BEHAVIOR OF GEOSYNTHETIC-REINFORCED WALLS IN BACK-TO-BACK CONFIGURATION

C.S. Yoo[1], and D.W. Jang[2]
[1]Sukyunkwan University/Dept. of Civil and Environmental Engineering; Tel: +82-31 290 7518;
Fax: +82-31 290 7549; Email: csyoo@skku.edu,
[1]Sukyunkwan University/Dept. of Civil and Environmental Engineering; Tel: +82-31 290 7537;
Fax: +82-31 290 7549; Email: jdw23@naver.com

ABSTRACT

In recent years, the use of geosynthetic reinforced soil walls in back-to-back (BTB) configuration has been increasing in roadway and railway construction. In this paper, the failure mechanism of geosynthetic-reinforced soil walls was examined using the discrete element method based numerical investigation. The discrete element model adopted was first validated using the results of 1-g model tests. The validated discrete element method-based numerical model was used to further investigate the failure mechanism of back-to-back walls with different boundary conditions. The results were then compared with the failure mechanisms defined in the FHWA design guideline.

Keywords: Geosynthetic-reinforced wall, back-to-back wall, discrete element method, failure mechanism

INTRODUCTION

Since the introduction of geosynthetic reinforced soil walls in geotechnical engineering practice in the early 1980's, most of the applications were limited to private sectors, at least in Korea. In recent years, however, the geosynthetic reinforced soil walls are commonly being used in public sectors for bridge abutment and embankment construction (Yoo and Jeon 2010).

Surprisingly, studies concerning this subject are very limited although the design method is well documented in the FHWA design guideline (Elias and Christopher 1997). Related studies include the work done by Yoo and Kim (2008) and Han and Leshchinsky (2010). For example, Yoo and Kim (2008) performed a series of reduced scale model test to examine the effect of reinforcement distribution on the load carrying capacity of the back-to-back walls. Han and Leshchinsky (2010) presented the results of a numerical investigation using the limit equilibrium and finite difference methods to investigate the effect of the width to height ratio and the quality of backfill material on the locations and shapes of critical slip surface, the required tensile strength of reinforcement, and the active thrust to the reinforced zone, among others.

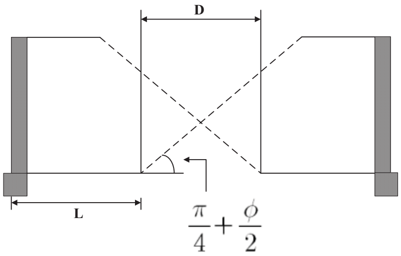
In this paper, the results of numerical investigation using the discrete element analysis on the failure patterns of back-to-back walls with various width to height ratios. The discrete element method was adopted in this study for realistic simulation of the back-to-back wall behavior.

REVIEW OF DESIGN APPORACH FOR BACK-TO-BACK WALL

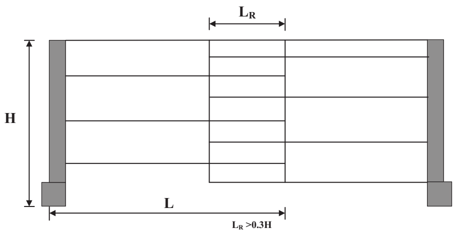
In the FHWA design guideline, two design cases are considered as shown in Fig. 1. Case 1 is that the width of wall is large enough so that there is no overlapping of the reinforcements. Case 1 is also divided into three sub-categories based on the distance between the back of reinforced zones of opposing walls D in comparison with the interaction distance $D_i = H \tan\left(45° - \frac{\phi}{2}\right)$, as below.

- Case 1-1: $D \geq D_i = H \tan\left(45° - \frac{\phi}{2}\right)$
  No interaction. Each wall is independently designed.
- Case 1-2: $D = 0$
  Two walls are designed independently for internal stability but with no active thrust to the reinforced zone.
- Case 1-3: $0 < D < D_i$
  Interaction between the opposing walls needs to be considered and it is suggested to linearly interpolate the active thrust to zero.

Case 2 involves those in which there is an overlapping of the reinforcements such that the two walls interact. In this case, it is assumed that no active earth thrust from the backfill needs to be considered for external stability calculations when the overlap, $L_R > 0.3H$. In this case L/H ratios for each wall as low as 0.6 can be adopted.



(a) Case 1



(b) Case 2

Fig. 1 Design cases of back-to-back walls.

However, the above criteria have no sound scientific justification and much still needs to be investigated to establish a more sound design approach. In this paper the results of a numerical investigation using the discrete element method on the failure mechanism of geosynthetic-reinforced back-to-back walls are presented. This study was conducted as part of an investigation aiming at validating assumptions for back-to-back walls adopted in the FHWA design guideline.

