and 9th cases respectively for 1st and 2nd group. When surcharge load is applied, the computed settlements increase. However, the highest settlement is calculated for 3rd case as 227.40 mm, 153.82 mm, 71.55 mm and for 9th case as 22.31 mm, 28.83 mm and 31. 55 mm under surcharge load. Change of settlement of wall is given on Figure 7.25 under surcharge load for clay backfill.

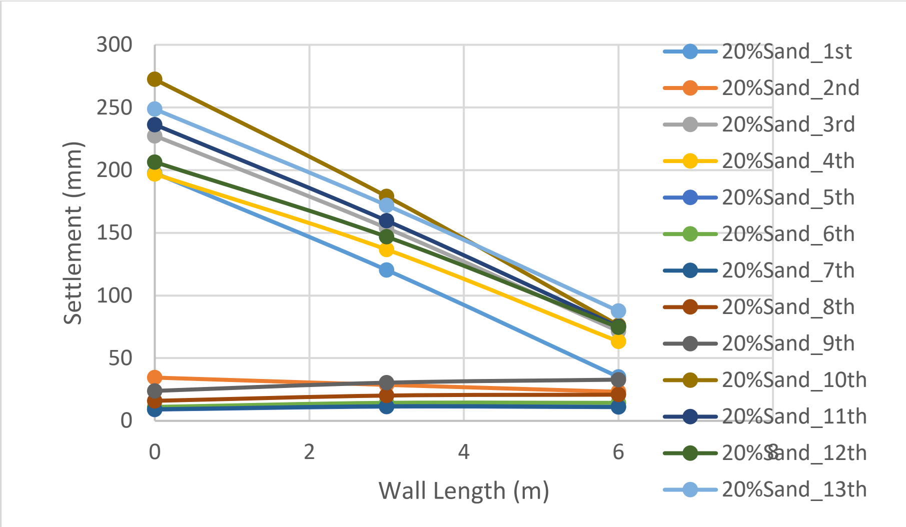


Figure 7.24 Settlement of Reinforced Earth Wall with 20% Sand Backfill Under Surcharge Load

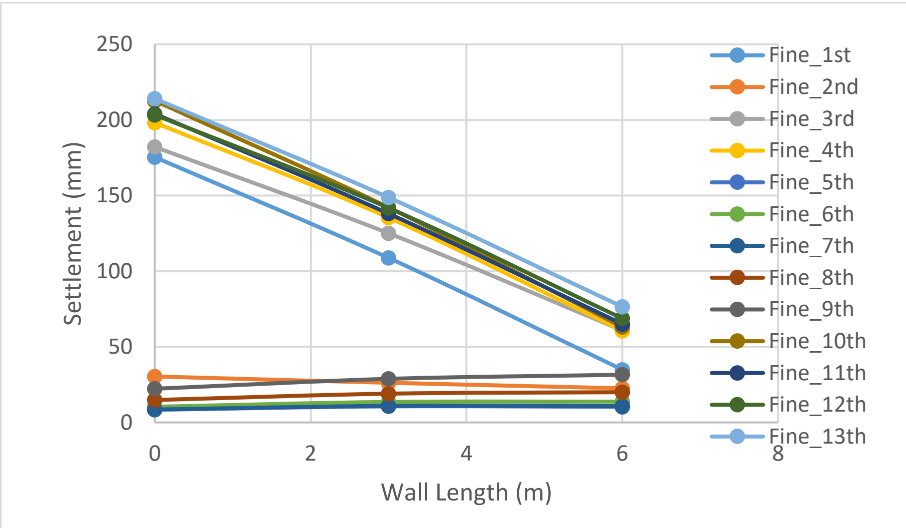


Figure 7.25 Settlement of Reinforced Earth Wall with Clay Backfill Under Surcharge Load

7.2.2.4 Effect of Foundation Soil Properties to Computed Maximum Horizontal Forces on Reinforcement

Finite element results show that, maximum forces on reinforcement also depends on the foundation conditions. Effect of foundation conditions will be evaluated separately for each type of backfill.

7.2.2.4.1 Maximum Forces Carried Out by Reinforcement – Sand Backfill

When forces developed on geosynthetic is investigated two different behaviour occurs as well. The first type is observed for 1st, 2nd, 3rd, 4th, 10th, 11th, 12th and 13th cases while the second type is observed for 5th, 6th, 7th, 8th and 9th cases. When first case is considered, it is seen that, maximum force increases as the depth of geosynthetic layer increases. However, in some cases such as 1st, 4th, 10th, 11th, 12th small decrease is observed on maximum force at 3rd geosynthetic layer. The amount of decrease equals to 0.24 kN/m averagely. The highest maximum force is calculated at the last layer geosynthetic. The magnitude of the maximum force changes between 17.43 kN/m and 9.56 kN/m computed for 4th and 2nd cases respectively for the first group. Resultant force on geosynthetics is also dependent on considered case. The highest and lowest resultant force is calculated as 78.59 kN/m and 65.01 kN/m for 4th and 2nd case respectively for the first group. When second group is investigated, maximum force increases with respect to depth until last layer. Maximum force significantly decreases at the last layer to 0.20 kN/m in general for the second group. Therefore, the highest maximum forces are calculated on previous layer on each case of second group. The highest and the lowest maximum forces are found as 3.40 kN/m and 2.81 kN/m for 9th and 5th cases respectively. The sum of the computed forces equals to approximately 26 kN/m for second group. There is not significant change in resultant maximum force with respect to considered case.

When surcharge load is applied, maximum force enormously increases at the first layer. This enormous increase generally equals to 200%. After first layer, calculated maximum force starts to decrease. This decrease is only valid at the second layer geosynthetic for 2nd, 3rd, 4th, 10th, and 13th cases of first group. Maximum force continues to decrease at the 3rd layer and then starts to increase. In case of second group, this behaviour is not observed. Similarly, without surcharge load, the highest and lowest maximum force is computed for 4th and 2nd case as 26.34 kN/m and 11.06 kN/m respectively for the first group. The highest and lowest total maximum force is also calculated for some cases as 105.63 kN/m and 83.64 kN/m respectively. When second group is considered, the lowest maximum force is calculated at the last layer while the highest maximum force calculated at the previous layer. The highest force increases to minimum 3.14 kN/m and maximum 3.85 kN/m for 5th and 9th cases respectively. The resultant of the maximum forces is found to be 32.50 kN/m averagely which means, resultant of the maximum force is not case dependent for second group. Calculated maximum horizontal forces and resultant horizontal forces are given on Table 7.26.

When forces given on Table 7.26 is compared with forces given on Table 7.8, it is seen that, the highest differences occur on the deepest layer of reinforcement. This could be due to that; extra forces might be generated due to sliding of wall. However, due to this high difference at the deepest layer, difference between resultant forces also increases. The computed differences between FHWA and Plaxis are given on Table 7.27. Table 7.27 shows that, FHWA method agrees well with Plaxis when reinforcement is not close to surface. The difference decreases below the 25% range after 3rd layer of reinforcement except for 5th, 6th, 7th, 8th and 9th cases.

Table 7.26 Computed maximum and resultant horizontal forces on each reinforcement layer for Sand backfill

Z (m)	1st Case kN/m	2nd Case kN/m	3rd Case kN/m	4th Case kN/m	5th Case kN/m	6th Case kN/m	7th Case kN/m	8th Case kN/m	9th Case kN/m	10th Case kN/m	11th Case kN/m	12th Case kN/m	13th Case kN/m
0	4.79	1.69	5.10	4.76	0.75	0.73	0.74	0.75	0.72	5.08	4.58	3.31	5.14
0.4	2.64	2.30	1.66	2.12	1.07	1.02	1.06	1.07	1.02	2.47	2.33	2.33	1.70
0.8	2.39	2.95	1.83	2.18	1.38	1.32	1.37	1.38	1.32	2.50	2.16	1.98	1.78
1.2	2.62	3.55	2.19	2.42	1.69	1.62	1.68	1.68	1.60	2.70	2.45	2.34	2.17
1.6	2.98	4.11	2.86	2.84	1.98	1.90	1.97	2.01	1.88	3.02	2.89	2.81	2.87
2.0	3.61	4.56	3.78	3.68	2.27	2.20	2.26	2.27	2.18	3.91	3.85	3.58	4.00
2.4	4.55	5.03	4.90	4.75	2.50	2.45	2.49	2.51	2.45	4.63	4.90	4.71	4.94
2.8	5.70	5.50	5.80	5.63	2.65	2.62	2.63	2.65	2.64	5.63	5.78	5.60	5.80
3.2	6.50	5.95	6.57	6.40	2.68	2.67	2.66	2.65	2.64	6.39	6.48	6.26	6.61
3.6	7.16	6.39	7.29	7.02	2.81	2.67	2.81	2.77	2.68	6.96	7.21	7.00	7.27
4.0	7.78	6.84	7.94	7.66	3.09	2.93	3.08	2.96	2.91	7.59	7.82	7.53	7.88
4.4	8.51	7.40	8.64	8.61	3.26	3.03	3.25	3.28	3.02	8.40	8.53	8.17	8.55
4.8	9.67	7.90	9.51	9.84	3.15	3.14	3.13	3.17	3.13	9.39	9.63	9.20	9.45
5.2	11.11	8.41	10.56	11.37	3.14	3.76	3.20	3.28	3.85	10.80	10.95	10.37	10.44
5.6	18.62	11.06	17.34	26.34	0.25	0.22	0.26	0.18	0.22	17.58	16.21	22.47	16.76
Resultant	98.63	83.64	95.97	105.63	32.69	32.28	32.59	32.62	32.25	97.06	95.77	97.66	95.36

Table 7.27 Differences between FHWA and Plaxis for maximum reinforcement loads for Sand Backfill

Z (m)	1st Case (%)	2nd Case (%)	3rd Case (%)	4th Case (%)	5th Case (%)	6th Case (%)	7th Case (%)	8th Case (%)	9th Case (%)	10th Case (%)	11th Case (%)	12th Case (%)	13th Case (%)
0	145.79	13.49	161.45	144.19	61.42	62.49	61.90	61.40	63.02	160.67	134.96	69.77	163.69
0.4	30.45	13.79	17.71	4.92	47.00	49.66	47.48	47.05	49.46	22.35	15.32	15.13	15.97
0.8	7.88	13.78	29.34	15.64	46.55	48.94	46.93	46.80	49.22	3.47	16.71	23.55	31.27
1.2	17.18	12.42	30.76	23.43	46.58	48.82	46.82	46.72	49.31	14.42	22.57	25.90	31.43
1.6	20.24	10.27	23.24	23.82	46.94	48.99	47.31	46.13	49.69	19.12	22.65	24.66	22.99
2.0	16.00	6.11	12.16	14.40	47.26	48.90	47.54	47.26	49.27	9.02	10.44	16.71	7.02
2.4	6.70	2.98	0.32	2.71	48.70	49.77	48.91	48.63	49.86	5.09	-0.44	3.52	1.13
2.8	4.62	0.93	6.48	3.33	51.38	52.00	51.77	51.37	51.51	3.38	6.10	2.80	6.41
3.2	7.96	1.23	9.11	6.34	55.46	55.62	55.75	55.94	56.10	0.06	0.08	0.04	0.10
3.6	8.71	3.03	10.66	6.48	57.30	59.47	57.42	57.91	59.36	5.57	9.44	6.22	10.38
4.0	8.65	4.48	10.94	7.01	56.84	59.13	56.96	58.70	59.30	6.00	9.27	5.19	10.10
4.4	10.10	4.28	11.80	11.39	57.77	60.77	57.97	57.53	60.95	8.67	10.36	5.67	10.66
4.8	16.40	4.93	14.46	18.37	62.15	62.17	62.35	61.85	62.35	12.98	15.87	10.71	13.72
5.2	25.16	5.27	18.91	28.07	64.59	57.67	63.94	63.05	56.65	21.61	23.27	16.81	17.61
5.6	300.39	137.89	272.82	466.42	94.56	95.27	94.41	96.04	95.35	278.09	248.53	383.17	260.35
Resultant	27.40	8.03	23.96	36.43	57.78	58.31	57.90	57.87	58.34	25.37	23.70	26.15	23.18

7.2.2.4.2 Maximum Forces Carried Out by Reinforcement – 80% Sand Backfill

When the forces acting over geosynthetic are investigated with 80% sand content backfill, two different behaviour is observed as well. The first group may be listed as 1st, 2nd, 3rd, 4th, 10th, 11th, 12th and 13th cases while the second group may be listed as 5th, 6th, 7th, 8th and 9th cases. When the first group is considered, it is seen that maximum force is lower at the second layer than first layer for 1st, 10th and 13th cases. In cases such as 3rd, 4th, 11th and 12th cases decrement of maximum force continues till 3rd layer. After decrements, calculated maximum force gradually increases with depth. When 2nd case is considered, the decrement is not observed. The maximum force is calculated at the deepest layer. The highest and lowest maximum forces are calculated as 21.64 kN/m and 10.03 kN/m for 4th and 2nd cases respectively. The highest and the lowest resultant forces are also computed for same cases as 78.75 kN/m and 53.93 kN/m. When second group is investigated, maximum force increases as the depth of the layer increases, however, maximum force decreases at the last layer dramatically. The computed maximum force changes between 2.14 kN/m and 2.74 kN/m depending on the considered case. Resultant of the maximum forces changes between 10.68 kN/m and 11.03 kN/m for 5th and 9th cases.

When surcharge load is applied, all maximum forces on geotextile are increased. The highest and the lowest load calculated for 1st group increases to 27.95 kN/and 12.66 kN/m for 4th and 2nd cases respectively. The highest and the lowest resultant forces are computed as 93.31 kN/m and 79.82 kN/m at first group. When second group is considered, the highest and lowest forces are computed as 3.37 kN/m and 2.64 kN/m respectively. The highest and the lowest resultant forces are calculated as 17.68 kN/m and 16.29 kN/m for 9th and 5th cases. Change of maximum horizontal forces depending on foundation conditions are given on Figure 7.26.

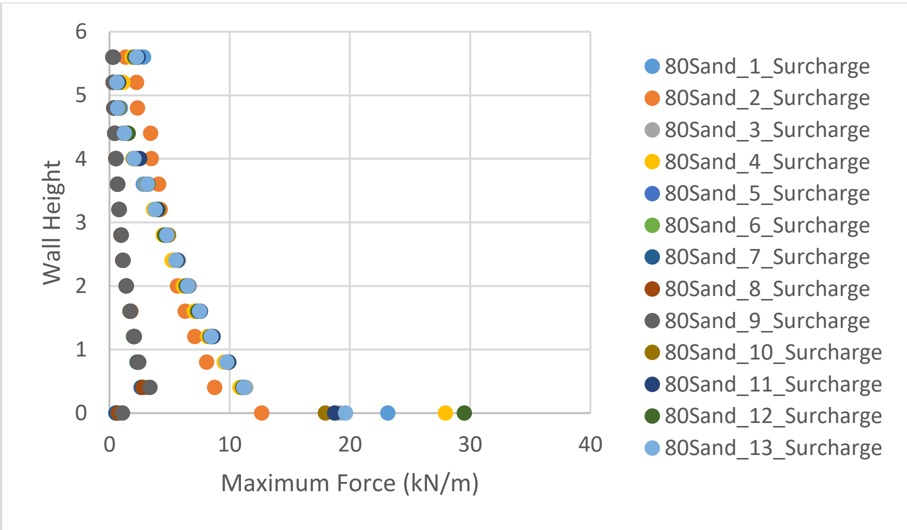


Figure 7.26 Settlement of Reinforced Earth Wall with Clay Backfill Under Surcharge Load

If results of finite element models are compared with results given on Table 7.8, difference is found to be higher due to effect of cohesion. Finite element method significantly lowered maximum horizontal forces until last reinforcement layer. Computed force significantly increases at last layer of reinforcement for 1st, 3rd, 4th, 10th, 11th, 12th and 13th cases. The computed maximum horizontal forces are given on Table 7.28 below.

Table 7.28 Computed maximum and resultant horizontal forces on each reinforcement layer for 80% Sand backfill

Z (m)	1st Case kN/m	2nd Case kN/m	3rd Case kN/m	4th Case kN/m	5th Case kN/m	6th Case kN/m	7th Case kN/m	8th Case kN/m	9th Case kN/m	10th Case kN/m	11th Case kN/m	12th Case kN/m	13th Case (kN/m)
0	2.86	1.36	2.26	1.88	0.31	0.28	0.31	0.29	0.28	2.29	2.39	2.09	2.25
0.4	1.05	2.24	0.66	1.13	0.32	0.31	0.31	0.32	0.33	0.70	0.76	0.66	0.63
0.8	0.94	2.33	0.73	0.70	0.38	0.37	0.38	0.38	0.39	0.80	0.66	0.71	0.68
1.2	1.13	3.44	1.31	1.50	0.45	0.44	0.45	0.45	0.45	1.30	1.26	1.57	1.25
1.6	2.26	3.47	2.22	1.97	0.55	0.53	0.55	0.54	0.54	2.21	2.53	2.02	2.06
2.0	2.81	4.09	3.17	3.07	0.68	0.67	0.68	0.67	0.67	3.19	3.13	3.21	3.17
2.4	4.01	4.23	4.10	3.66	0.82	0.80	0.81	0.80	0.80	4.09	3.98	3.81	3.79
2.8	4.73	4.84	4.80	4.46	0.97	0.97	0.97	0.95	0.98	4.91	4.63	4.60	4.79
3.2	5.46	5.27	5.75	5.21	1.12	1.14	1.12	1.11	1.13	5.62	5.67	5.56	5.53
3.6	6.38	5.67	6.64	6.14	1.40	1.38	1.40	1.41	1.41	6.55	6.54	6.38	6.50
4.	7.41	6.31	7.59	7.04	1.75	1.69	1.76	1.77	1.73	7.43	7.57	7.33	7.50
4.4	8.48	7.10	8.60	8.17	2.04	2.00	2.04	2.07	2.07	8.60	8.62	8.37	8.46
4.8	9.82	8.08	9.93	9.56	2.33	2.29	2.33	2.41	2.43	9.88	9.94	9.80	9.76
5.2	11.21	8.74	11.35	10.87	2.64	3.15	2.68	2.80	3.37	11.08	11.15	11.11	11.19
5.6	23.17	12.66	19.17	27.95	0.53	0.54	0.55	0.73	1.09	17.95	18.74	29.53	19.63
Resultant	91.72	79.82	88.28	93.31	16.29	16.57	16.34	16.69	17.68	86.60	87.58	96.74	87.19

7.2.2.4.3 Maximum Forces Carried Out by Reinforcement – 60% Sand Backfill

When forces formed on geosynthetic is investigated, two groups can be seen. These group can be listed as 1st, 2nd, 3rd, 4th, 10th, 11th, 12th, 13th and 5th, 6th, 7th, 8th and 9th cases as first and second groups respectively. The resultant forces change between 42.13 kN/m and 50.30 kN/m. The lowest resultant force is computed for 13th case while the highest resultant force is computed for the 4th case. When the second group is considered, the resultant maximum forces found to be between 5 kN/m and 5.39 kN/m for 5th and 9th cases respectively. When the surcharge load is applied, computed horizontal resultant forces increase. The highest and lowest resultant forces are computed as 69.56 kN/m and 51.18 kN/m for 4th and 2nd cases respectively. When second group is considered, the lowest and highest resultant forces increased to 6.47 kN/m and 7.02 kN/m for 5th and 9th cases. Calculated maximum horizontal forces and resultant horizontal forces are given on Table 7.29 below. When computed forces by finite element method are compared with maximum horizontal forces calculated by FHWA method, huge difference is observed. Due to cohesion of backfill, all forces are calculated as negative. In all cases positive forces are computed by finite element method, however, almost computed forces are slightly higher than zero for 5th, 6th, 7th, 8th and 9th cases.

