Undrained vertical bearing capacity of strip footing on GCL layer

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ABSTRACT: Geosynthetic Clay Liner (GCL), nowadays, represent a new technology that acceptance as a barrier system in sealing and waste landfills implementation. This paper investigates the undrained vertical capacity of strip footing on sand when a layer of GCL applied under it. Numerical solutions are obtained by small strain finite element analysis. In this analysis the GCL modeled as a layer of very loose permeable clay. Based on the analysis results, the effects of GCL that applied under strip footing on bearing capacity of strip footing will be demonstrated.

 Keywords: Geosynthetic Clay Liner (GCL), Numerical model, Bearing capacity, Finite element

1 INTRODUCTION

Geosynthetic clay liners (GCLs) are waterproofing composite materials used in geotechnical and environmental engineering applications. GCLs are manufactured hydraulic barriers consist of sodium (calcium) bentonite clay boned to a layer (layers) of geosynthetic material (stitching, needle punching, heat bonding, wrapping, etc.).
Bentonite, has a high swelling potential that has very low hydraulic conductivity when hydrated with water. GCLs represent a new technology that acceptance as a barrier system in landfills, foundation and sealing applications. GCL technology offers some unique advantages. They are fast and easy to install, have low hydraulic conductivity and have the ability to self-repair any rips or holes caused by the swelling properties of the bentonite from which they are made. When installed in composite liners, GCLs hydrate under a compressive stress corresponding to the overburden load. After that, the GCL hydrates typically from both the liquid flux through defects in the geomembrane (GM) and the transfer of vapor and liquid water from the underlying soil through the GCL (Azad et al., 2011; Beddoe et al., 2010).
 Manufacturers' installation guidelines typically recommend that following installation of the GCL, either alone or as part of a composite liner, the liner be covered by at least 0.3 m (and sometimes 0.5 m) of drainage layer, soil cover, or ballast layer shortly after installation (Rowe et al., 2004). However this recommendation is often more honoured in the breach than the observance, with delays between the geomembrane placement and the covering that may range from months to many years (Thiel et al., 2006). When a composite liner involving a black geomembrane is not covered, solar radiation can heat the geomembrane to 70 _C in south-eastern Ontario, Canada (Rowe et al., 2012; Take et al., 2014a, b) and in excess of 80 _C in Texas, USA (Peggs, 2008).
The use of reinforced soils to support shallow foundations has recently received considerable attention. The benefits of including reinforcements in the soil mass to increase the bearing capacity and to reduce the settlement of the soil foundation have been widely recognized. Experimental,

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numerical, and analytical studies have been performed to investigate the behavior of reinforced soil by using different soil, reinforced and footing types (Sharma et al., 2013).