DISCRETE ELEMENT ANALYSIS

A series of discrete element (DE) analysis was carried out to investigate failure patterns of back-to-back walls with different geometries and the reinforcement lengths. Details of the discrete element analysis and the results are presented under the subsequent subheadings.

Discrete Element Method (DEM)

The discrete element method introduced by Cundall and Strack (1979) is increasingly popular for the study of the mechanics and behavior of granular materials (Zhang and Thornton 2007, Zhao and Evans 2009, Vinod et al. 2011, Lee and Schubert 2008, Bhandari and Han 2009, Funatsu et al. 2008, Chen et al. 2011).

In this study, the particle flow code (PFC^{2D}) was used, which is a commercial DEM software developed by Itasca Consulting Group Inc. (Itasca 2004). In PFC^{2D} particles are considered to be connected y springs, visco-pots, slides and couplers (Fig. 2). Unlike other continuum codes, PFC does not require mesh generation and only requires two elements, a wall and a particle. Details of the PFC can be found elsewhere (Itasca 2004).

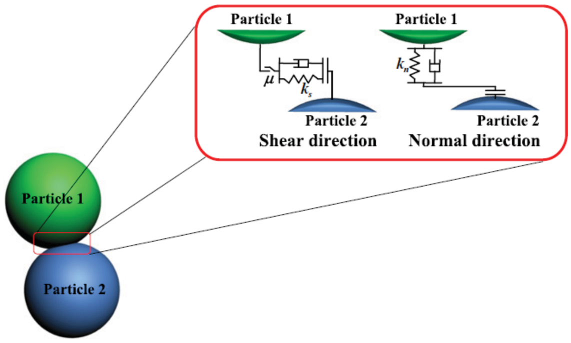


Fig. 2 Illustration of actions between particles (after Chen et al. 2011)

Cases Analyzed

In this study, 0.8 m high reduced scale, back-to-back walls having a width to height ratio of $W = 1.25H \sim 3.75H$ was considered. Note that reduced scale walls were considered in order to carry out the analyses with reasonable computation time. A number of relevant cases were developed considering various width (W) and reinforcement length (L) as summarized in Table 1. In all cases, it was assumed that three layers of reinforcement with a tensile strength of $T_m = 1.4\times10^{-2} kN/m$ are installed at a 24 cm vertical spacing. The tensile strength was determined using the similitude law assuming that a rupture strength of 55 kN/m are used in a 8 m full-scale wall. The model wall was assumed to be backfilled with a fine sand at a relative density of 70%, giving an internal friction angle of 40 degrees.

Table 1 Cases analyzed.

Case	W	L
Case 1	1.25H	0.0H, 0.44H, 0.63H
Case 2	2.50H	0.44H, 0.63H, 0.88H
Case 3	3.75H	0.5H, 0.7H, 0.88H

Discrete Element Modeling

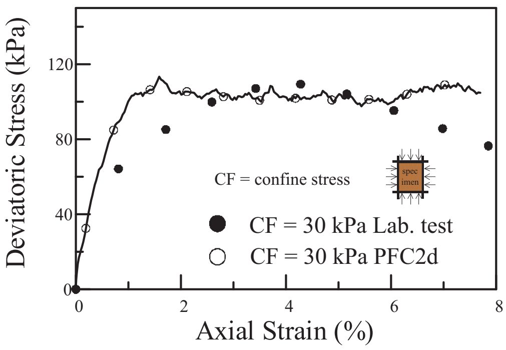
For modeling of the various components of the geosynthetic reinforced soil wall, various constitutive models were used. For example, the backfill material was modeled using the stiffness model as well as the slip model assuming that the model ground does not have cohesion (Chareyre and Villard 2003). For the reinforcements and the wall facing, the bonding model was used. For modeling of the wall facing, the parallel bond model was used while the reinforcement layers were modeled by a chain of circular particles by assigning contact bond between particles horizontally at predetermined locations. Note that contact bond can be envisioned as a kind of glue joining two particles and can only transmit a force (Itasca 2004).

The micro parameters of the materials were determined through a calibration process. In this study plane strain compression (PSC) tests were used to calibrate the micro parameters of the backfill material. PFC^{2D} analyses were repeated to match with the results of PSC tests on specimens prepared at the same relative density of the backfill. Table 2 summarizes the calibrated micro-parameters of the backfill. The results of the PSC tests and the PFC^{2D} simulations are compared in Fig. 3 for different confining pressures. As noted, a good agreement can be observed. For the reinforcement, the contact normal stiffness of $5.0 \times 10^{-2}\, kN/m$ was assigned considering the mechanical properties of the model reinforcement.