7.2.2.4.4 Maximum Forces Carried Out by Reinforcement – 40% Sand Backfill

There are two types of behaviour as previous backfills considered for the same cases. The highest force is computed at the last layer for the first group. When second group is considered, the highest reinforcement force is computed previous layer of last layer. The highest and lowest resultant forces are computed as 50.72 kN/m and 37.57 kN/m for 4th and 2nd cases for 1st group. When second group is considered the highest and lowest resultant forces are computed for 9th and 5th cases as 7.09 kN/m and 6.81 kN/m respectively. When surcharge load is applied, those forces increase to 9.22 kN/m and 8.56 kN/m respectively. When those forces are compared with calculated forces with FHWA method, huge difference is observed. However, as 60% sand backfill, almost zero maximum horizontal forces are computed for 5th, 6th, 7th, 8th and 9th cases. Computed maximum horizontal forces are provided on Table 7.30 below. Force jump also observed at the last reinforcement layer for 1st behaviour group.

Table 7.29 Computed maximum and resultant horizontal forces on each reinforcement layer for 60% Sand backfill

Z (m)	1st Case kN/m	2nd Case kN/m	3rd Case kN/m	4th Case kN/m	5th Case kN/m	6th Case kN/m	7th Case kN/m	8th Case kN/m	9th Case kN/m	10th Case kN/m	11th Case kN/m	12th Case kN/m	13th Case kN/m
0	2.85	1.55	1.79	2.08	0.25	0.22	0.25	0.23	0.18	2.16	2.29	2.02	1.86
0.4	0.67	2.00	0.50	0.73	0.20	0.19	0.20	0.19	0.17	0.91	0.65	0.53	0.56
0.8	0.62	1.87	0.53	0.51	0.21	0.21	0.21	0.20	0.18	0.48	0.53	0.53	0.52
1.2	0.94	2.26	0.82	0.91	0.22	0.21	0.22	0.21	0.19	0.86	0.95	0.90	0.83
1.6	1.25	2.14	1.39	1.37	0.23	0.23	0.23	0.22	0.22	1.26	1.50	1.38	1.21
2.0	1.59	2.53	1.73	1.69	0.26	0.26	0.26	0.26	0.26	1.71	1.79	1.71	1.63
2.4	2.18	2.51	2.07	2.18	0.29	0.29	0.29	0.29	0.29	2.04	2.37	2.16	2.40
2.8	2.56	2.88	2.73	2.64	0.33	0.33	0.33	0.33	0.33	2.60	2.72	2.75	2.75
3.2	2.97	3.00	3.15	3.05	0.37	0.37	0.37	0.37	0.37	2.96	3.30	3.15	3.15
3.6	3.46	3.24	3.63	3.49	0.41	0.42	0.41	0.41	0.42	3.54	3.78	3.58	3.67
4.0	3.95	3.61	4.31	4.15	0.47	0.47	0.47	0.47	0.47	4.00	4.41	4.19	4.35
4.4	4.72	4.05	4.73	4.81	0.58	0.59	0.58	0.60	0.63	4.59	4.96	4.84	4.75
4.8	5.48	4.72	5.61	5.72	0.83	0.81	0.84	0.89	0.88	5.42	5.76	5.70	5.64
5.2	7.02	5.44	7.02	7.27	1.54	1.86	1.59	1.71	2.20	6.71	7.10	7.15	7.03
5.6	18.80	9.38	18.94	28.94	0.27	0.24	0.26	0.19	0.22	15.26	17.30	27.91	18.10
Resultant	59.07	51.18	58.95	69.56	6.47	6.70	6.51	6.59	7.02	54.50	59.39	68.49	58.45

Table 7.30 Computed maximum and resultant horizontal forces on each reinforcement layer for 40% Sand backfill

Z (m)	1st Case $kN/_m$	2nd Case $kN/_m$	3rd Case $kN/_m$	4th Case $kN/_m$	5th Case $kN/_m$	6th Case $kN/_m$	7th Case $kN/_m$	8th Case $kN/_m$	9th Case $kN/_m$	10th Case $kN/_m$	11th Case $kN/_m$	12th Case $kN/_m$	13th Case $kN/_m$
0	3.34	1.62	2.34	2.45	0.31	0.29	0.31	0.29	0.23	2.57	2.76	2.35	2.11
0.4	0.74	1.53	0.55	0.80	0.27	0.25	0.27	0.25	0.23	0.63	0.65	0.62	0.59
0.8	0.63	2.08	0.69	0.47	0.28	0.28	0.28	0.27	0.24	0.62	0.69	0.50	0.51
1.2	0.79	1.99	0.79	0.72	0.30	0.29	0.30	0.29	0.29	0.78	0.79	0.92	0.78
1.6	1.38	2.51	1.35	1.11	0.34	0.34	0.34	0.34	0.33	1.22	1.41	1.26	1.15
2.0	1.77	2.47	1.73	1.56	0.39	0.39	0.39	0.39	0.38	1.72	1.78	1.65	1.55
2.4	2.14	2.52	2.09	1.97	0.44	0.44	0.44	0.44	0.44	2.09	2.14	2.03	2.34
2.8	2.67	2.60	2.80	2.45	0.49	0.49	0.49	0.49	0.49	2.65	2.79	2.57	2.70
3.2	3.12	2.82	3.24	2.84	0.55	0.55	0.55	0.55	0.55	3.00	3.20	3.00	3.09
3.6	3.62	3.08	3.74	3.34	0.61	0.62	0.61	0.62	0.62	3.56	3.67	3.48	3.62
4.0	4.13	3.49	4.38	3.87	0.69	0.69	0.69	0.69	0.70	4.09	4.32	4.11	4.34
4.4	4.85	3.96	5.00	4.69	0.81	0.80	0.81	0.83	0.83	4.72	5.03	4.80	4.87
4.8	5.67	4.76	5.87	5.62	1.10	1.08	1.10	1.17	1.18	5.70	5.84	5.73	5.69
5.2	7.37	5.63	7.41	7.24	1.73	2.11	1.77	1.91	2.40	7.00	7.33	7.28	7.26
5.6	19.00	9.23	18.04	29.57	0.26	0.23	0.25	0.29	0.31	16.86	16.92	28.82	18.88
Resultant	61.22	50.28	60.01	68.68	8.56	8.83	8.60	8.81	9.22	57.21	59.33	69.13	59.48

7.2.2.4.5 Maximum Forces Carried Out by Reinforcement – 20% Sand Backfill

When forces on geosynthetic considered backfill with 20% sand content, two different type of behaviour is determined from the results. In case of first group, computed maximum forces decreases at initial layers and then increases as the depth of layer increases. The highest maximum forces calculated at the last layer for this group. This group may be listed as 1st, 2nd, 3rd, 4th, 10th, 11th, 12th and 13th cases. When second group is considered, maximum force on geosynthetic decreases insignificantly and then increases until the second to last layer. The maximum force at the last layer is lower than force at previous layers. The highest and the lowest resultant forces are computed as 40.02 kN/m and 30.90 kN/m, 4.84 kN/m and 4.75 kN/m for first and second group respectively. When surcharge load is applied those forces increase to 58.25 kN/m, 31.81 kN/m and

6.10 kN/m, 5.82 kN/m respectively. Computed forces for each case are provided in Table 7.31 below.

7.2.2.4.6 Maximum Forces Carried Out by Reinforcement – Clay Backfill

When only a clay type backfill is used, two different behaviour is observed depending on foundation conditions. In case of first type of behaviour, maximum force on geosynthetic, decreases for the top layers and then increases. The highest maximum force is computed at the last layer. The highest and lowest resultant forces are computed as 32.73 kN/m and 23.43 kN/m. When surcharge load is applied, these loads increase to 51.22 kN/m and 25.53 kN/m. When the second group is considered, the maximum force decreases initially like first group. After that, computed force increases with respect to depth, but lower maximum force is computed at last layer. The highest and lowest resultant force is computed as 4.40 kN/m and 4.33 kN/m. When surcharge load is applied, the highest and lowest resultant forces increase to 5.54 kN/m and 5.42 kN/m. The computed maximum horizontal loads are given on Table 7.32 below.

Table 7.31 Computed maximum and resultant horizontal forces on each reinforcement layer for 20% Sand backfill

Z (m)	1st Case kN/m	2nd Case kN/m	3rd Case kN/m	4th Case kN/m	5th Case kN/m	6th Case kN/m	7th Case kN/m	8th Case kN/m	9th Case kN/m	10th Case kN/m	11th Case kN/m	12th Case kN/m	13th Case kN/m
0	2.57	1.00	1.77	1.99	0.24	0.23	0.25	0.24	0.10	2.26	2.29	1.84	1.77
0.4	0.74	0.98	0.46	0.71	0.22	0.19	0.21	0.20	0.17	0.74	0.60	0.49	0.48
0.8	0.50	1.13	0.50	0.40	0.23	0.21	0.22	0.21	0.19	0.37	0.42	0.38	0.43
1.2	0.88	1.09	0.63	0.68	0.24	0.22	0.23	0.22	0.20	0.84	0.93	0.64	0.64
1.6	1.14	1.40	1.07	0.92	0.25	0.24	0.24	0.23	0.23	1.03	1.16	1.00	1.05
2.0	1.41	1.35	1.33	1.29	0.27	0.27	0.27	0.27	0.27	1.41	1.43	1.28	1.28
2.4	1.87	1.71	1.58	1.59	0.30	0.30	0.30	0.30	0.30	1.67	1.68	1.54	1.98
2.8	2.15	1.65	2.16	1.97	0.34	0.34	0.34	0.34	0.34	2.06	2.24	2.04	2.25
3.2	2.45	1.65	2.42	2.24	0.38	0.38	0.38	0.38	0.38	2.28	2.52	2.33	2.51
3.6	2.79	1.75	2.73	2.60	0.43	0.43	0.43	0.43	0.43	2.63	2.79	2.62	2.85
4.0	3.19	2.00	3.07	2.95	0.48	0.48	0.48	0.48	0.48	2.90	3.14	2.98	3.23
4.4	3.63	2.39	3.52	3.42	0.53	0.54	0.54	0.54	0.54	3.46	3.61	3.42	3.57
4.8	4.27	2.87	4.30	4.19	0.62	0.64	0.63	0.65	0.67	4.04	4.18	4.17	4.35
5.2	5.54	3.37	5.28	5.19	1.04	1.35	1.16	1.28	1.60	5.03	5.35	5.13	5.45
5.6	17.32	7.48	17.27	28.10	0.27	0.24	0.26	0.24	0.18	14.95	14.56	27.01	16.27
Resultant	50.46	31.81	48.10	58.25	5.82	6.06	5.94	6.00	6.10	45.69	46.90	56.88	48.12

Table 7.32 Computed maximum and resultant horizontal forces on each reinforcement layer for Clay backfill

Z (m)	1st Case $kN/_m$	2nd Case $kN/_m$	3rd Case $kN/_m$	4th Case $kN/_m$	5th Case $kN/_m$	6th Case $kN/_m$	7th Case $kN/_m$	8th Case $kN/_m$	9th Case $kN/_m$	10th Case $kN/_m$	11th Case $kN/_m$	12th Case $kN/_m$	13th Case $kN/_m$
0	2.42	0.22	1.29	1.89	0.25	0.23	0.24	0.24	0.19	1.63	1.91	1.69	1.53
0.4	0.51	0.97	0.53	0.55	0.21	0.19	0.21	0.20	0.17	0.38	0.56	0.67	0.44
0.8	0.45	0.94	0.36	0.46	0.21	0.21	0.22	0.20	0.19	0.46	0.35	0.37	0.36
1.2	0.54	0.90	0.66	0.49	0.22	0.21	0.23	0.21	0.19	0.48	0.64	0.51	0.50
1.6	0.81	1.41	0.88	0.77	0.23	0.23	0.24	0.22	0.22	0.81	0.86	0.77	0.78
2.0	0.98	1.32	1.13	1.00	0.25	0.25	0.25	0.25	0.25	0.99	1.06	1.01	1.31
2.4	1.25	1.24	1.35	1.22	0.28	0.28	0.28	0.28	0.28	1.17	1.54	1.23	1.53
2.8	1.45	1.20	1.78	1.61	0.32	0.32	0.32	0.32	0.32	1.59	1.77	1.68	1.76
3.2	1.70	1.20	2.04	1.86	0.36	0.36	0.35	0.36	0.36	1.79	2.01	1.92	2.01
3.6	1.98	1.29	2.36	2.15	0.40	0.40	0.40	0.40	0.40	2.01	2.32	2.22	2.33
4.0	2.34	1.48	2.68	2.46	0.45	0.45	0.44	0.45	0.45	2.28	2.62	2.54	2.68
4.4	2.85	1.78	3.07	2.90	0.50	0.50	0.50	0.50	0.50	2.64	2.94	2.93	3.06
4.8	3.52	2.14	3.76	3.60	0.58	0.59	0.58	0.60	0.61	3.26	3.54	3.61	3.83
5.2	4.25	2.60	4.71	4.34	0.92	1.10	0.90	1.05	1.29	3.97	4.32	4.31	4.70
5.6	15.95	6.82	14.91	25.89	0.27	0.24	0.26	0.26	0.12	14.99	13.00	25.26	14.79
Resultant	41.00	25.53	41.50	51.22	5.45	5.55	5.42	5.54	5.54	38.44	39.43	50.71	41.62

8 CONCLUSION

Performance of reinforced earth walls evaluated with tyre crumbs and foundation conditions. Different type of backfills are considered in order to evaluate their effects to performance. Different backfills are derived from mixing sand and clay at different proportions. Standard proctor tests are performed in order to determine their maximum unit weight and corresponding optimum water content. After that, direct shear tests are conducted to determine shear strength parameters of the soils. The following conclusions can be made according to results of direct shear tests.

- Maximum dry unit weight increases up to threshold value of clay content. Maximum dry unit weight then starts to decrease after that threshold value.
- Optimum water content decreases until threshold value of clay content. After threshold, required water content to achieve maximum dry unit weight increases.
- Shear strength increases as both clay content and normal stress increases. Shear strength increase becomes more pronounced in case of lower normal stress.
- Dilative behaviour is observed after initial contraction of samples. Contraction of samples is smaller than their expansion.
- Angle of friction increases and cohesion decreases as sand content increases. Constant value of cohesion observed at first sample when shearing rate is decreased.

Direct shear tests are modelled in finite element model in 3D using software Abaqus. Mohr – Coulomb material model is used. Shear tests modelled in two steps. Vertical load applied at first stage and shear displacement applied in second step. Results of finite element modelling can be summarized as follows.

- The peak stresses measured at the laboratory and calculated by finite element method are quite compatible with each other. The maximum deviation is found as 11.26% and the minimum deviation is found to be 0.16% between peak shear stresses.
- Shear stress – horizontal displacement graphics agreed well for sand samples. As clay content increased, agreement between experimental graphics and finite element graphics are broken.
- Computed angle of frictions from finite element analysis complies well with experimentally found angle of friction. Computed cohesion also complies with experimentally found cohesion when clay content is high in sample.
- The highest shear stress distribution inside sample depends on the amount of the displacement exerted over sample. Failure wedge is formed starting from failure plane and propagated through bottom part of the sample.

In order to determine effect of tyre crumb into behaviors of earth walls, laboratory tests are conducted with sand backfill and clay backfill. Walls are designed with a height of 45 cm and reinforcement length equals to 0.7H. Retaining walls designed with respect to FHWA method. Test results can be given as follows.

- Resisting moment and sliding resistance forces calculated are slightly higher for reinforced earth wall with clay backfill.
- Forces on reinforcements are computed to be higher for sand backfill. In case of clay backfill, computed horizontal forces are negative due to cohesion.
- Pull – out capacity is significantly higher in case of sand backfill.
- Measured settlements of loading plate increases as the tyre crumb content increases.
- Horizontal displacements are almost equal to without tyre crumb case and 10% tyre crumb content when sand backfill is considered. This behaviour is valid until some load threshold. After this threshold, horizontal displacements are higher for 10% tyre crumb content. Higher horizontal displacements are computed for 20% and 30% tyre crumb content.
- Settlement of loading plate and horizontal displacement increase as tyre crumb content increases in case of clay backfill.
- Failure load decreases as the tyre crumb content increases both for sand and clay backfills.
- Settlements and horizontal displacements are smaller for sand backfill. Similar behaviour is valid when same amount of tyre crumb is added to sand and clay.

Laboratory tests without tyre crumb contents modelled on finite element model to conduct parametric study. Plaxis software is used in this stage. Effect of reinforcement stiffness, reinforcement density and reinforcement length are considered.Following results are obtained from finite element study. Parametric study is conducted only for sand backfill.

- Computed settlements of loading plate decrease as the stiffness of reinforcement increases. The change is more obvious in case of sand backfill.
- Horizontal displacements decrease as reinforcement stiffness increases. Decrement is more visible at higher points of reinforced earth wall. Decrement is more pronounced when reinforcement stiffness increases from 1048 kN/m to 2096 kN/m.
- Calculated maximum horizontal forces and resultant maximum horizontal force increase as reinforcement stiffness increases.
- As higher number of reinforcements are used, settlement of loading plate decreases.
- Horizontal displacements significantly decrease when reinforcement density increases from less dense case to medium dense case. However, as the reinforcement density increase further, decrement becomes insignificant.
- As reinforcement density decreases, calculated maximum forces on each reinforcement layer increase. However, smaller change is observed for resultant maximum horizontal forces.
- Settlement of loading plate insignificantly effected from reinforcement length.

- Horizontal displacements tend to increase at top and middle of wall as reinforcement length increases. Horizontal displacements initially increase and then decrease at the bottom of the reinforced earth wall.
- Maximum horizontal forces and resultant maximum horizontal force slightly increase as length of reinforcement increase.
- Factor of safety against slope stability increases when reinforcement length, reinforcement stiffness and density increases while, factor of safety decreases as the magnitude of load increases.