Depending on the depth and soil underlying compressibility of a shallow foundation, different of failure may occur (Fig. 1). When a shallow foundation is loaded, two distinct shear planes will develop directly below the foundation and create a triangular zone. Three kind of failure model of shallow foundations are shown in Fig. 1. As this wedge move downward, the adjustment soil will yield according and its ultimate bearing capacity will be reached. If the soil is uncompressible and shear planes develop to the surface, general shear failure arise (Fig 1.a). If the soil is very compressible then volume change is promoted, shear planes will barley develop and punching shear failure will occur (Fig 1.c). Between the above models of failure, local shear failure will occur (Fig 2.b).
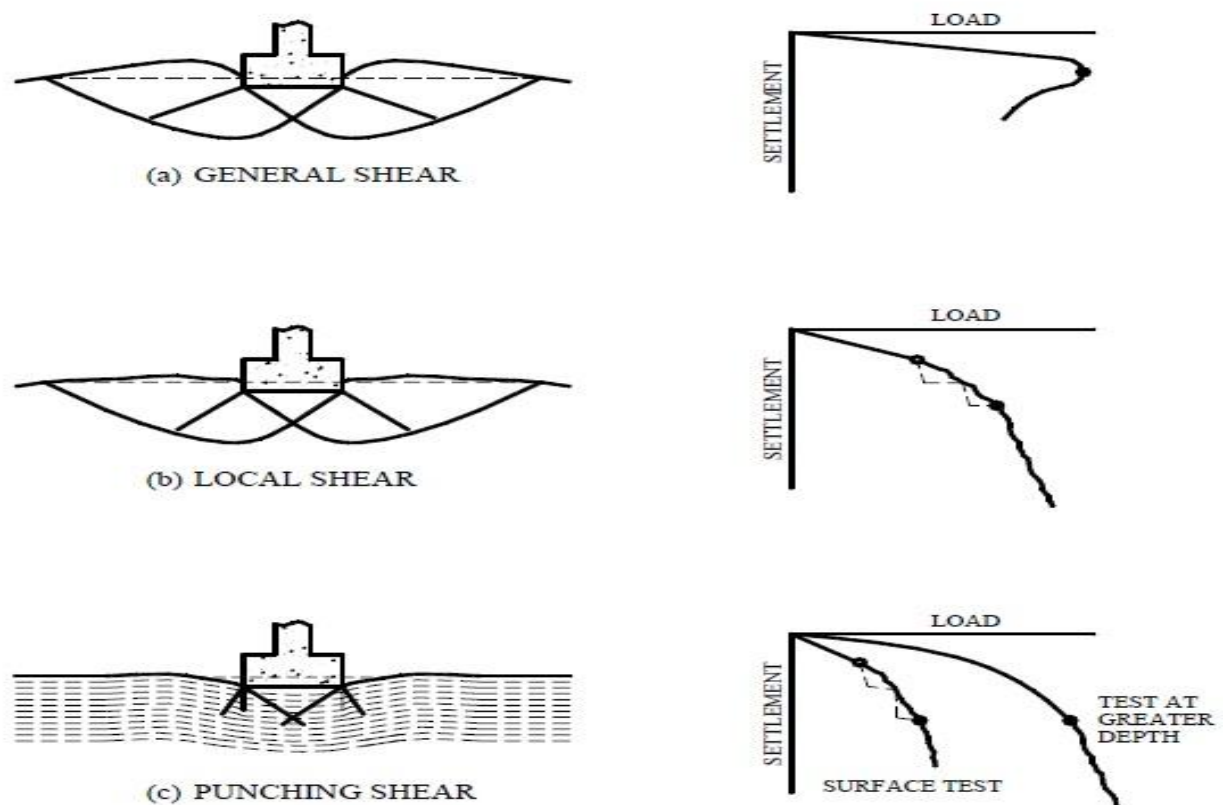

Fig.1 (a) General shear failure (b) Local shear failure (c) Punching shear failure3 (principles of geotechnical engineering,B.M.Das)

Fig. 2 shows the problem geometry studied and defines the key parameters. As shown in Fig. 2a, a strip rigid footing of width B is placed on an isotropic, non-homogenous soil with an undrained Young's modulus E_u and uniform unit weight γ rested on a layer of GCL.

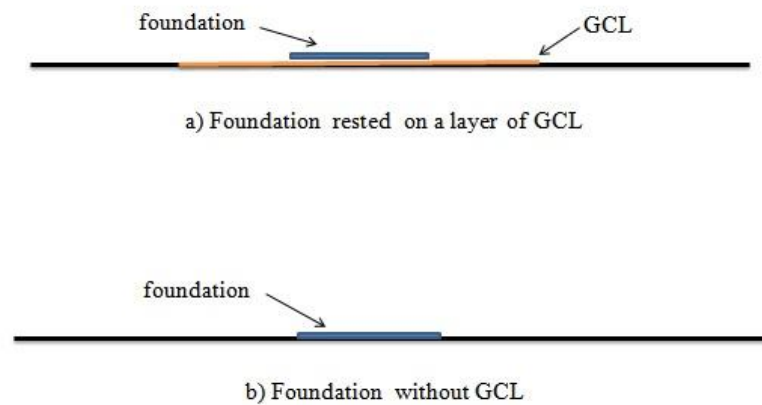In Fig. 2b the strip rigid foundation rested on ground (without GCL).

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Fig. 2 Problem definition

In this paper the effects of GCL on foundation's settlement with numerical solution will be investigated and the result of numerical models with a layer GCL and without GCL presented and discussed.