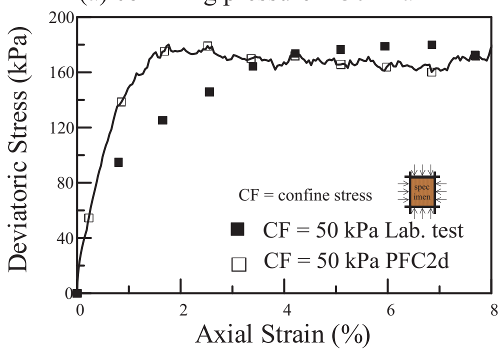
Table 2 Micro-parameters of backfill

$\gamma_s\ (kN/m^3)$	$k_n\ (MN/m^2)$	k_n/k_s	μ
15.5	200	1.2	0.6

Note: k_n =contact normal stiffness, k_s =contact shear stiffness, μ =friction coefficient of particle contact



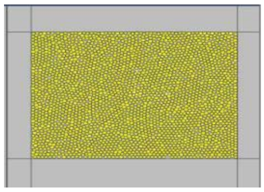
(a) confining pressure = 30 kPa



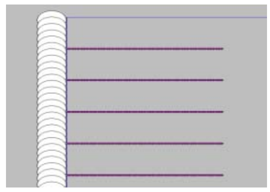
(b) confining pressure = 50 kPa

Fig. 3 Comparison between tested and computed for PSC tests

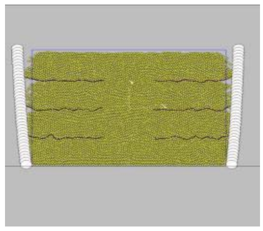
The PFC^{2D} simulation includes a number of step as shown in Fig. 4. Side walls were first created. Uniform sized cylindrical particles of 2.0mm in diameter were then generated at a porosity of 0.16 to create the model ground, the reinforcement, and the wall facing. Compaction of the assemblies was followed to achieve relevant density at a strain rate of 10^{-5}/s using isotropic compression. Upon completion of the wall construction, the side walls were removed to allow for the wall to deform. Gravity was then applied to induce failure. Figure x illustrates the simulation procedure.



(a) wall and ground creation



(b) wall facing and reinforcement creation



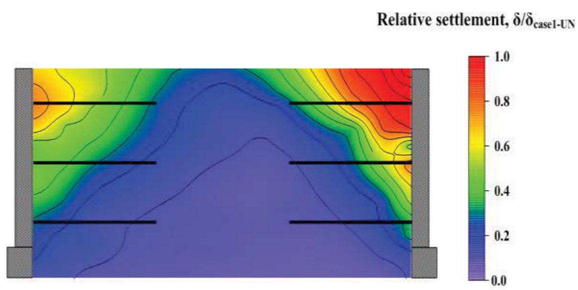
(c) turning on gravity

Fig. 4 Simulation process

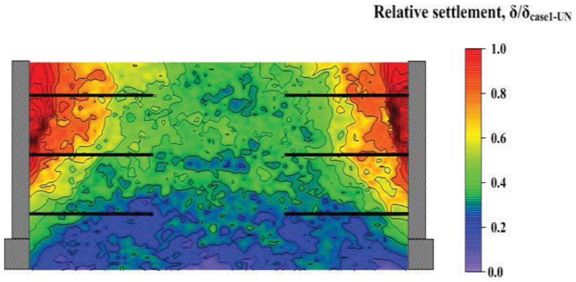
RESULTS AND DISCUSSIONS

The DEM modeling approach for the back-to-back wall was first validated by comparing the DEM results with those of the reduced scale model tests on 8 m high back-to-back walls having a width to height ratio of $W/H = 1.25$. Details of the model tests can be found in Yoo et al. (2011).

In Figs. 5 and 6, comparisons are made between the model test and PFC^{2D} in terms of the contour plots of the displacement field at the ultimate state. Note that the displacement contour plots for the model tests were obtained from the Particle Image Velocimetry (PIV) analysis using GeoPIV (White and Take 2002). Note here that the displacements are normalized using the respective maximum value for ease of comparison. As can be seen in these figures, the results compare fairly well each other, warranting that the PFC^{2D} analysis could be extended to different cases.