In order to determine effect of type of backfill and foundation, parametric study is conducted on Plaxis software. Thicknesses and depths are changed as well as their strength parameters changed. Results are expressed as below.

- If different type of backfill is used, the lowest horizontal displacements are computed at wall face for reinforced earth wall with clay. The lowest displacements of retained soil are computed when sand backfill is used. The highest maximum horizontal forces are computed for sand. It is also seen that, as cohesion increases in backfill, computed horizontal maximum force decreases.
- Two type of behaviour is observed in general with respect to foundation conditions. In first type of behaviour, reinforced earth walls tend to move forward, while in second kind of behaviour, reinforced earth walls tend to lean back. Second type of behaviour is observed when strength parameters of first foundation layer is increased. Lower horizontal displacements, lower settlements and lower maximum horizontal forces are computed for second type of behaviour. Settlement pattern of the wall changes as well.
- When thickness of 1st foundation layer is increased, higher horizontal deformations are computed at the end of construction. However, when surcharge load is applied, higher horizontal deformations are computed when 1st foundation layer has lower thickness for sand backfill. The opposite behaviour is observed for other backfill containing sand. In case of clay backfill, higher displacements are computed for low thickness of foundation layer 1. Same behaviour is observed in case of horizontal displacement of retained soil. Differences between horizontal displacements are more pronounced when surcharge load is not exerted. Settlement of foundation soil increases for all cases except for 80% sand backfill when thickness of 1st foundation soil layer increases. Maximum horizontal forces are insignificantly affected from thickness change of 1st foundation soil.
- Horizontal displacements computed at wall face and retained soil decreases significantly for all types of soil. Settlement of wall decreases enormously and tends to remain constant along the wall. Similarly, lower forces are computed when strength parameters of 1st foundation layer increases.
- When thickness and strength parameters of 1st foundation layer increases at the same time, lower horizontal displacements, settlements and maximum horizontal forces are computed for all types of backfills.

- If thickness of 2nd foundation layer is decreased 50%, higher displacement on reinforced earth wall face increases slightly when there is not surcharge load for sand backfill. Displacements become almost equal when surcharge load is applied in case of sand backfill. Lower horizontal displacements are computed at reinforced earth wall's face with and without surcharge load for 80% sand, 60% sand, 40% sand, 20% sand content backfill when thickness of 2nd foundation layer decreased to half of its initial value. In case of clay backfill, lower displacements are computed when surcharge load is not applied, however when surcharge load is applied, higher displacements are computed. Displacements computed for retained soil show same behaviour with displacements of reinforced earth wall face. Settlement of earth wall increases if thickness of 2nd foundation layer decreases. Maximum horizontal forces tend to decrease when thickness of 2nd foundation layer decreases. This tendency disappears when surcharge load is exerted.
- When strength of 2nd foundation layer is decreased following differences are computed. Computed horizontal displacements increase when surcharge load is not exerted. However horizontal displacements computed smaller after application of surcharge load than reference case at reinforced earth wall face for sand backfill. Lower displacements at reinforced earth wall face are computed for other backfills except for 20% sand backfill under surcharge load. Equal horizontal displacements are computed for 20% sand backfill under surcharge load. Similar behaviors are observed in case of computed displacements of retained backfill for all backfill types. Computed settlements increase for each case except for 80% sand backfill. Settlements are found to be increased for 80% sand backfill. Computed maximum horizontal forces tend to decrease especially at deeper layers, however after application of surcharge load, computed maximum horizontal forces do not change.
- When thickness and strength of 2nd foundation layer is decreased at the same time, horizontal displacement of wall face increase for sand backfill independent of surcharge load. In case of other backfills, horizontal displacements decrease without surcharge load, while horizontal displacements increase under surcharge load. Horizontal displacements of retained soil show similar behaviour except for 80% sand. In case of 80% sand backfill, horizontal displacements of retained soil increase for both without surcharge and with surcharge load. Settlements of walls increase with respect to change on 2nd foundation conditions except for 80% sand backfill. In case of 80% sand backfill, settlement of wall decreases. Maximum horizontal forces slightly decrease due to change of properties of 2nd foundation layer.

If strength and thickness of 1st foundation layer increase and thickness of 2nd foundation layer decrease, following conclusions can be made.

- Horizontal displacements of reinforced earth wall face and retained soil significantly decrease. Change of horizontal displacements with respect to height also changes from linear behaviour to curvature behaviour. Degree of curvature depends on considered condition.
- Computed settlements significantly reduce. Computed settlements become constant or slightly increase along the wall length depending on foundation condition.
- Computed maximum horizontal forces decrease as the foundation conditions are changed to above conditions.

When the thickness of 1st foundation layer increase and strength and thickness of 2nd foundation layer decrease, following conclusions can be made.

- Horizontal displacements computed at reinforced earth wall face increase for sand backfill and decrease for remaining backfills when there is no surcharge load. When surcharge load is applied horizontal displacements decreases except for clay backfill which it remains the same. Horizontal displacements of retained soil follows the behaviour of wall face.
- Settlement of walls increases under without surcharge load conditions except for 80% sand backfill type. When surcharge load is applied, higher settlements are also observed for 60% sand backfill, 40% sand backfill, 20% sand backfill and clay backfill.
- Computed forces on reinforcement decrease until surcharge load is applied for all type of backfills. However, when surcharge load is applied, computed maximum horizontal forces increase for 80% sand and 60% sand, remain constant for 40% sand and clay backfills.

When thickness of 1st foundation layer increase and strength of 2nd foundation layer decrease, following conclusions can be made.

- Horizontal displacement of sand increases while, it remains constant for 80% sand, 40% sand and 20% sand backfill. Horizontal displacements decrease for remaining backfills. These are valid when surcharge load is not applied. When surcharge load is applied, lower deformations are computed for sand, 80% sand, 20% sand and clay backfill. Horizontal displacements of 60% backfill remain constant and increase 40% sand backfill.
- Horizontal displacements of retained soil increase for sand and 40% sand backfill while remain constant for 80% sand, 20% sand and clay backfill. Horizontal displacement of remaining backfills decreased when surcharge load is not applied. When surcharge load is applied, lower displacements are computed for sand, 80% sand and clay backfill.
- Settlement of walls with sand, 40% sand, 20% sand and clay backfill increase independent of surcharge load. Settlement of wall with 80% sand decreases independent of surcharge load. In case of 60% backfill, settlement computed

lower under wall face while it is computed as higher at the other edge when surcharge load is not applied. When surcharge load is applied, higher settlements are computed for this type of backfill too.

- Computed maximum horizontal forces decreases for all type of backfills when surcharge load is not applied. When surcharge load is applied, maximum horizontal forces decreases for sand backfill, remains constant for 80% sand backfill and increases for remaining backfills.

According to results of this study, tyre crumbs may be used when their content is below 10%. Higher contents may be used at higher depths. However, behaviour of those kinds of walls should be investigated.

It is also clear that, behaviour of reinforced earth wall is highly influenced from conditions of foundation layer or layers. Change of behaviour of reinforced earth wall with respect to foundation conditions are not presented in design codes. Further studies should be conducted to incorporate foundation conditions to design of reinforced earth wall.

It may be said that, following contributions are made from the results of this study.

- Tyre crumbs can be used as a backfill material with sand up to 10% tyre crumb content. Several researchers found contradicting results about usage of tyre chips in reinforced earth wall, however, experimental part of study proved that, tyre crumbs can be used.
- Effect of backfill materials are generally considered by working conditions of reinforced earth wall. This study proved that, not only working conditions, but also change of working conditions of reinforced earth wall should be considered during design of reinforced earth walls, especially for the walls which contains clay particles.
- Foundation conditions are important property of the design stage. Changing foundation conditions may yield to totally different behaviour of the wall. Amount of change is revealed by this study.

The outcome of this study can be used for a further research in the following areas.

- Investigation of decreasing settlement of loading plate when tyre crumbs are used with sand backfill.
- Implementing a coefficient to analytical design of reinforced earth walls in order to account foundation conditions. It is clear that, checking for a bearing capacity of foundation is not enough to design a reinforced earth wall.

REFERENCES

1 - E. Bourgeois, L. Soyez, A. Le Kouby “Experimental and numerical study of the behaviour of a reinforced-earth wall subjected to a local load" Computers and Geotechnics, 2011, 38, 515-525

2 – Suliman B.A. Mohamed, Kuo-Hsin Yang, Wen-Yi Hung “Finite element analyses of two-tier geosynthetic-reinforced soil walls: Comparison involving centrifuge tests and limit equilibrium results”, Computers and Geotechnics 2014, 61, 67 – 84

3 – Myoung – Soo Won, You – Seong Kim “Internal deformation behavior of geosynthetic-reinforced soil walls” Geotextiles and Geomembranes, 2007, 25, 10 – 22

4 – Guangqing Yang, Baojian Zhang, Peng Lv, Qiaoyong Zhou “Behaviour of geogrid reinforced soil retaining wall with concrete-rigid facing”, Geotextiles and Geomembranes, 2009, 27, 350 – 356

5 – Ching – Chuan Huang, Woei – Ming Luo "Behavior of cantilever and geosynthetic-reinforced walls on deformable foundations" Geotextiles and Geomembranes, 2010, 28, 448 – 459

6 – Eder C.G. Santos, Ennio M. Palmeira, Richard J. Bathurst "Behaviour of a geogrid reinforced wall built with recycled constructionand demolition waste backfill on a collapsible foundation", Geotextiles and Geomembranes, 2013, 39, 9 –19

7 – Chengzhi Xiao, Jie Han, Zhen Zhang "Experimental study on performance of geosynthetic-reinforced soil model walls on rigid foundations subjected to static footing loading", Geotextiles and Geomembranes, 2016, 44, 81 – 94

8 – M.Ehrlich, S.H. Mirmoradi , R.P. Saramago "Evaluation of the effect of compaction on the behavior of geosynthetic-reinforced soil walls", Geotextiles and Geomembranes, 2012, 34, 108 – 115

9 – Chungsik Yoo, Hyuck – Sang Jung "Measured behavior of a geosynthetic-reinforced segmental retaining wall in a tiered configuration", Geotextiles and Geomembranes, 2004, 22, 359 – 376

10 – Taesoon Parka, Siew Ann Tan "Enhanced performance of reinforced soil walls by the inclusion of short fiber" Geotextiles and Geomembranes, 2005, 23, 348-361

11 – Mario Riccio, Mauricio Ehrlich, Daniel Dias "Field monitoring and analyses of the response of a block-faced geogrid wall using fine-grained tropical soils", Geotextiles and Geomembranes, 2014, 42, 127 – 138

12 – Chungsik Yoo, Sun – Bin Kim "Performance of a two-tier geosynthetic reinforced segmental retaining wall under a surcharge load: Full-scale load test and 3D finite element analysis", Geotextiles and Geomembranes, 2008, 26, 460 – 472

13 – Carina Maia Lins Costa, Jorge Gabriel Zornberg, Benedito de Souza Bueno Yuri Daniel Jatob a Costa "Centrifuge evaluation of the time-dependent behavior of geotextile-reinforced soil walls", Geotextiles and Geomembranes, 2016, 44, 188 – 2007

14 – A. L. Shinde Æ J. N. Mandal "Behavior of Reinforced Soil Retaining Wall With Limited Fill Zone", Geotech Geol Eng, 2007, 25, 657 – 672

15 – Sutapa Hazra Æ Nihar Ranjan Patra "Performance of Counterfort Walls with Reinforced Granular and Fly Ash Backfills: Experimental Investigation", Geotech Geol Eng, 2008, 26, 259 – 267

16 – M. Pinho-Lopes, D. M. Carlos, M. L. Lopes "Flume Tests on Fine Soil Reinforced with Geosynthetics: Walls of the Salt Pans (Aveiro Lagoon, Portugal)", Int. J. of Geosynth. and Ground Eng, 2015, 1 – 12

17 – H. Ahmadi, M. Hajialilue-Bonab "Experimental and analytical investigations on bearing capacity of strip footing in reinforced sand backfills and flexible retaining wall", Acta Geotechnica, 2012, 7, 357 – 373

18 – S. Bali Reddy, A. Murali Krishna "Recycled Tyre Chips Mixed with Sand as Lightweight Backfill Material in Retaining Wall Applications: An Experimental Investigation", Int. J. of Geosynth. and Ground Eng, 2015, 1 – 31

19 – Richard J. Bathurst, Sebastian Althoff, and Peter Linnenbaum "Influence of Test Method on Direct Shear Behavior of Segmental Retaining Wall Units", Geotechnical Testing Journal, 31, 1 – 9

20 – Guangqing Yang, Huabei Liu, Peng Lv, Baojian Zhang "Geogrid-reinforced lime-treated cohesive soil retaining wall: Case study and implications", Geotextiles and Geomembranes, 2012, 35, 112 – 118

21 – Guang – Qing Yang, Huabei Liu, Yi-Tao Zhou, Bao – Lin Xiong "Post-construction performance of a two – tiered geogrid reinforced soil wall backfilled with soil – rock mixture", Geotextiles and Geomembranes, 2014, 42, 91 – 97

22 – Sompote Youwai, Dennes T. Bergado "Numerical analysis of reinforced wall using rubber tyre chips–sand mixtures as backfill material", Computers and Geotechnics, 2004, 31, 103 – 114

23 – Chia – Cheng Fan "Three-dimensional behaviour of a reinforced earth-retaining structure within a valley", Computers and Geotechnics, 2006, 33, 69 – 85

24 – Jian – Feng Chen, Jun – Xiu Liu, Jian – Feng Xue, Zhen – Ming Shi "Stability analyses of a reinforced soil wall on soft soils using strength reduction method", Engineering Geology, 2014, 177, 83 – 92

25 – M. Ehrlich, S.H. Mirmoradi "Evaluation of the effects of facing stiffness and toe resistance on the behavior of GRS walls", Geotextiles and Geomembranes, 2013, 40, 28 – 36

26 – Dov Leshchinsky, Farshid Vahedifard, Ben A. Leshchinsky "Revisiting bearing capacity analysis of MSE walls", Geotextiles and Geomembranes, 2012, 34, 100 – 107

27 – Huabei Liu "Long-term lateral displacement of geosynthetic-reinforced soil segmental retaining walls", Geotextiles and Geomembranes, 2012, 32, 18 – 27

28 – Yan Yu, Ivan P. Damians, Richard J. Bathurst "Influence of choice of FLAC and PLAXIS interface models on reinforced soil–structure interactions", Computers and Geotechnics, 2015, 65, 164 – 174

29 – Huabei Liu, Xiangyu Wang, Erxiang Song "Long-term behavior of GRS retaining walls with marginal backfill soils", Geotextiles and Geomembranes, 2009, 27, 295 – 307

30 – Jie Han, Dov Leshchinsky "Analysis of back-to-back mechanically stabilized earth walls" , Geotextiles and Geomembranes, 2010, 28, 262 – 267

31 – Abdelkader Abdelouhab, Daniel Dias, Nicolas Freitag "Numerical analysis of the behaviour of mechanically stabilized earth walls reinforced with different types of strips", Geotextiles and Geomembranes, 2011, 29, 116 – 129

32 – Hoe I. Ling Æ, Huabei Liu "Deformation analysis of reinforced soil retaining walls—simplistic versus sophisticated finite element analyses", Acta Geotechnica, 2009, 4, 203 – 2013

33 – Yonggui Xie, Ben Leshchinsky “MSE walls as bridge abutments: Optimal reinforcement density”, Geotextiles and Geomembranes, 2015, 43, 128 – 138

34 – I. P. Damians, R. J. Bathurst, A. Josa and A. Lloret "Numerical study of the influence of foundation compressibility and reinforcement stiffness on the behavior of reinforced soil walls", International Journal of Geotechnical Engineering", 2014, 8, 247 – 259

35 - Lazhar Belabed, Hacene Benyaghla,Jarir Yahiaoui "Internal Stability Analysis of Reinforced Earth Retaining Walls", Geotech Geol Eng, 2011, 29, 443 – 452

36 – Ben Leshchinsky "Limit Analysis Optimization of Design Factors for Mechanically Stabilized Earth Wall-Supported Footings", Transp. Infrastruct. Geotech., 2014, 1, 111 – 128

37 – XUE Jian – Feng, CHEN Jian – Feng "Reinforcement strength reduction in FEM for mechanically stabilized earth structures" , J. Cent. South Univ., 2015, 22, 2691 – 2698

38 – Graeme D. Skinnera, R. Kerry Roweb "Design and behaviour of a geosynthetic reinforced retaining wall and bridge abutment on a yielding foundation", Geotextiles and Geomembranes, 2005, 23, 234 – 260

39 – Jie Huang, Robert L. Parsons, Jie Han, Matthew Pierson “Numerical analysis of a laterally loaded shaft constructed within an MSE wall”, Geotextiles and Geomembranes, 2011, 29, 233 – 241

40 – Jie Huang, Jie Han, Robert L. Parsons, Matthew C. Pierson "Refined numerical modeling of a laterally-loaded drilled shaft in an MSE wall", Geotextiles and Geomembranes, 2013, 37, 61 – 73

41 – Guangqing Yang, Junxia Ding, Qiaoyong Zhou, and Baojian Zhang "Field Behavior of a Geogrid Reinforced Soil Retaining Wall with a Wrap-Around Facing" Geotechnical Testing Journal, 2010, 33, 1 – 6

42 – Krystyna Kazimierowicz-Frankowska "A case study of a geosynthetic reinforced wall with wrap-around facing", Geotextiles and Geomembranes, 2005, 23, 107 – 115

43 – Jian – Feng Xue, Jian – Feng Chen, Jun – Xiu Liu, Zhen – Ming Shi "Instability of a geogrid reinforced soil wall on thick soft Shanghai clay with prefabricated vertical drains: A case study", Geotextiles and Geomembranes, 2014, 42, 302 – 311

44 – Abdolhosein Haddad and Gholamali Shafabakhsh "FAILURE OF SEGMENTAL RETAINING WALLS DUE TO THE INSUFFICIENCY OF BACKFILL PERMEABILITY", Proceedings of the 4th Asian Regional Conference on Geosynthetics, June 17 - 20, 2008 852 – 856

45 – Chungsik Yoo "Performance of a 6-year-old geosynthetic reinforced segmental retaining wall", Geotextiles and Geomembranes, 2004, 22, 377 – 397

46 – R. Kerry Rowe and Allen Lunzhu Li "INSIGHTS FROM CASE HISTORIES: REINFORCED EMBANKMENTS AND RETAINING WALLS"

47 – James G. Collin "Lessons learned from a segmental retaining wall failure", Geotextiles and Geomembranes, 2001, 19, 445 – 454

48 – A. Sengupta "Numerical Study of a Failure of a Reinforced Earth Retaining Wall", Geotech Geol Eng, 2012, 30, 1025 – 1034