2 NUMERICAL MODEL

The following part provides the verification of the numerical analysis by the model test results.Small strain finite element analysis of surface strip footings were carried out using a commercially available Plaxis 2D. The soil (Qom's alluvium) was modeled with fifteen node triangular elements and used fine mesh (Fig. 3)

The soil was modeled as Mohr-coulomb material using the elastic –plastic Mohr-Coulomb failure criteria. Poisson's ratio of $\upsilon=0.3$, friction angle $\varphi=34$ and dilation angle of $\psi=4$ were set to simulate the undrained Qom's alluvium soil. The undrained Young's modulus and bulk unit weight were assumed to be $Eu = 88$ MPa and $\gamma=18.5$ kN/m3. It is worth noting that the undrained bearing capacity of a surface footing resting a layer of GCL is insensitive to the soil unit weight.

The GCL was model as Mohr-coulomb material using elastic Mohr-coulomb failure criteria. The GCL was model as a thin layer bentonite between two layers of geotextile. For Bentonite, Poisson's ratio of $V=0.35$, friction angle and dilation angle $\varphi= \psi=0$ were set to simulate. The undrained Young's modulus and bulk unit weight were assumed to be $Eu = 105$ MPa and $\gamma=18$ kN/m3. For geotextile, tensile strength 12 kN/m2 were set to simulate.
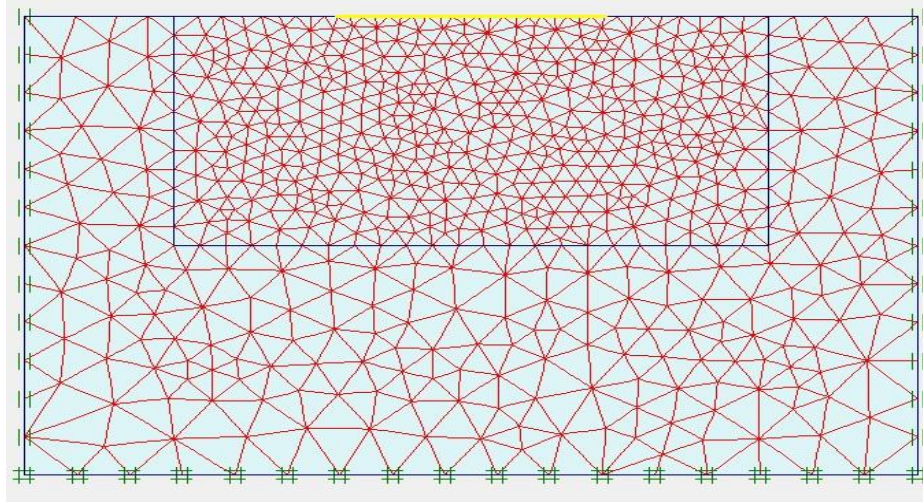
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Fig.3 Numerical simulation to verify

3 VERIFICATION OF NUMERICAL MODEL

Simulation was proposed by Fox & Bebatista,1996, to analysis the bearing capacity of GCL for cover soils. The basic idea of the subset simulation approach is that the small failure probability can be expressed as a product of larger conditional failure probabilities (Fox & Bebatista,1996).

The geometry for experimental model is shown in Figure 4, a GCL specimen was fit to bottom of a confining mold having a diameter, D=235 mm, and a layer of cover soils was placed over the GCL. Surcharge weight were placed on top of the specimen. The vertical stress on the specimen applied with a piston with 50 mm diameter to 20mm penetration. The geometry of the penetration is shown in Fig 4.a. The distance from the top of the mold to the GCL surface was measured using a caliper, Figure 4b.

One bearing capacity test was initially applied for GCL covered with 50 mm of sand and force-displacement curve was drawn.
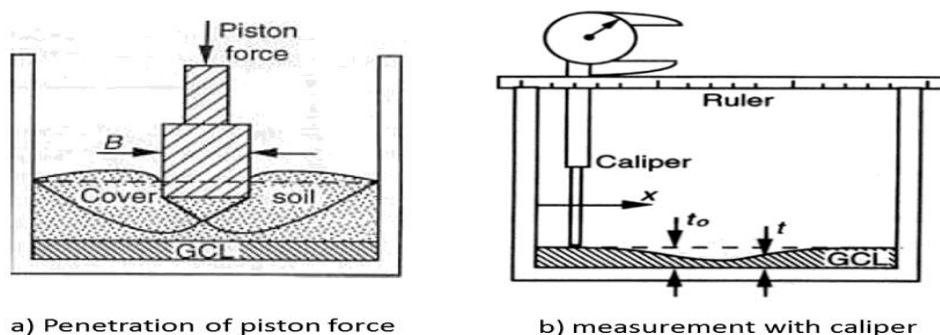