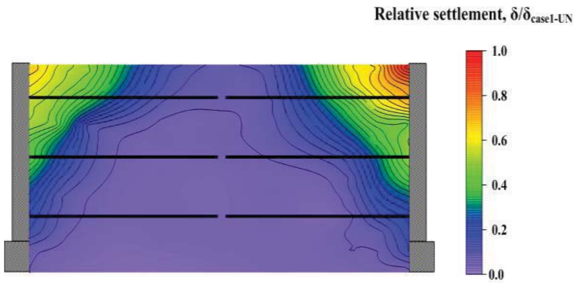


(a) model test

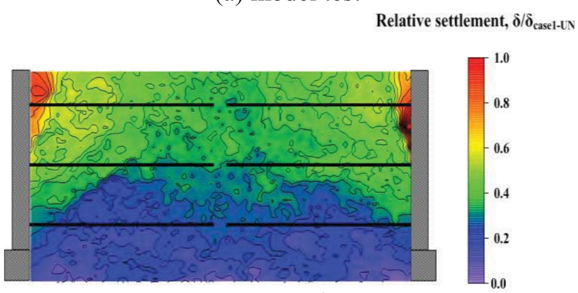


(b) PFC2D

Fig. 5 Displacement contour plots ($L = 0.35W$)



(a) model test

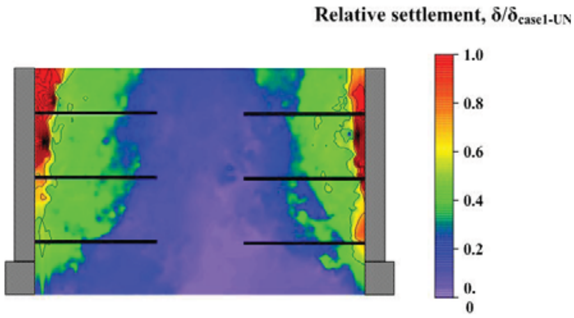


(b) PFC^{2D}

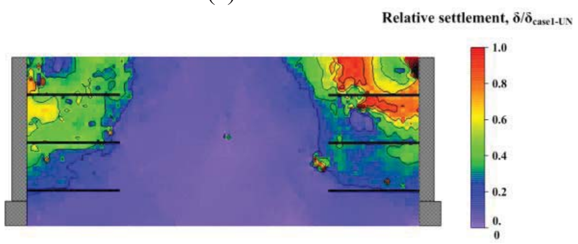
Fig. 6 Displacement contour plots ($L = 0.5W$)

Figure 7 shows the displacement contour plots for cases with different back-to-back wall widths with a constant reinforcement length of $L = 0.5H$. As can be noticed, it is seen that for cases of $W = 2.5H$ and $3.75H$ the failure surfaces in two opposing walls do not seem to intercept each other, suggesting that two opposing walls behave independently. For $W = 1.25H$, however, the active zones do not seem to fully develop, suggesting that the critical failure surfaces intercept each other. Considering that the interaction distance D_i for the back-to-back wall with an internal friction angle of $\phi = 40°$ given by FHWA is $0.47H$, such a result supports the FHWA assumption.

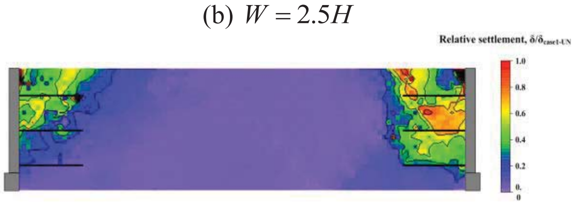
The variation of the displacement field with the reinforcement length L for cases with the back-to-back wall width of $W = 1.25H$ is shown in Fig. 8. Three levels of the reinforcement length were considered, i.e., $L = 0.5H$, $0.63H$, and $0.88H$. Note that $L = 0.63H$ yields $D = 0\ m$ in which there is no retained fill between two opposing walls. The case with $L = 0.88H$ falls into Case 2 in the FHWA design assumption as the reinforcement overlap is $L_R = 0.51H > 0.3H$. Therefore no active earth thrust from the backfill needs to be considered for external stability calculations and L/H ratios for each wall as low as 0.6 can be adopted. As can be seen in this figure, the active zones are not fully developed for $L = 0.63H$ and $0.88H$.