49 – Assaf Klar, Tal Sas "The KC method: Numerical investigation of a new analysis method for reinforced soil walls", Computers and Geotechnics, 2010, 37, 351 – 358

50 – Thai Son Quang, Hassen Ghazi, Buhan de Patrick "A multiphase approach to the stability analysis of reinforced earth structures accounting for a soil–strip failure condition", Computers and Geotechnics, 2009, 36, 454 – 462

51 – Suliman B.A. Mohamed, Kuo-Hsin Yang, Wen-Yi Hung "Limit equilibrium analyses of geosynthetic-reinforced two-tiered walls: Calibration from centrifuge tests", Geotextiles and Geomembranes, 2013, 41, 1 – 16

52 – V. A. Barvashov and I. M. Iovlev "METHOD OF ANALYSIS FOR REINFORCED SOIL MASSES", Soil Mechanics and Foundation Engineering, 2010, 47, 189 – 196

53 – Jonathan T. H. Wu, Jean-Baptiste Payeur "Connection Stability Analysis of Segmental Geosynthetic Reinforced Soil (GRS) Walls", Transp. Infrastruct. Geotech., 2015, 2, 1 – 17

54 – SATYENDRA MITTAL, K. G. GARG and SWAMI SARAN "Analysis and design of retaining wall having reinforced cohesive frictional backfill" Geotechnical and Geological Engineering, 2006, 24, 499 – 522

55 – Richard J. Bathursta, Tony M. Allenb, Dave L. Waltersc"Reinforcement loads in geosynthetic walls and the case for a new working stress design method", Geotextiles and Geomembranes, 2005, 23, 287 – 322

56 - Dov Leshchinsky, Beongjoon Kang, Jie Han, Hoe Ling "Framework for Limit State Design of Geosynthetic-Reinforced Walls and Slopes", Transp. Infrastruct. Geotech., 2014, 1, 129 – 164

57 - R. Baker, Y. Klein "An integrated limiting equilibrium approach for design of reinforced soil retaining structures: Part I—formulation", Geotextiles and Geomembranes, 2004, 22, 119 – 150

58 – D. M. Carlos, Margarida Pinho-Lopes "Reinforcement with Geosynthetics of Walls of the Saltpans of the Aveiro Lagoon", Geotech Geol Eng, 2011, 29, 519 – 536

59 – R. Baker, Y. Klein "An integrated limiting equilibrium approach for design of reinforced soil retaining structures: Part II—design examples", Geotextiles and Geomembranes, 2004, 22, 151 – 177

60 – Dov Leshchinskya, Yuhui Hua, Jie Hanb "Limited reinforced space in segmental retaining walls", Geotextiles and Geomembranes, 2004, 22, 543 – 553

61 – B. Munwar Basha, G. L. Sivakumar Babu "Reliability Based LRFD Approach for External Stability of Reinforced Soil Walls" Indian Geotech J, 2013, 43 (4) 292 – 302

62 – Ömer Bilgin "Failure mechanisms governing reinforcement length of geogrid reinforced soil retaining walls", Engineering Structures, 2009, 31, 1967 – 1975

63 – O. Al Hattamleha, B. Muhunthanb "Numerical procedures for deformation calculations in the reinforced soil walls", Geotextiles and Geomembranes 2006, 24, 52 – 57

64 – Robert M. Koerner and George R. Koerner "THE IMPORTANCE OF DRAINAGE CONTROL FOR GEOSYNTHETIC REINFORCED MECHANICALLY STABILIZED EARTH WALLS", Journal of GeoEngineering, 2011, 6, 3 – 13

65 – Robert M. Koerner, Te-Yang Soong "Geosynthetic reinforced segmental retaining walls", Geotextiles and Geomembranes, 2001, 19, 359 – 386

66 – P.K. Basudhar, Amol Vashistha, Kousik Deb, Arindam Dey "Cost Optimization of Reinforced Earth Walls", Geotech Geol Eng, 2008, 26, 1 – 12

67 – F. Tatsuokaa, M. Tateyamab, Y. Mohric, K. Matsushimac "Remedial treatment of soil structures using geosynthetic – reinforcing technology", Geotextiles and Geomembranes, 2007, 25, 204 – 220

68 - Sajna Sayed , G.R. Dodagoudar, K. Rajagopal "Finite element reliability analysis of reinforced retaining walls" Geomechanics and Geoengineering, 2010, 5, 3, 187-197

69 – Kalehiwot Nega Manahiloh, Mohammad Motalleb Nejad, Mohammad Sadegh Momeni "Optimization of Design Parameters and Cost of Geosynthetic – Reinforced Earth Walls Using Harmony Search Algorithm", Int. J. of Geosynth. and Ground Eng, 2015, 1 – 15

70 – Design and Construction of Mechanically Stabilized Earth Walls and Reinforced Slopes, FHWA-NHI-10-024 FHWA GEC 011, 2009.

71 – CSN CEN ISO/TS 17892-10 Geotechnicky pruzkum a zkouseni – Laboratorni zkousky zemin Cast 10: Krabicova smykova zkouska

72 – ASTM 4318-10 (2010) Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils

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- Balaban Eren,Onur Mehmet İnanç (2018). Comparison of behaviour of basal reinforced piled embankment with two layer of reinforcement. Geomechanics and Engineering, 16(3), 233-245., Doi: 10.12989/gae.2018.16.3.233
- Balaban Eren,Onur Mehmet İnanç (2018). A Numerical Model on Geosynthetic Reinforced Pile Supported Embankments. Anadolu University Journal of Science and Technology, 19(2), 484-498., Doi: 10.18038/aubtda.376144
- Eren Balaban, Ales Smejda, Mehmet İnanç Onur. Influence of Tire Crumbs on Mechanical Properties of Sand-Fine Soil Mixtures, Geomechanics and Geoengineering: An International Journal (Under Review)
- Eren Balaban, Mehmet İnanç Onur, Ales Smejda (2019). Shear Strength Behaviour and Finite Element Modelling of Soil Mixtures Under Low Confining Pressure. 3rd International Conference on Information Technology in Geo-Engineering (Accepted)
- Balaban Eren,Smejda Ales,Onur Mehmet İnanç (2019). An Experimental Study on Shear Strength Behavior of Soils Under Low Confining Pressure. 4th World Congress on Civil, Structural, and Environmental Engineering (CSEE'19), Doi: 10.11159/icgre19.110
- Balaban Eren,Smejda Ales,Onur Mehmet İnanç (2019). Performance Analysis of Reinforced Earth Walls with Different Soils. International Disaster and Resilience Congress
- Balaban Eren,Smejda Ales,Onur Mehmet İnanç (2018). MSE Wall Design for Bridge Abutment by Determination of Critical Parameters. VII. International Scientific Conference of the Faculty of Transport Engineering (ISC2018), 40-56.
- Bozkurt Fatih, Yurdakul Özgür,Hojka Premysl,Routil Ladislav,Balaban Eren,Artagan Salih Serkan (2018). Mechanical Properties of Structural Steel Exposed to Passive Fire. 27th International Conference on Metallurgy and Materials - METAL 2018
- Balaban, E. Onur, M. İ. Smejda, A. Investigation of Stresses Acting Over Basal Reinforced Piled Embankments. 20th International Conference of Doctoral Students (Juniorstav 2018)
- Balaban, E. Dağdeviren, U. Onur, M. İ. Tuncan, M. Tuncan, A. 2013. Investigation of Consolidation Behaviour of Saturated Clays Under Different Loading Conditions. 5th Geotechnical Symposium, Adana

- Balaban, E. Çelik, A. O. Kılınç, K. Tuncan, M. Tuncan, A. 2012. Investigation of the Systematic Compressive Strength Variation within Concrete Elements Using Small Diameter Cores. 18th İbausil, Weimar – Almanya

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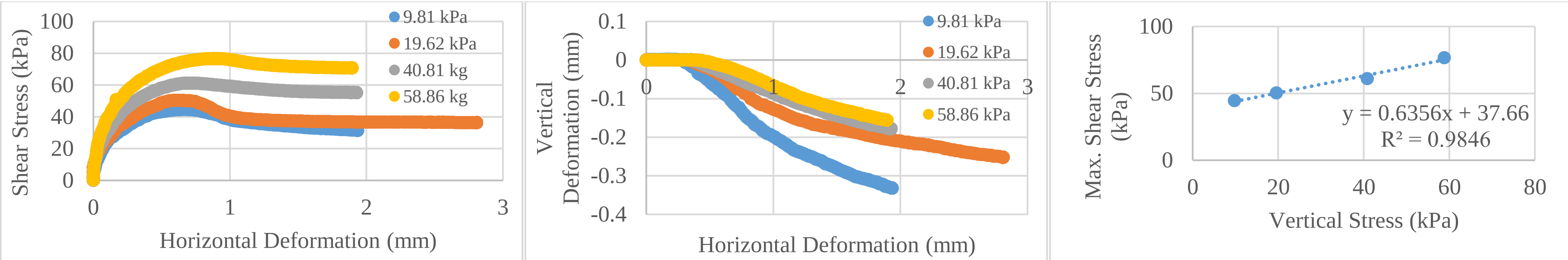
ATTACHMENTS

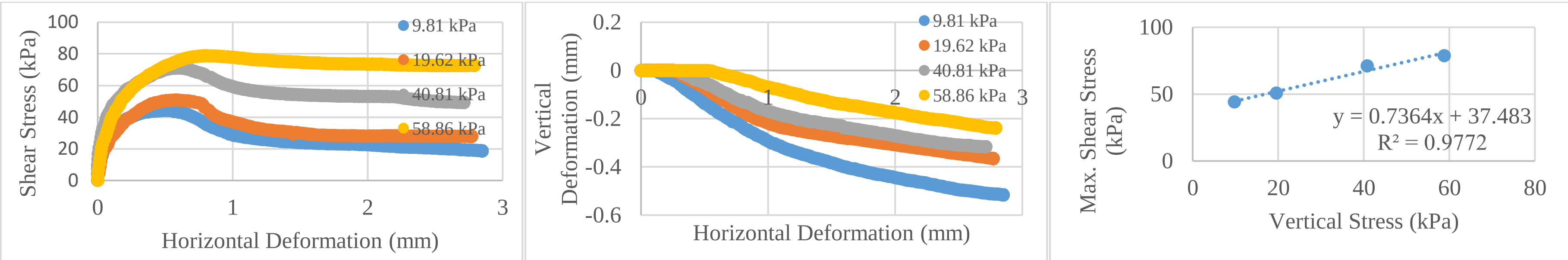
Attachment 1: Graphics from Direct Shear Tests