Fig. 4 Geometry for experimental model (Fox & Bebatista, 1996)

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With finite element analysis the model box simulated. A GCL assumed in bottom of model and a layer of sand with 50 mm height considered top of GCL. The vertical force applied to numerical model to access 20mm penetration and result compare. In Fig. 5 Force-displacement diagram is provided and it shows that numerical analysis has a good validation with experience model.
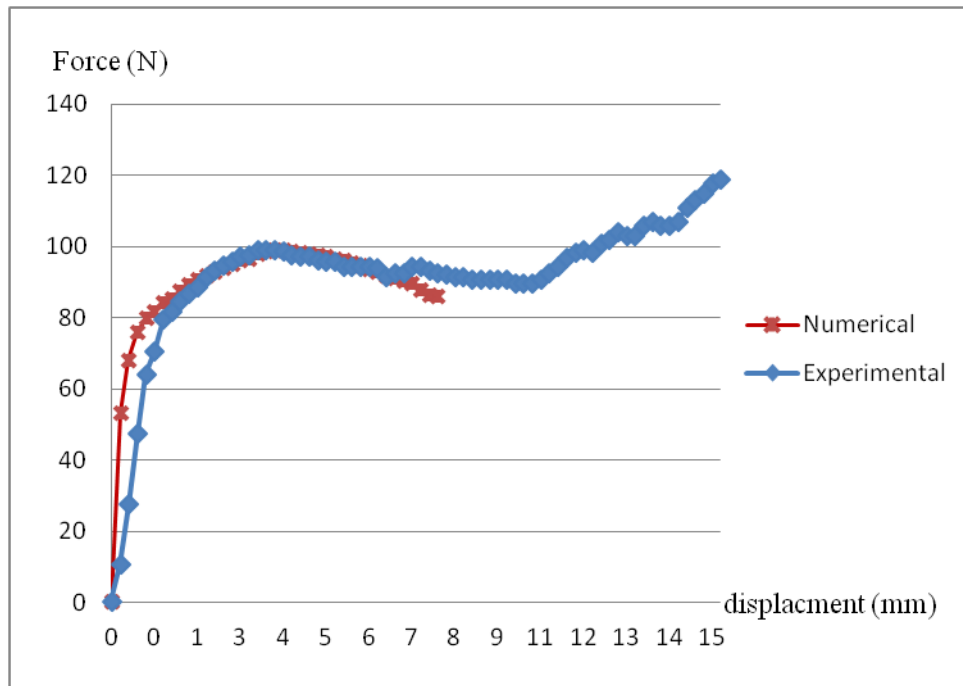

Fig. 5 Force- displacement diagram

4 NUMERICAL MODEL ANALYSIS

The results which obtained from the model tests were verified by carrying out numerical studies by using the finite element method. This analysis aims to identify the increase of bearing capacity of foundation that applied on a layer of GCL. The soil in this analysis was simulated by the Mohr–Coulomb failure criteria, which is an explicit and rather compatible and agrees with experimental testing results compared with other models. The plain strain condition and 15-node triangle elements were used for this analysis. The parameters of Qom alluvium obtained from experimental tests. GCL element which is defined by a layer of bentonite boned two layers of geotextile. The virtual interface element with GCL element was simulated before mesh generation. In all calculations described in this research, force control technique is considered, that is mean, load applied on alluvium until failure occurred and ultimate load clarified. The input values distribution load are given in force per unit of length (for example kN/m). The value of applied point (load system A) is taken according to the obtained value from the model test divided by the footing width in plane.

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The properties of the adopted alluvium which were modeled and defined in the program are (γ= 18.5 kN/m3, υ =0.3, E =88000 Pa, friction angle φ=34° and angle of dilatancy = 4°). The GCL is simulated as a layer bentonite that boned in two geotextile layers (EA =12 kN/m).