Without chip tyre content

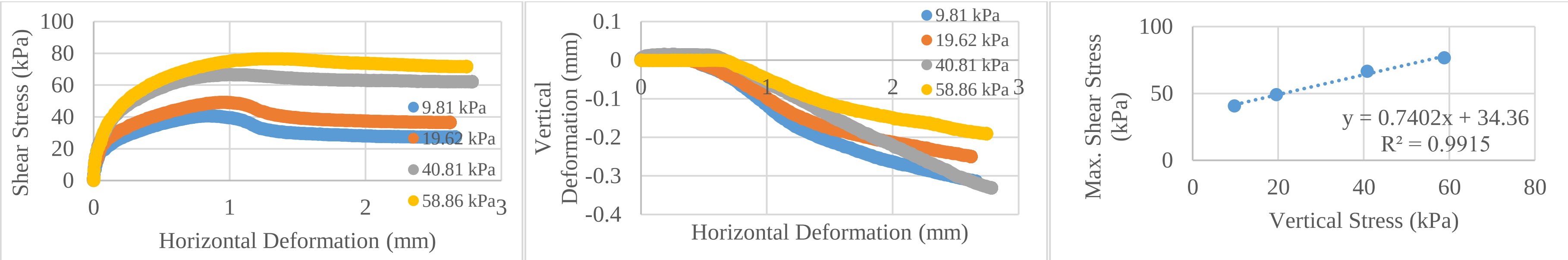
Clay



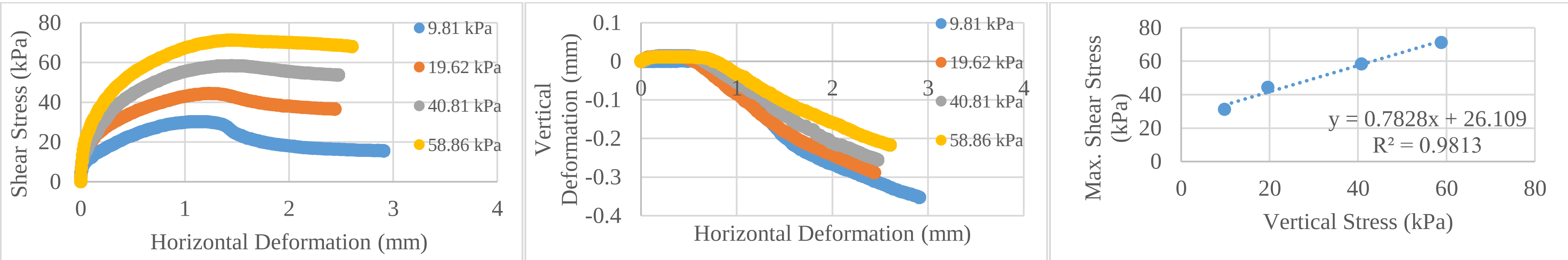
90% Clay + 10% Sand



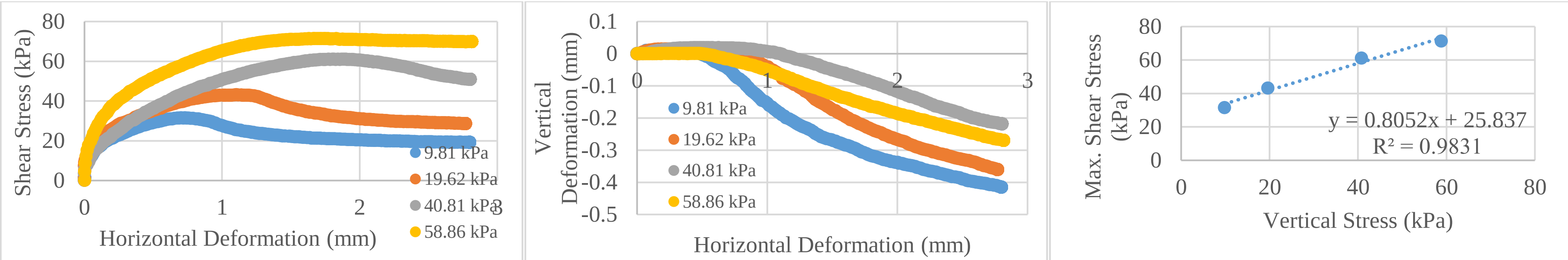
80% Clay + 20% Sand



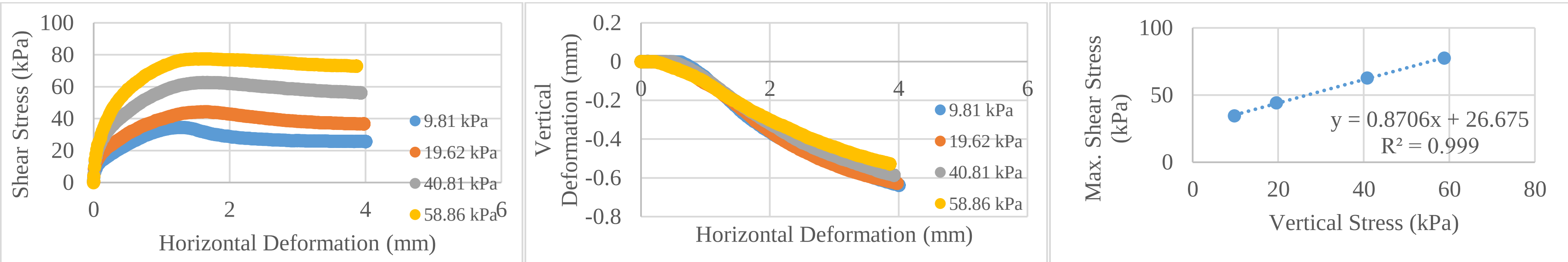
70% Clay + 30% Sand



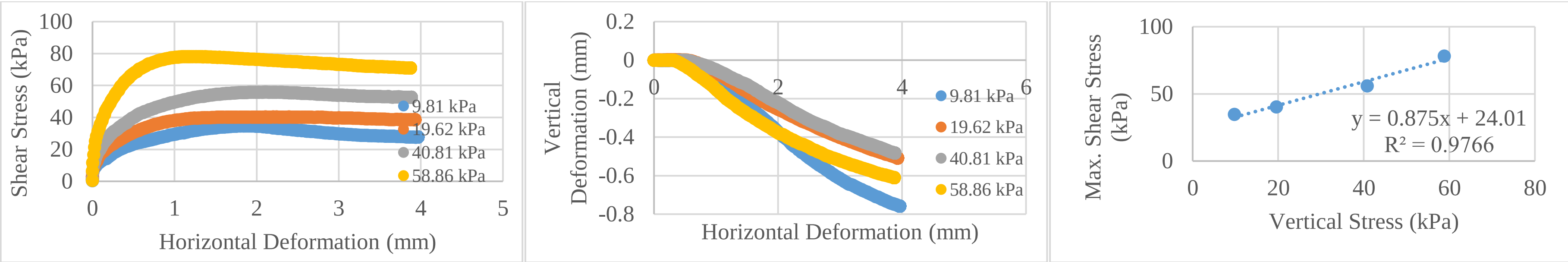
60% Clay + 40% Sand



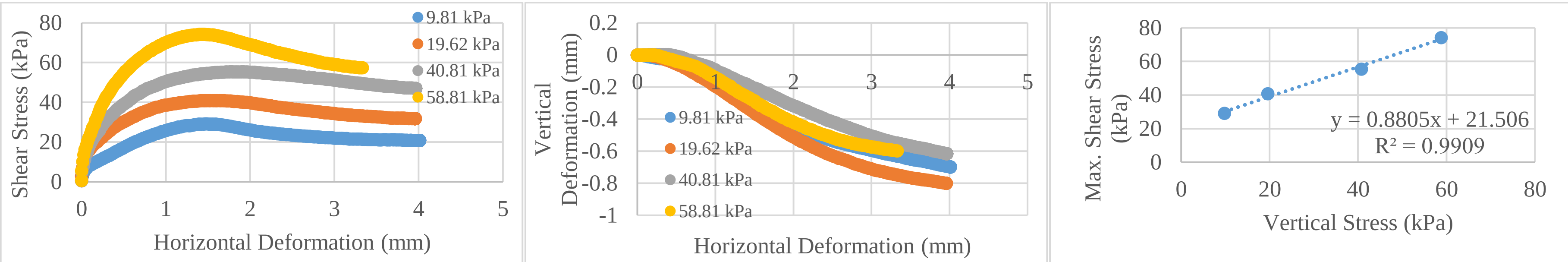
50% Clay + 50% Sand



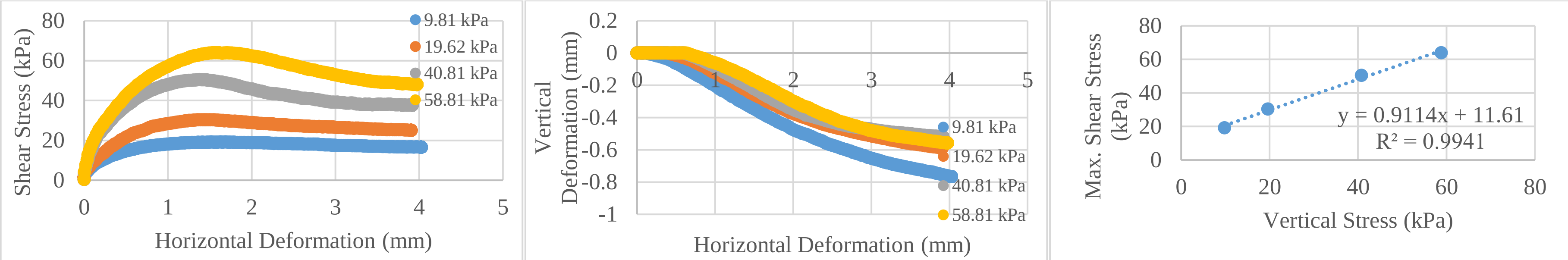
40% Clay + 60% Sand



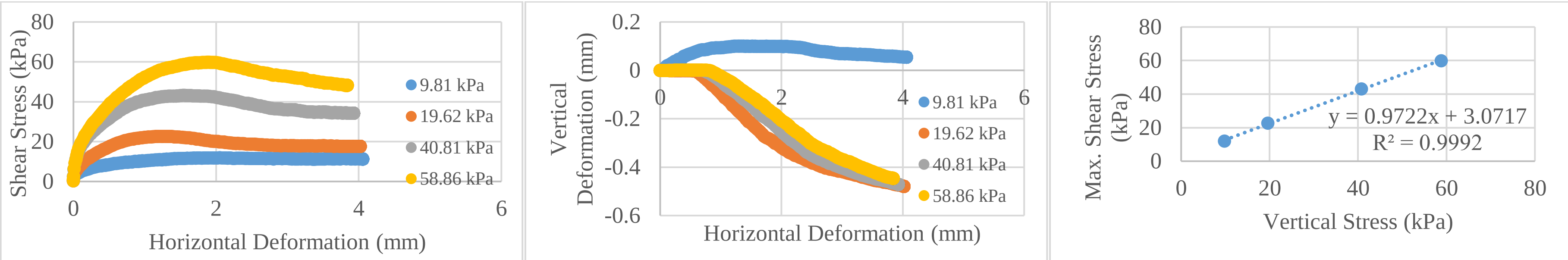
30% Clay + 70% Sand



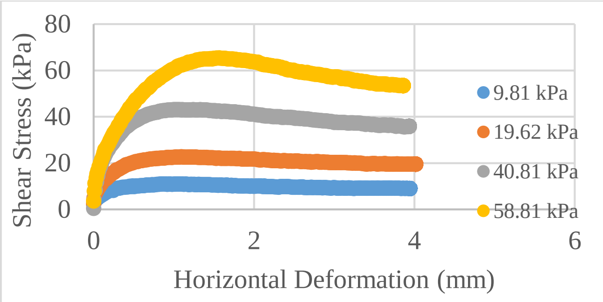
20% Clay + 80% Sand

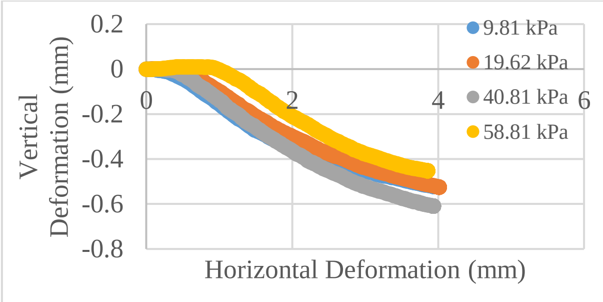


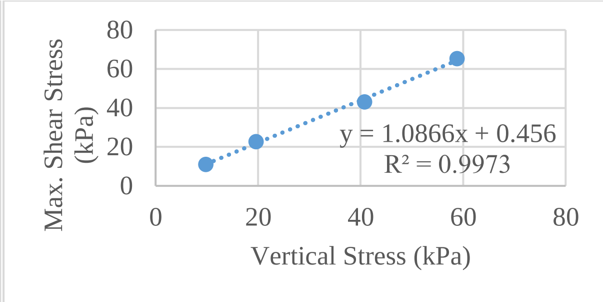
10% Clay + 90% Sand



100% Sand

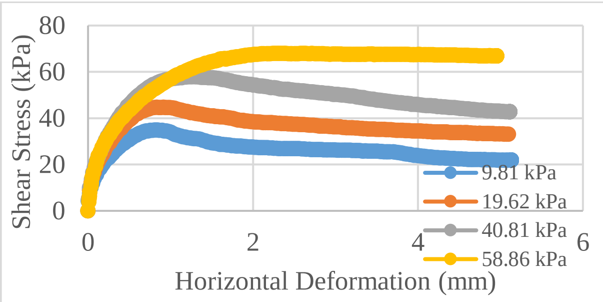


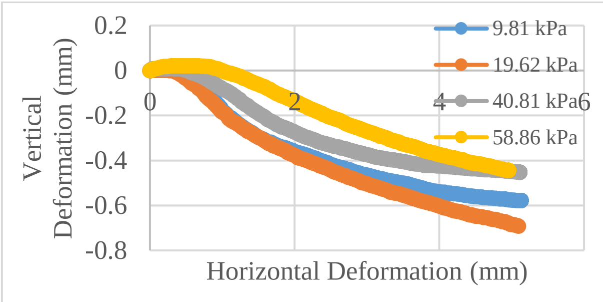


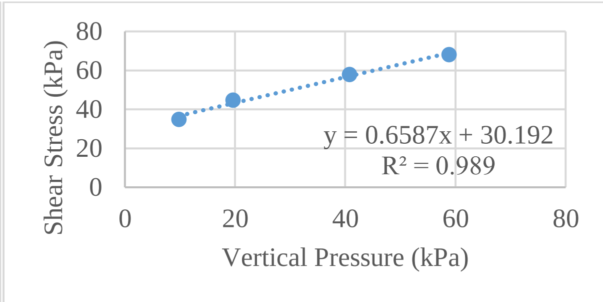


10% Chip tyre content

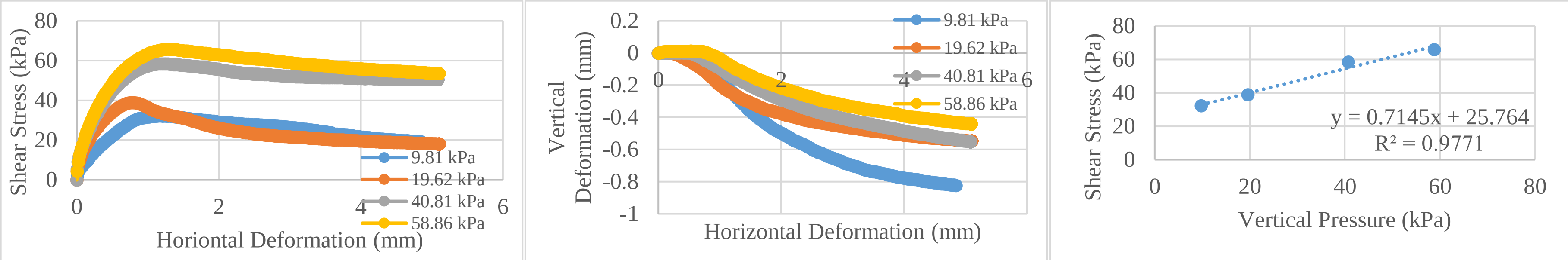
Clay



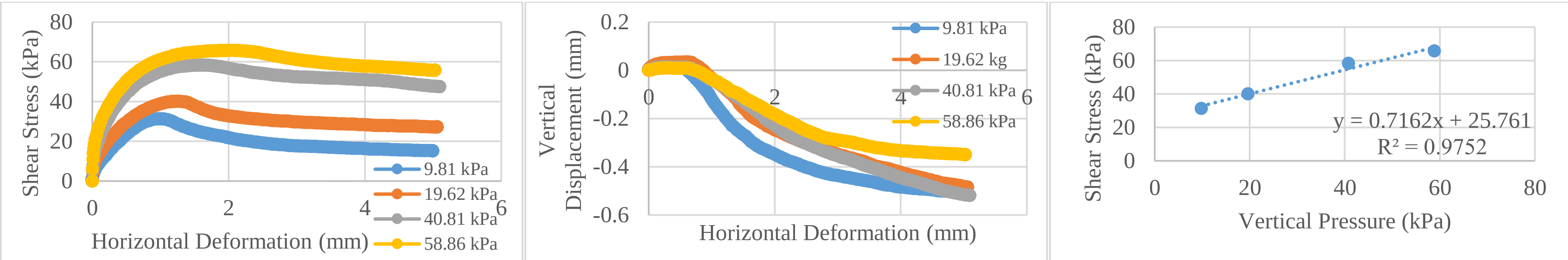




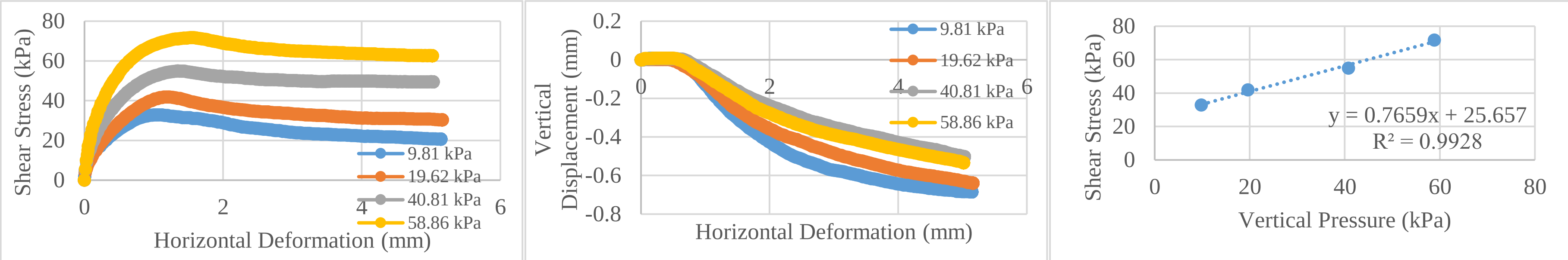
90% Clay + 10% Sand



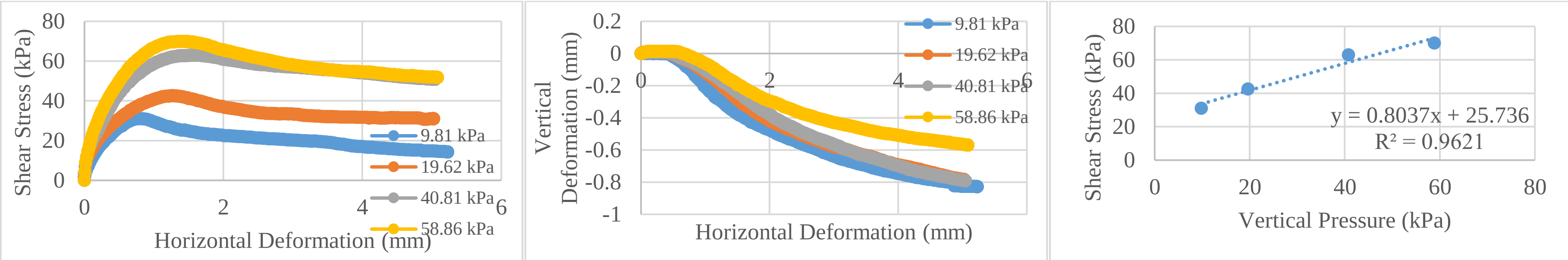
80% Clay + 20% Sand



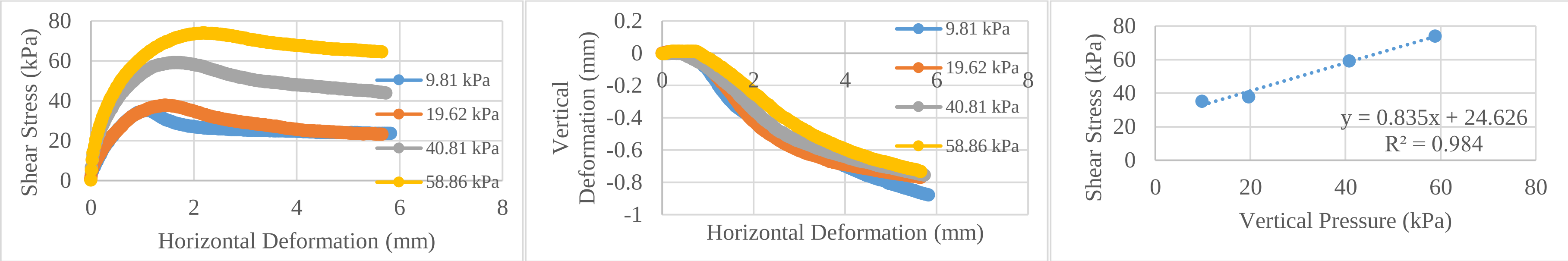
70% Clay + 30% Sand



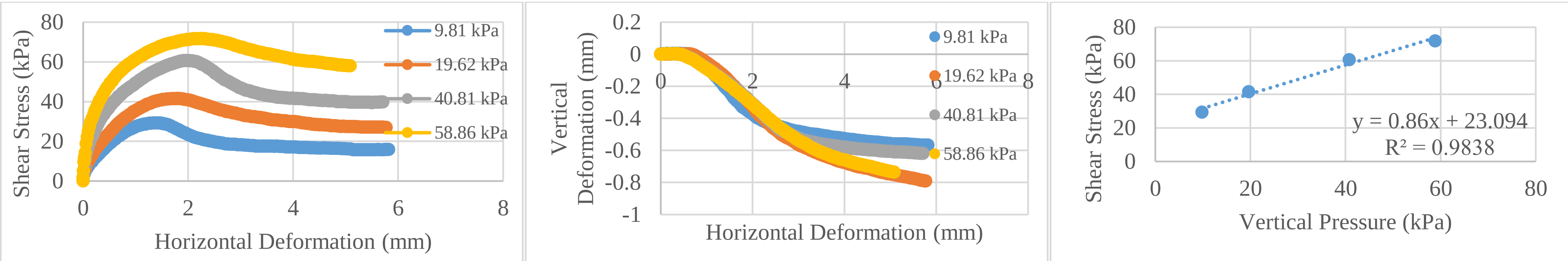
60% Clay + 40% Sand



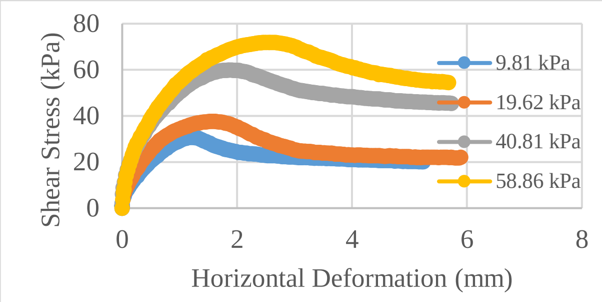
50% Clay + 50% Sand

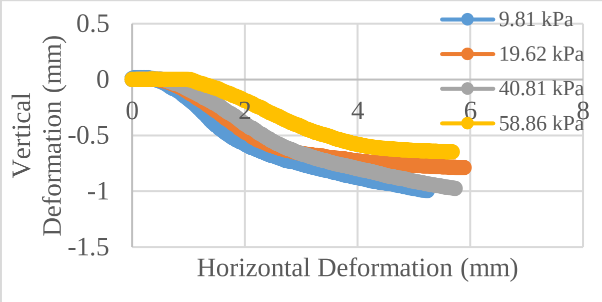


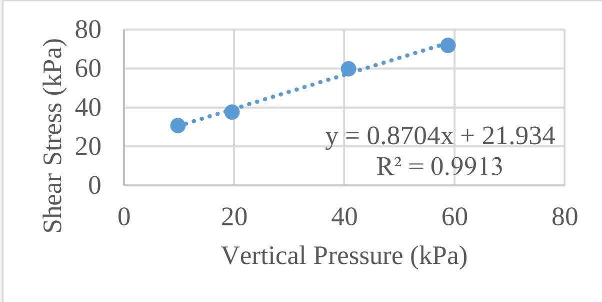
40% Clay + 60% Sand



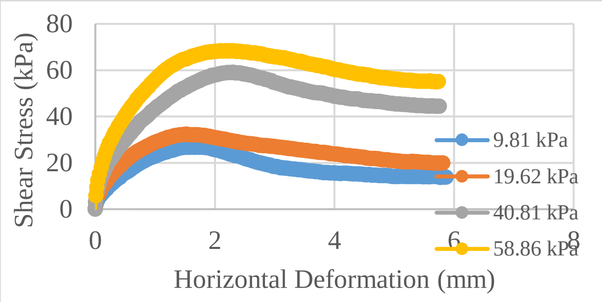
30% Clay + 70% Sand

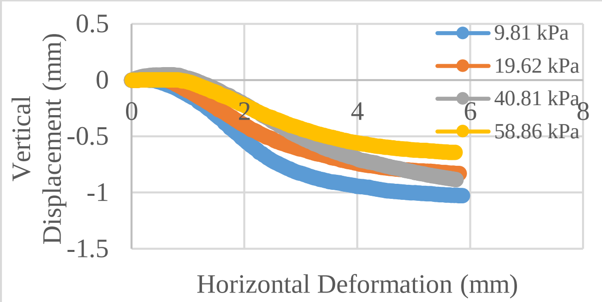


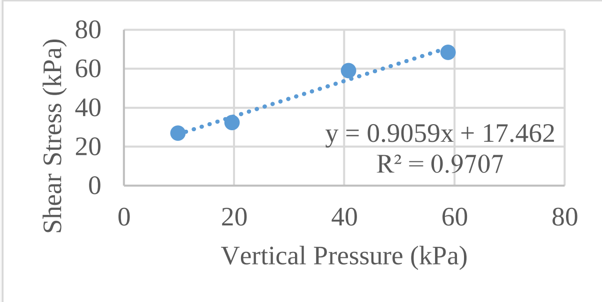




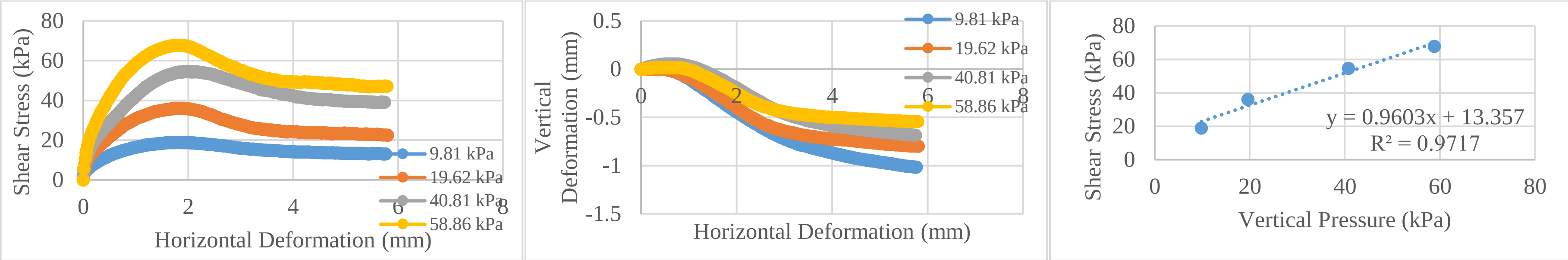
20% Clay + 80% Sand



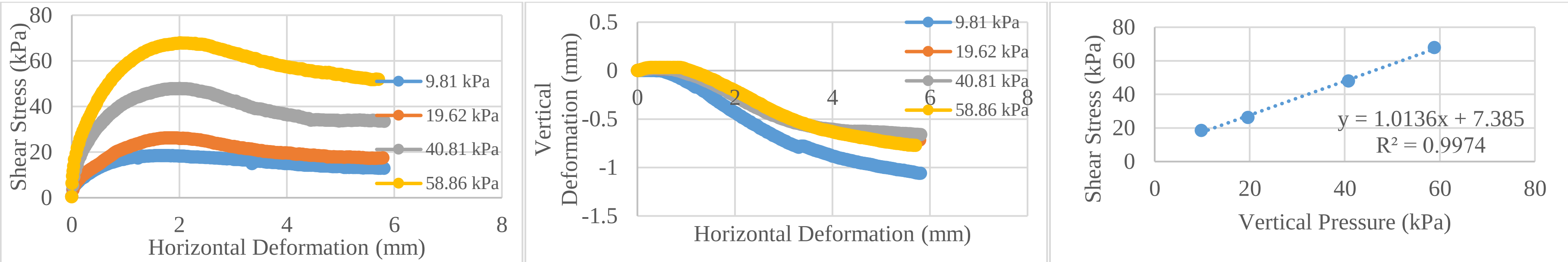




10% Clay + 90% Sand

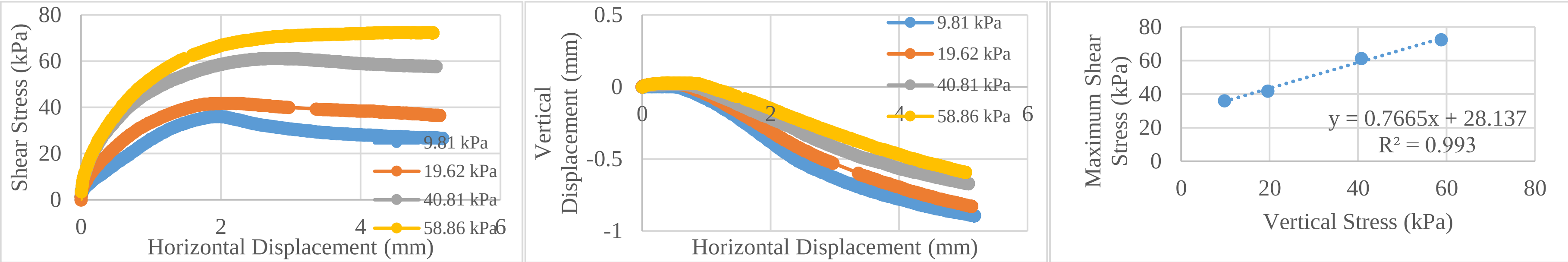


100% Sand

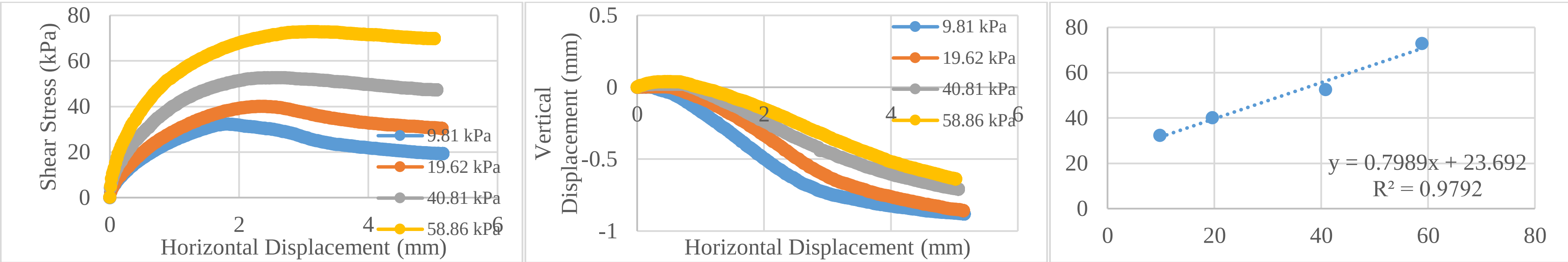


20% Chip tyre content

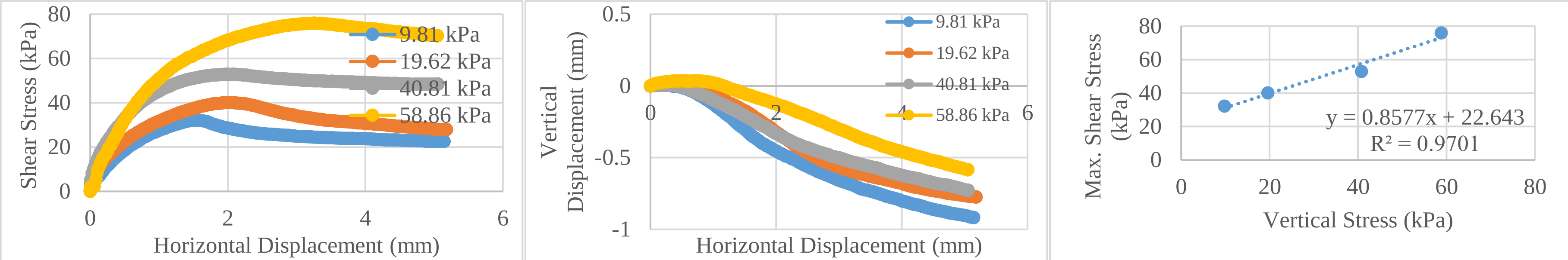
Clay



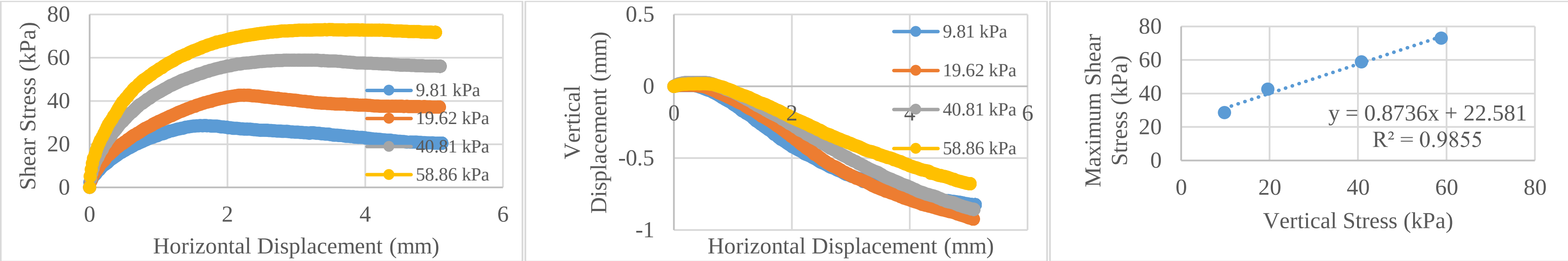
90% Clay + 10% Sand



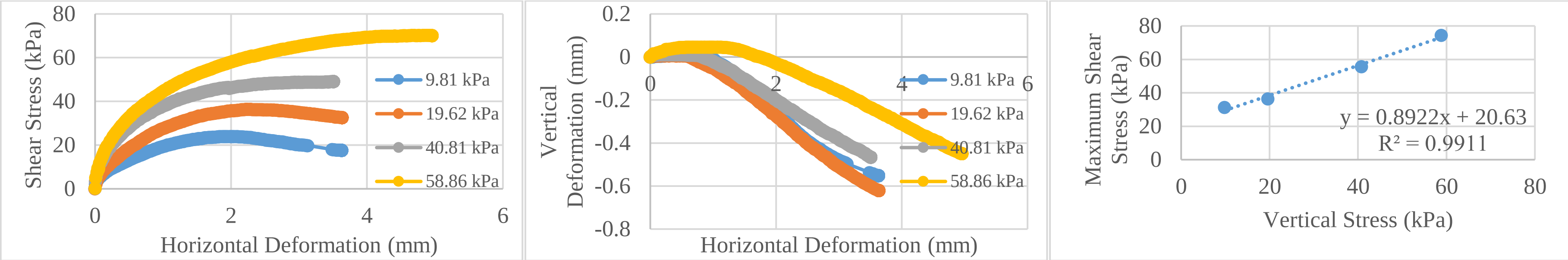
80% Clay + 20% Sand



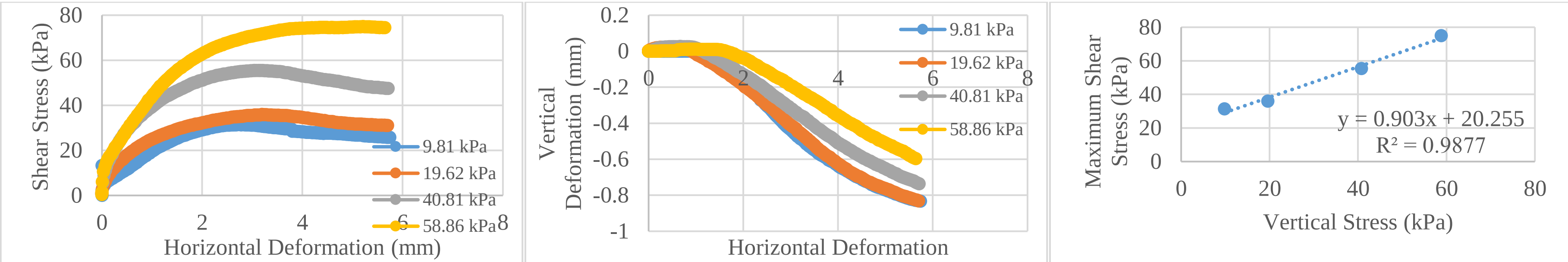
70% Clay + 30% Sand



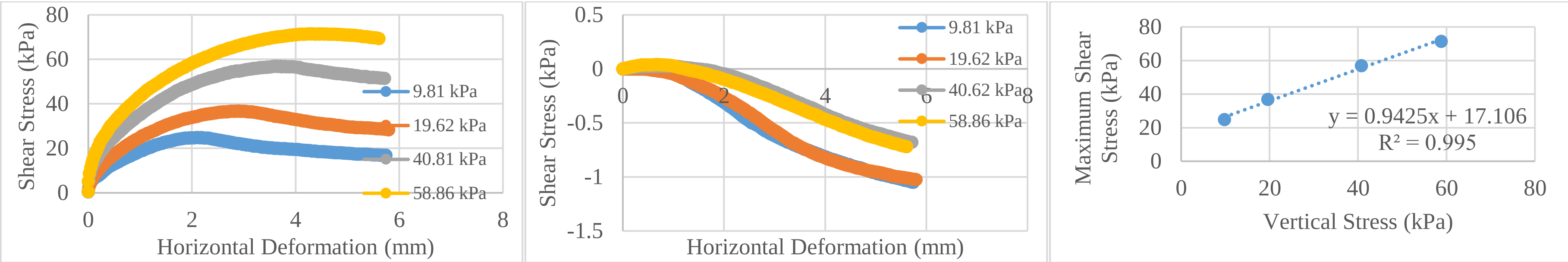
60% Clay + 40% Sand



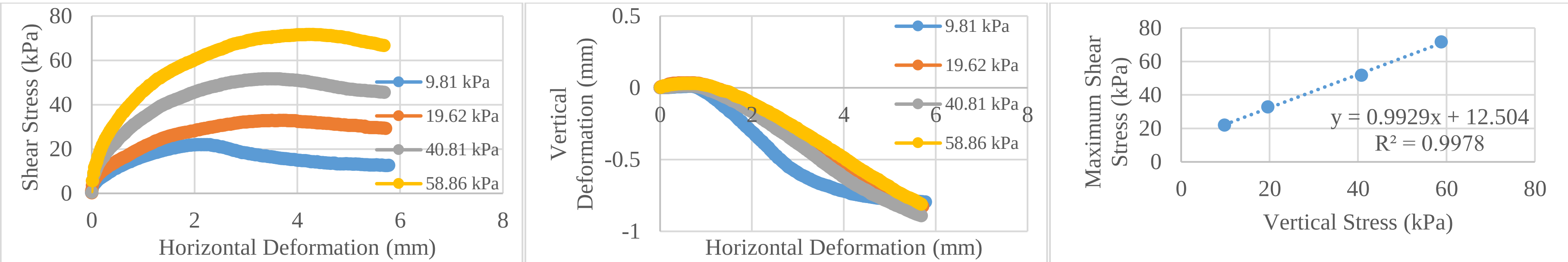
50% Clay + 50% Sand



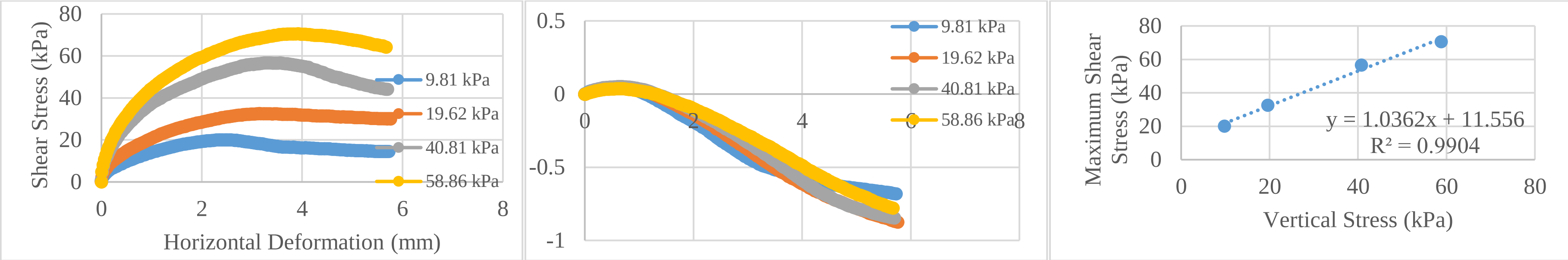
40% Clay + 60% Sand



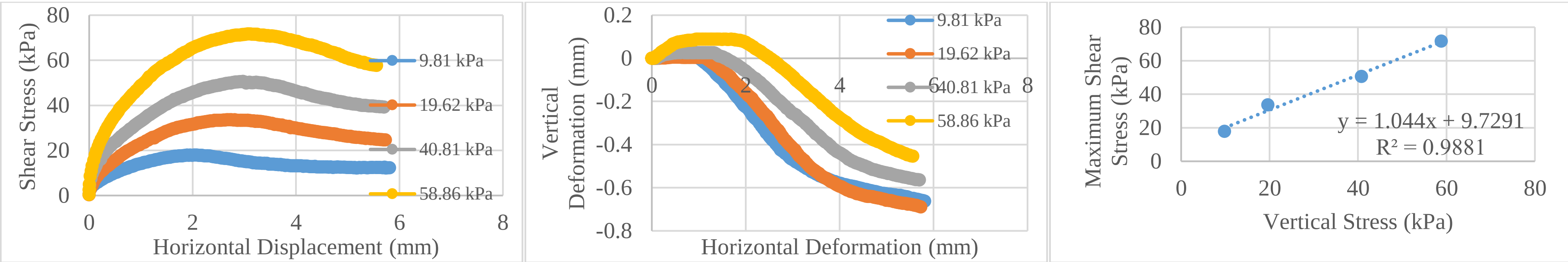
30% Clay + 70% Sand



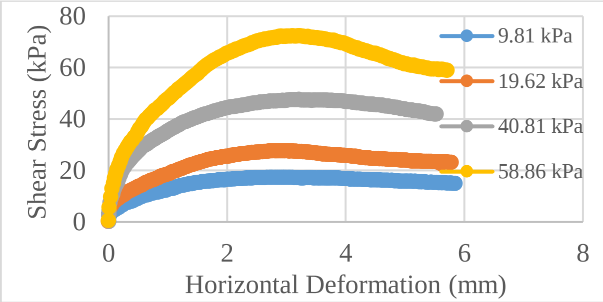
20% Clay + 80% Sand

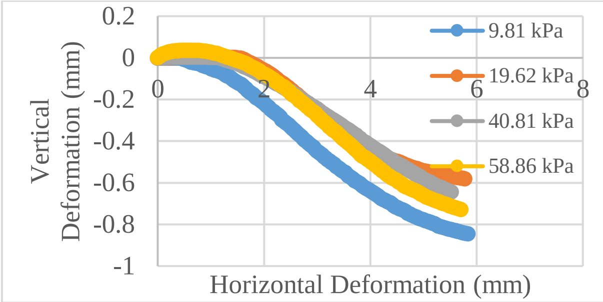


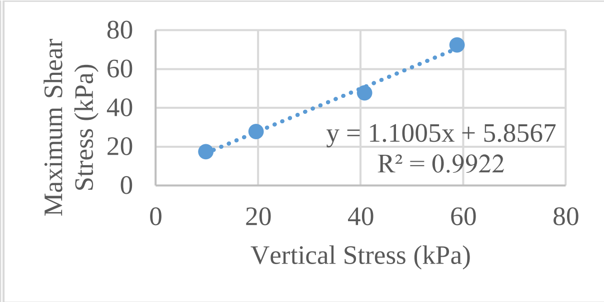