A comparison between the load displacement responses was calculated using the finite element analysis and the results obtained from the relevant model tests for various shallow foundation that rested on a layer GCL is applied.

The results of the finite element analysis and its output are provided in Fig.6, for different foundation without and with GCL. The total stress obtained from the analysis is shown in Fig. 6 (for 1, 3, 5, 7 and 9 meter foundations width).That is shown that the value of loading and settlement increase with foundation width. Generally, the comparison between foundations rested on a layer of GCL and without GCL foundations indicates that the value of loading to failure in foundations rested on GCL is more than failure loading in foundations without GCL. The total stress associated at failure are shown in Fig. 6, for different foundation type. The distribution of the extreme total stress is presented in shading area, where the red shading refers to maximum strains. It is noticed that for foundations without GCL, the maximum strains or high stressed zones are found directly below the footing within the depth equal to B (foundation width) and distinctly reduced in both lower depth and horizontally at adjacent foundations sides, also for foundation with GCL, the maximum strains or high stressed zones are found directly below the footing within the depth equal to B (foundation width) (Fig.6).

Generally, it can be observed that, the contact pressure at failure increases a little with using a layer of GCL under the foundations (Fig.7). The comparison between the foundations with various width with and without GCL indicates that the GCL layer possessed more confined pressure as shown in the relevant Figure, while the values of the contact pressure of the foundation without GCL was smaller than that of foundation with GCL cases.
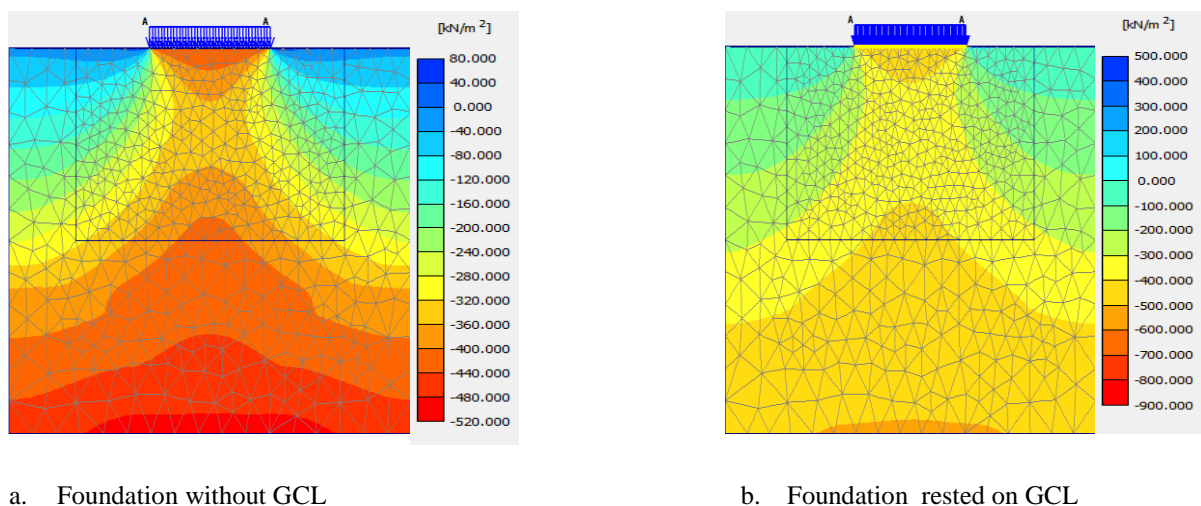
a. Foundation without GCL   b. Foundation rested on GCL