10% Clay + 90% Sand



100% Sand

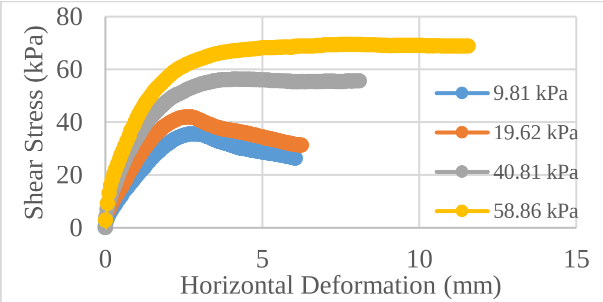


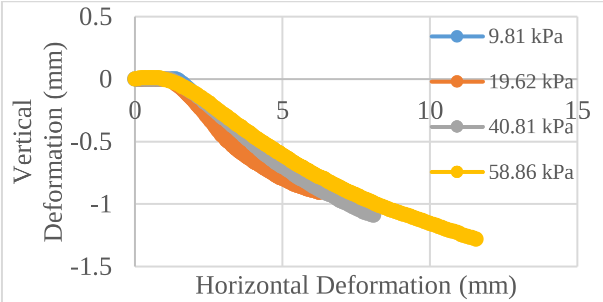


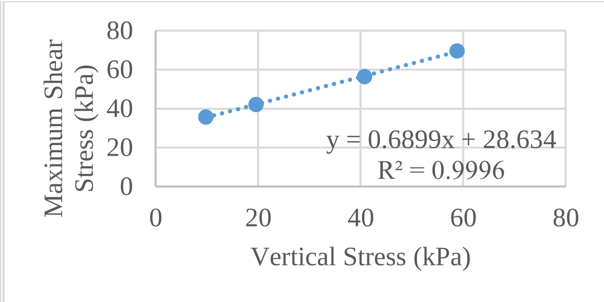


30% Chip tyre content

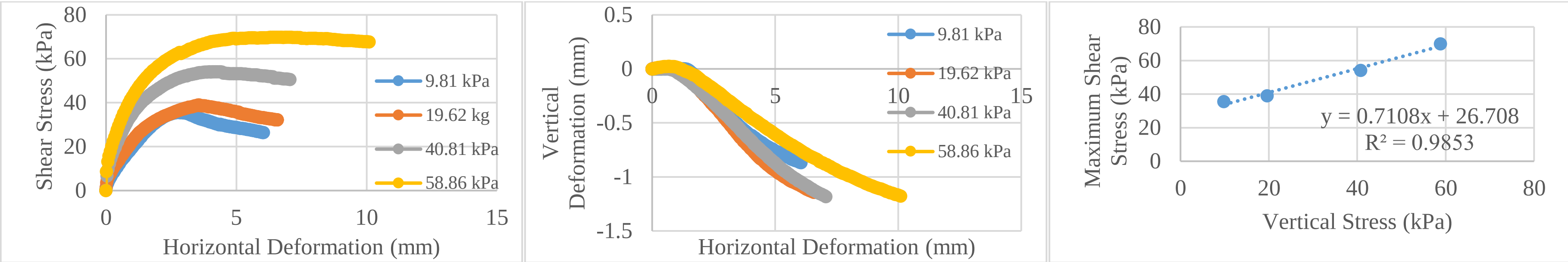
Clay



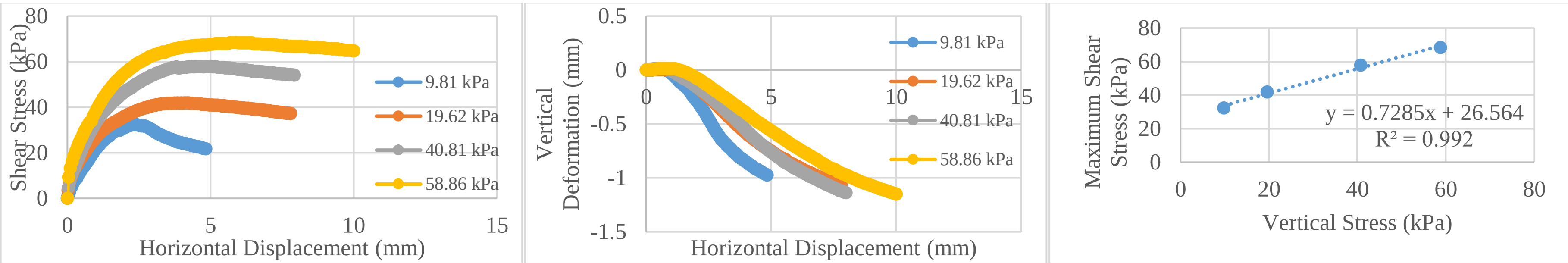




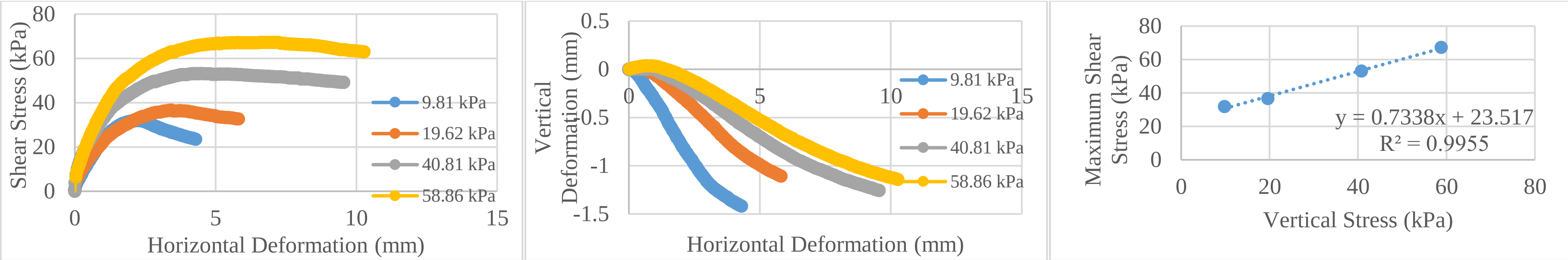
90% Clay + 10% Sand



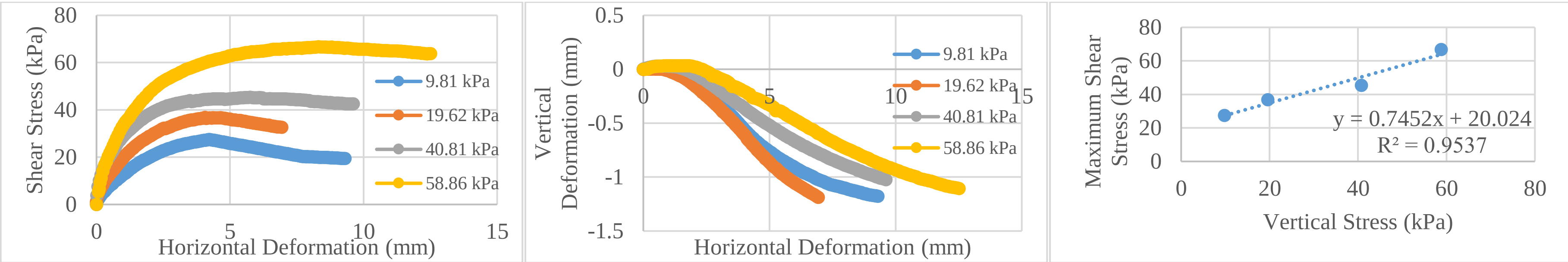
80% Clay + 20% Sand



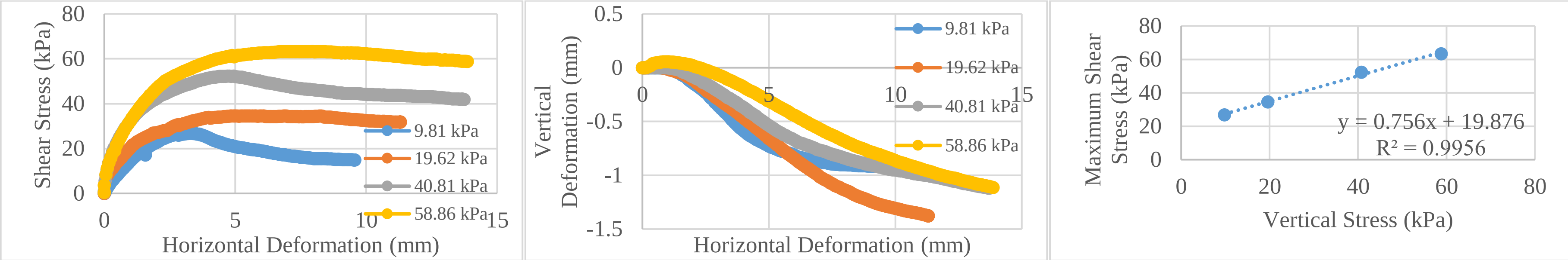
70% Clay + 30% Sand



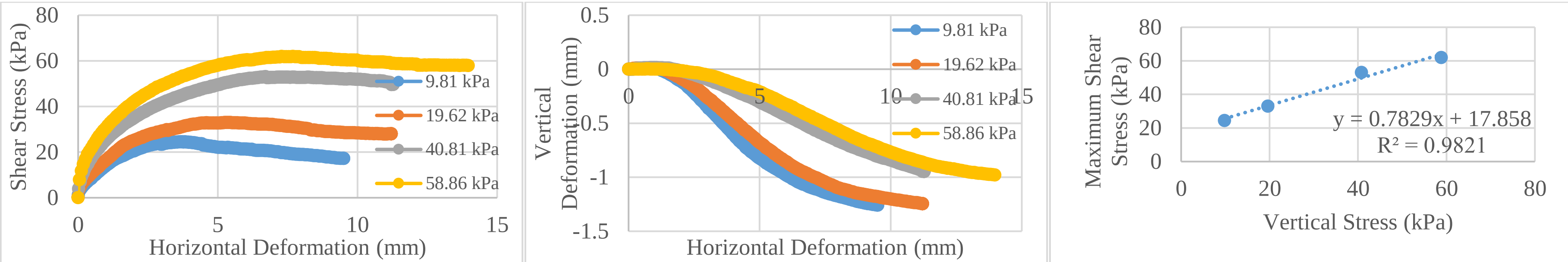
60% Clay + 40% Sand



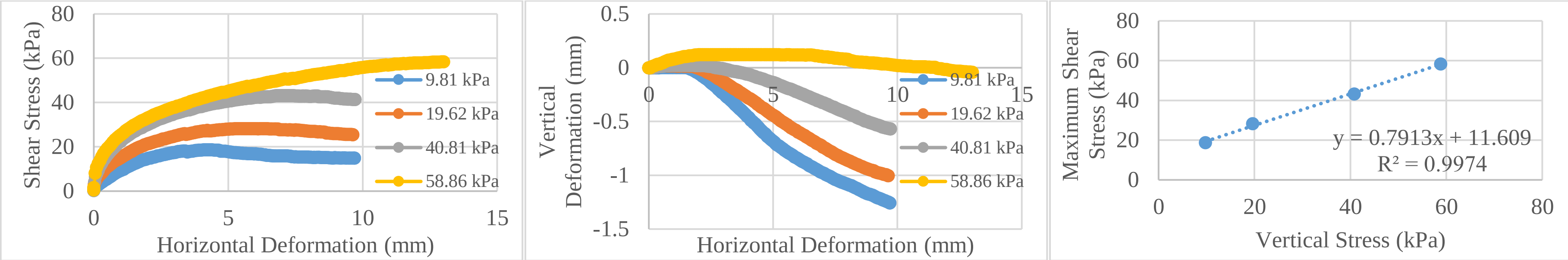
50% Clay + 50% Sand



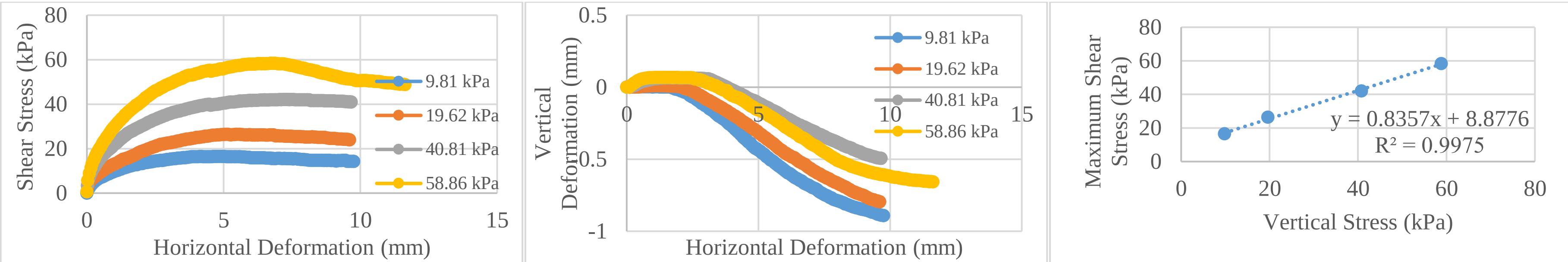
40% Clay + 60% Sand



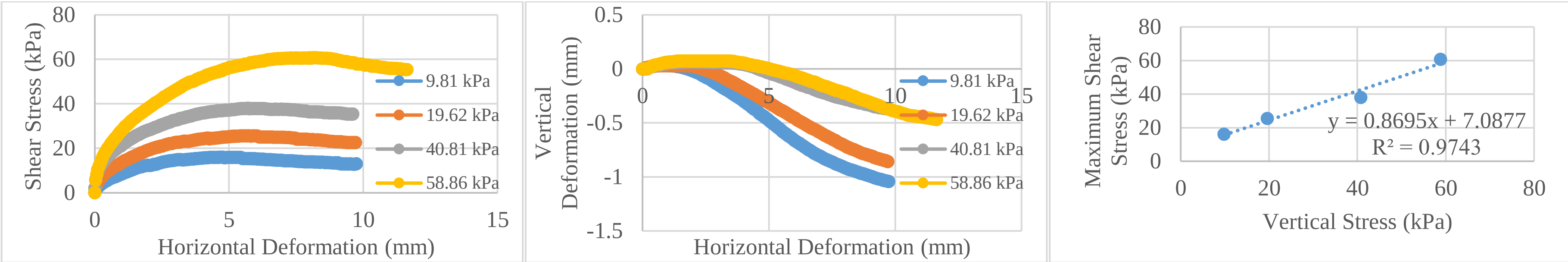
30% Clay + 70% Sand



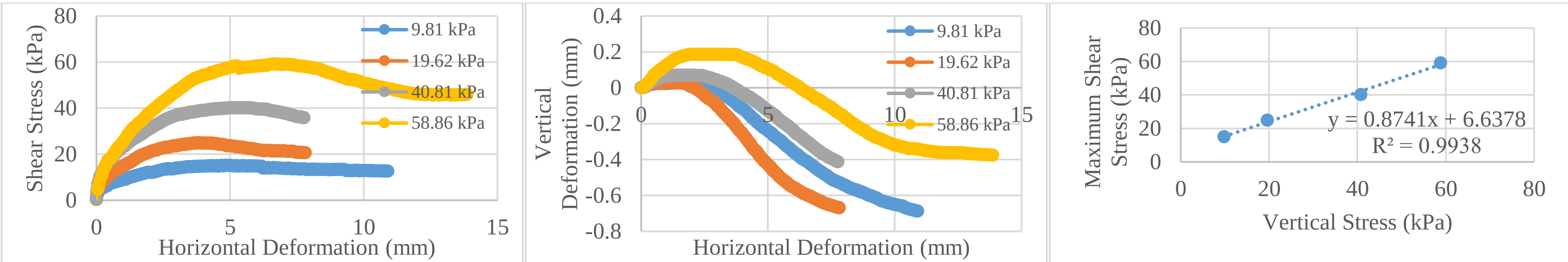
20% Clay + 80% Sand



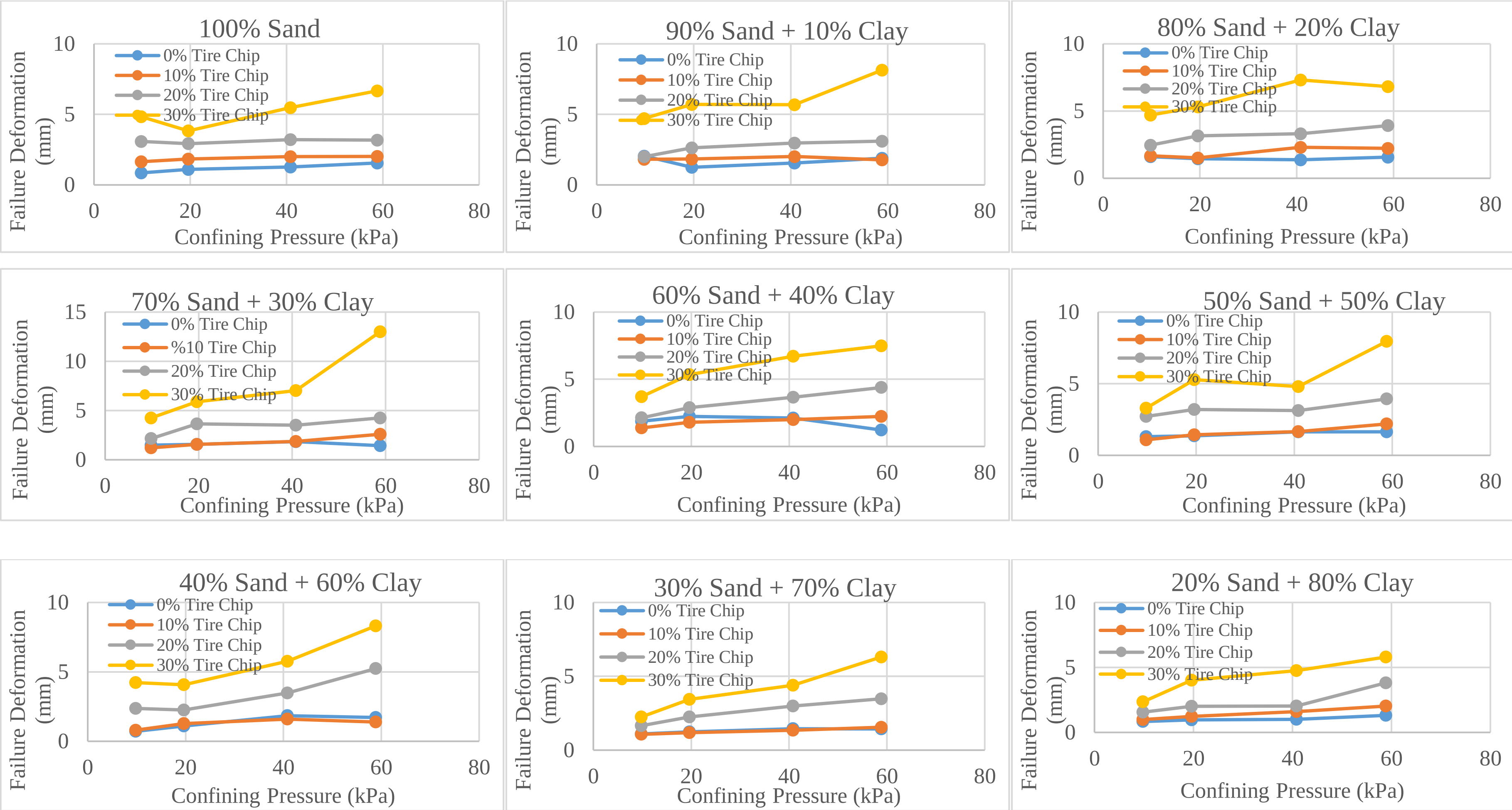
10% Clay + 90% Sand

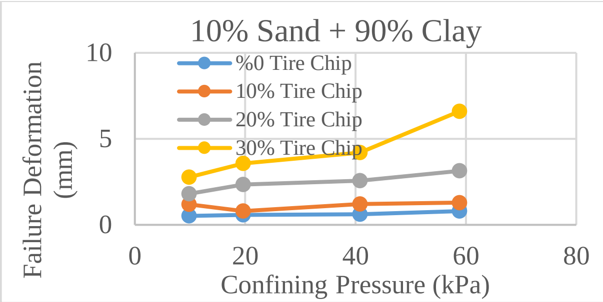


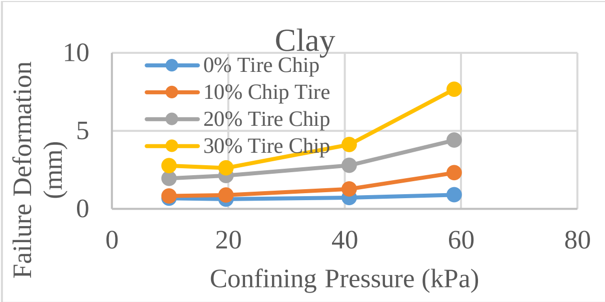
100% Sand



Attachment 2: Graphics of Failure Displacement



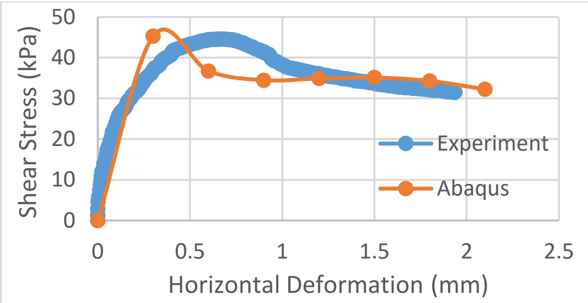

10% Sand + 90% Clay
Failure Deformation (mm)
10
5
0
0
20
40
60
80
Confining Pressure (kPa)
%0 Tire Chip
10% Tire Chip
20% Tire Chip
30% Tire Chip


Clay
Failure Deformation (mm)
10
5
0
0
20
40
60
80
Confining Pressure (kPa)
0% Tire Chip
10% Chip Tire
20% Tire Chip
30% Tire Chip

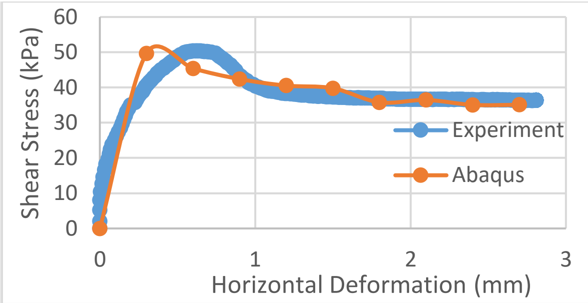
Attachment 3: Graphics of Finite Element Analysis

Clay

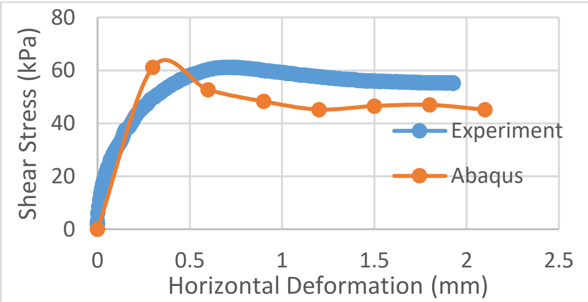
9.81 kPa



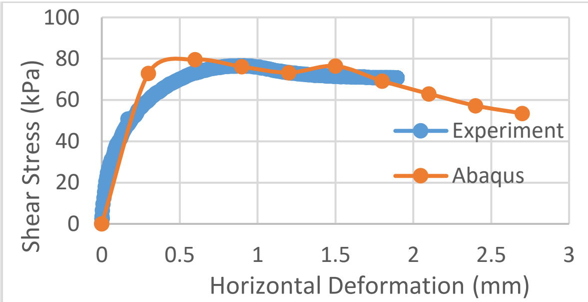
19.62 kPa

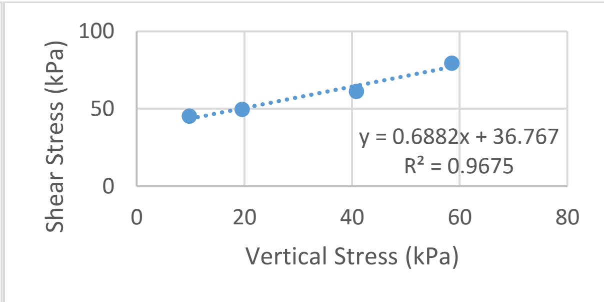


40.81 kPa



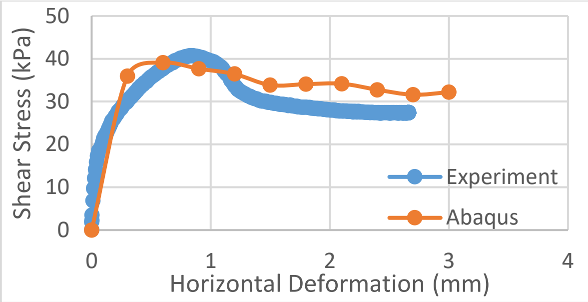
58.86 kPa



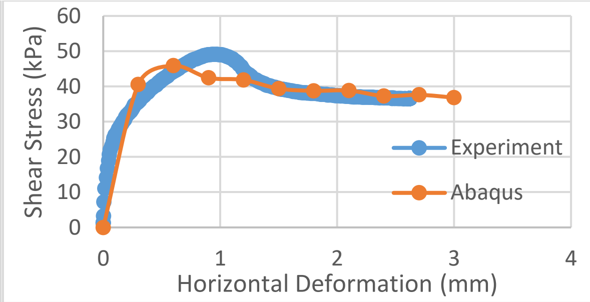


80% Clay + 20% Sand

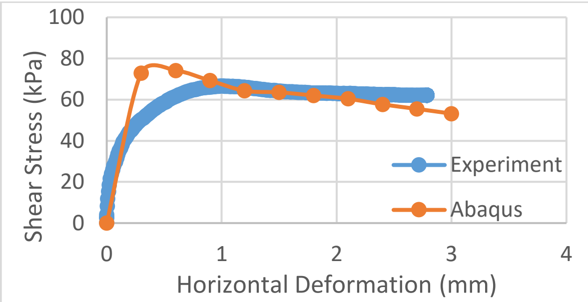
9.81 kPa



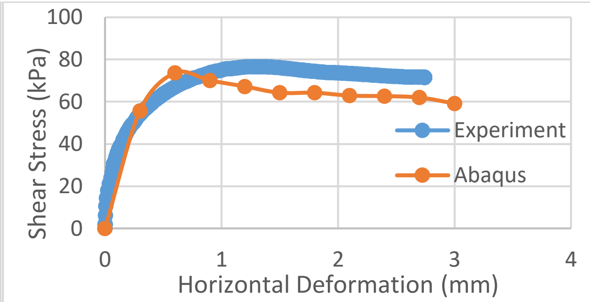
19.62 kPa

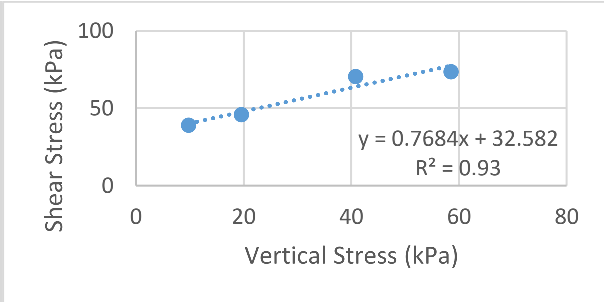


40.81 kPa



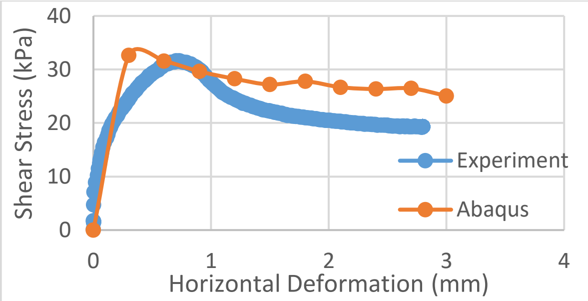
58.86 kPa

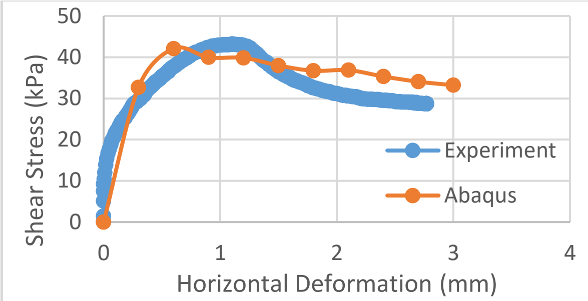




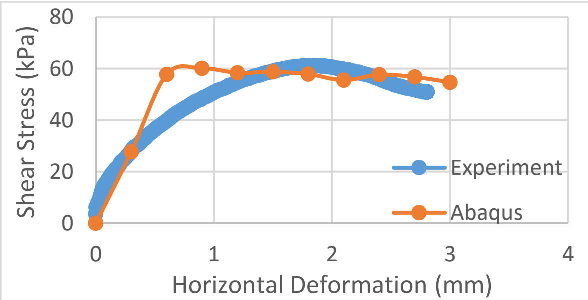
60% Clay + 40% Sand

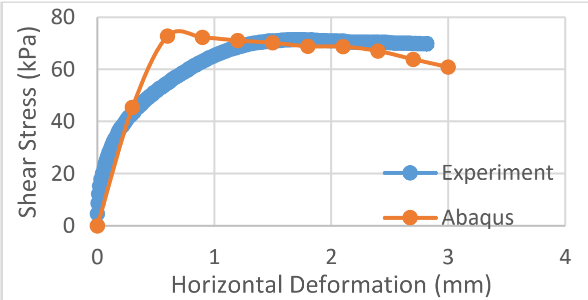
9.81 kPa **19.62 kPa**

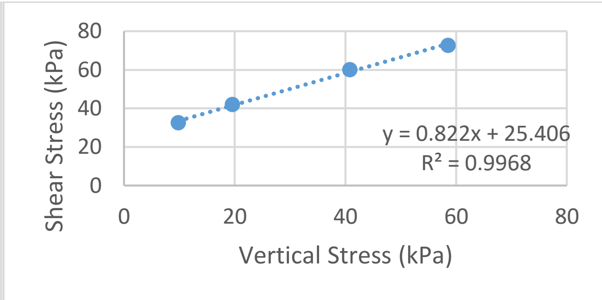




40.81 kPa **58.86 kPa**

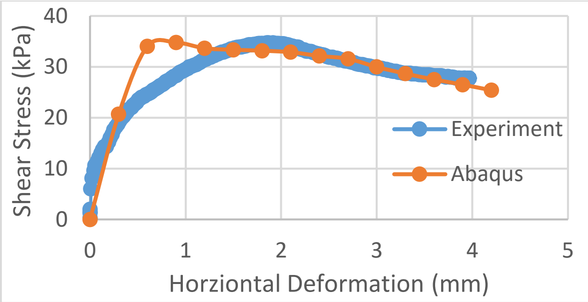




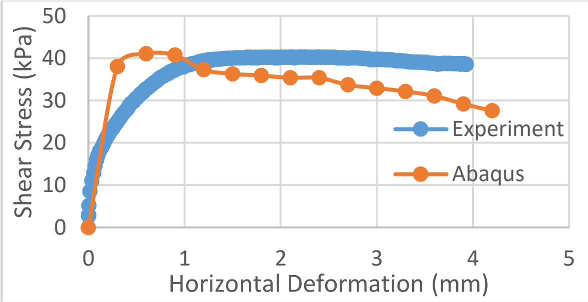


40% Clay + 60% Sand

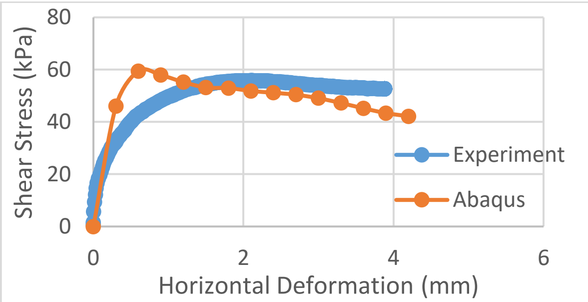
9.81 kPa



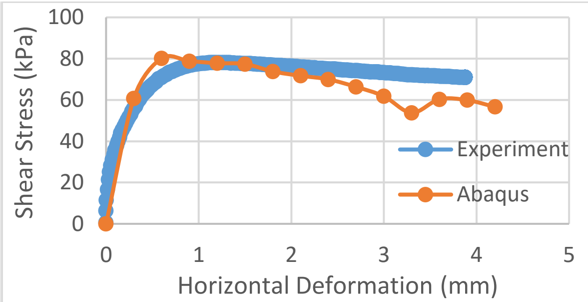
19.62 kPa

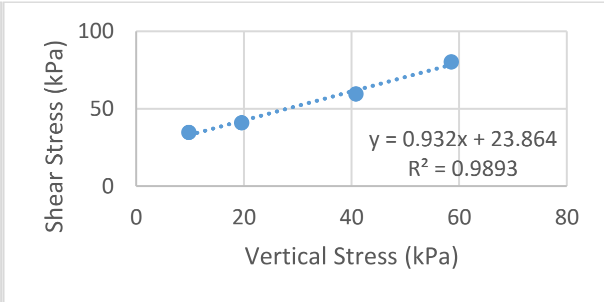


40.81 kPa



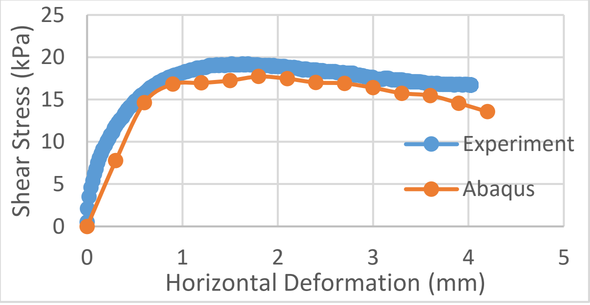
58.86 kPa



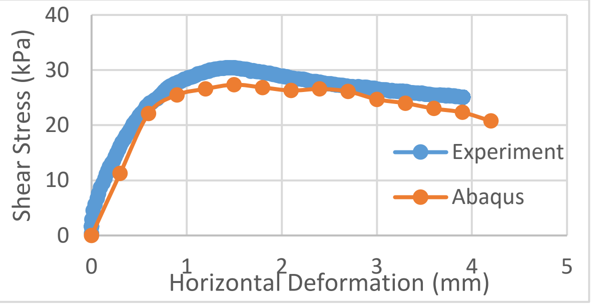


20% Clay + 80% Sand

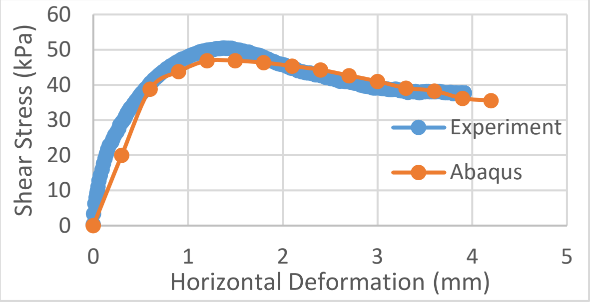
9.81 kPa



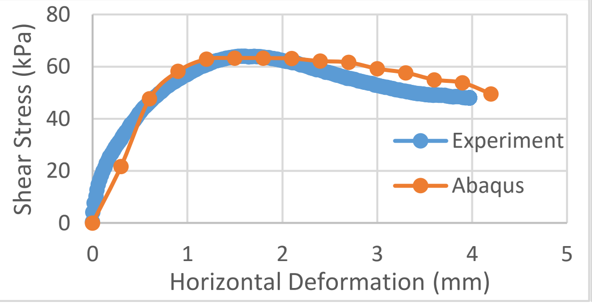
19.62 kPa

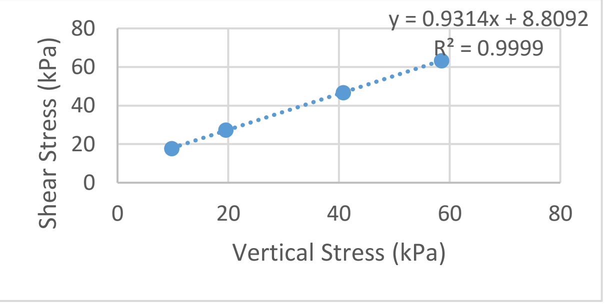


40.81 kPa



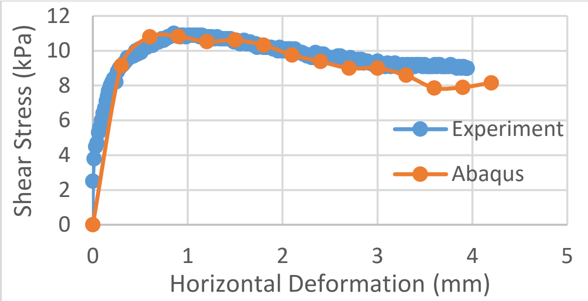
58.86 kPa

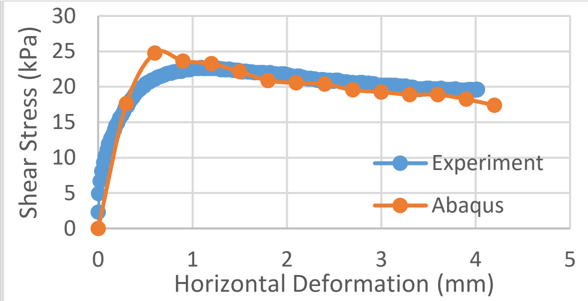




100% Sand

9.81 kPa **19.62 kPa**





40.81 kPa **58.86 kPa**