Fig.6 . Numerical model analysis of foundation with B (1m, 3m, 5m, 7m, 9m) width

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A comparison between foundations without GCL and foundations rested on a layer of GCL is shown in Fig.7. The following diagram reports some useful comments about the failure of foundations with and without a layer of GCL.
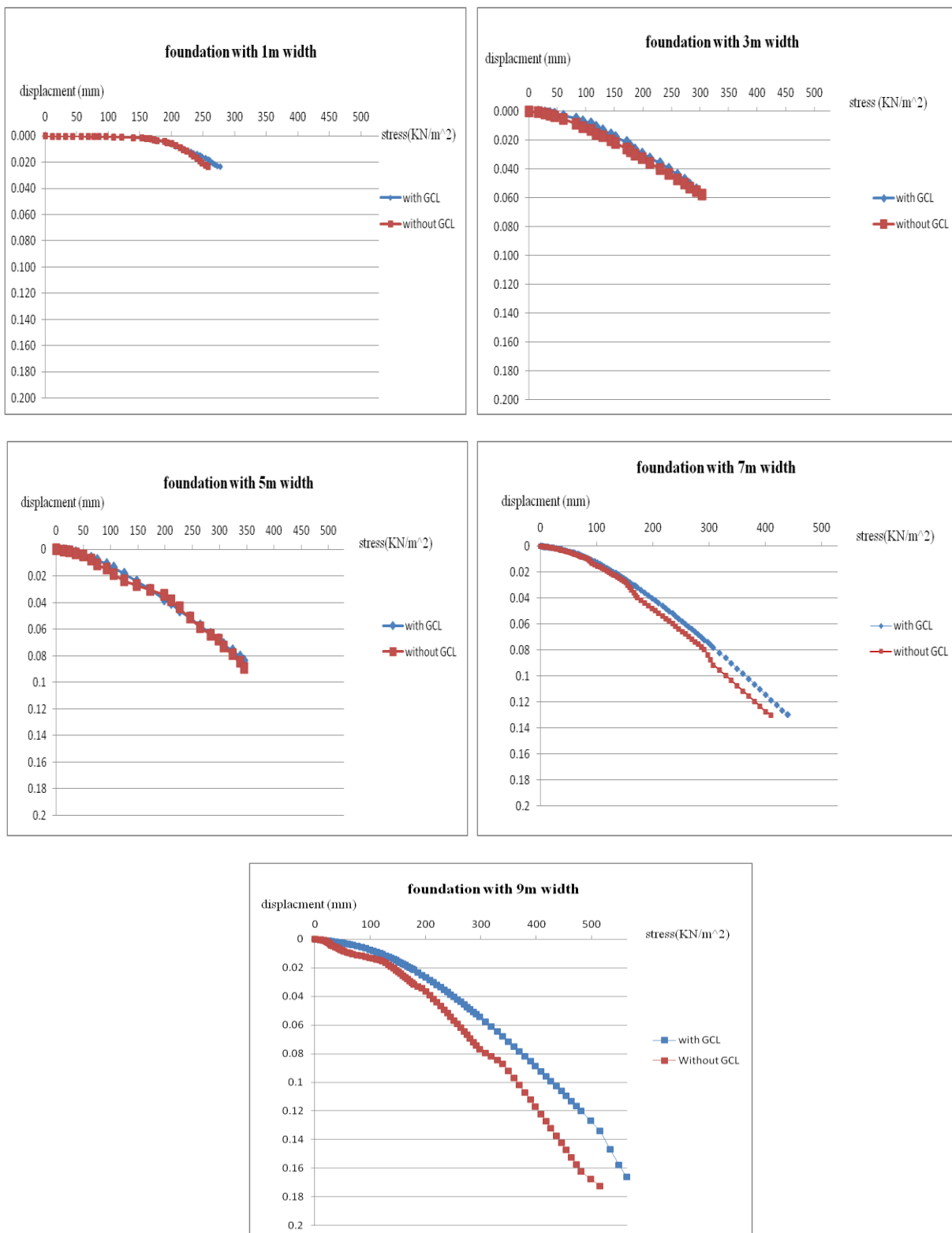

Fig.7. Diagrams of stress-displacement variation

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The major results are provided in table.1. Bearing capacity increase to use of GCL is almost 4.5% for various shallow foundations. There is no different in settlement of both foundation.

foundation width	Failure load(KN/m^2)		
B	without GCL	with GCL	Percentage(%)
1m	262	277	5.73
3m	303	310	2.31
5m	355	365	2.82
7m	416	441	6.01
9m	503	531	5.57

Table.1 Bearing capacity increase

5 CONCLUSION

In the present paper, the geotechnical behavior of foundation with and without GCL was investigated. It can be observed that, the contact pressure at failure, increases a little with using a layer of GCL under the foundations (almost 4.5%) and there is no different in settlement by using GCL under foundations. It is recommended for future work to analysis this kind of model in various situation and clarify the bearing capacity in that situation.

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