(a) $W = 1.25H$

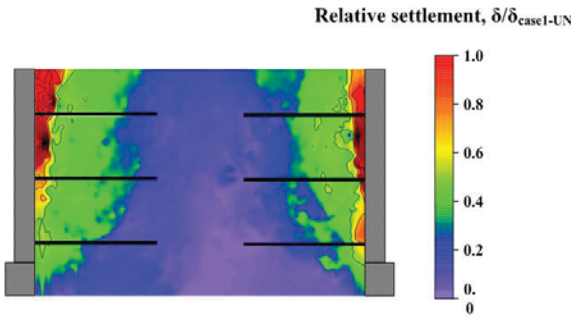


(b) $W = 2.5H$

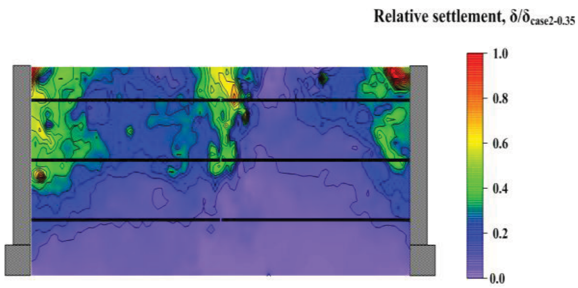


(c) $W = 3.75H$

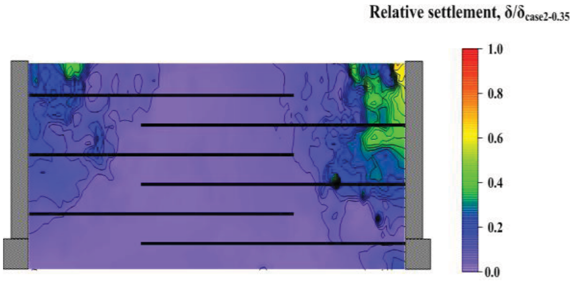
Fig. 7 Variation of displacement field with W



(a) $L = 0.5H$



(b) $L = 0.63H$



(c) $L = 0.88H$

Fig. 8 Variation of displacement field with L

CONCLUSIONS

In this paper, the results of an investigation into the failure mechanism of geosynthetic reinforced soil wall in back-to-back wall configuration are presented. The DEM modeling approach was first validated using the results from 1-g reduced-scale model tests. A series of discrete element (DE) analyses were then carried out on back-to-back walls with various widths to height ratios reinforcement distributions.

The results indicated that the DEM approach can be effectively used for investigating failure mechanisms of geosynthetic reinforced soil walls with complex geometry. Also found was that the current FHWA design assumptions for back-to-back walls are reasonable in terms of the adopted failure mechanisms although further studies are warranted.

ACKNOWLEDGEMENTS

This research is supported by Grant No. 20100008227 from the Basic Research Program of the Korea Science & Engineering Foundation. The financial support is gratefully acknowledged.

REFERENCES

Bhandari A. and Han J. (2009). DEM Study of a shallow foundation under a vertical loading. Proc. Intl. Foundation Congress and Equipment Expo.:465-472.

Chareyre B. and Villard P. (2003) Discrete element modeling of curved geosynthetic anchorages with known macro-properties. Proc. Numerical Modeling in Micromechanics via Particle Method: 197-204.

Chen R.P., Tang L.J., Ling D.S., Chen Y.M. (2011). Face stability analysis shallow shield tunnels in dry sand ground using the discrete element method. Computers and Geotechnics. 38:187-195.

Cundal P.A. and Strack O.D.L. (1979) A discrete numerical model for granular soils. Geotechnique 29(1):47-65.

Elias V. and Christopher, B.R. (1997). Mechanically Stabilized Earth Walls and Reinforced Soil Slopes, Design and Construction Guidelines. FHWA Demonstration Project 82, FHWA, Washington, DC, FHWA-SA-96-071.

Funatsu T., Hoshino T., Sawae H., and Shimizu N. (2008). Numerical analysis to better understand the mechanism of the effects of ground supports and reinforcements on the stability of tunnels using the distinct element method. Tunnelling and Underground Space Technology. 23:561-573.

Han J. and Leshchinsky D. (2010) Analysis of back-to-back mechanically stabilized earth walls. Geotextiles and Geomembranes. 24:262-267.

Itasca Consulting Group, Inc. (2004) Particle Flow Code (PFC^{2D}), User's Guide, Itasca Consulting Group, Minneapolis.

Lee Y.Z. and Schubert W. (2008). Determination of the round length for tunnel excavation in weak rock. Tunnelling and Underground Space Technology. 23:221-231.

Yoo C. and Kim S.B. (2008) Load carrying capacity of back-to-back reinforced soil walls. Korean Geotechnical Society. 24(12):41-52.

Yoo C. and Jeon H.Y. (2010) Long-term behavior of geogrid reinforced soil abutment – a numerical investigation. of Korean Geotechnical Society. 27(1):65-76.

Yoo C., Woo, S.J., Jeon, H.M., and Shin, B.N. (2011) Investigation on failure mechanism of back-to-back geosynthetic reinforced wall using discrete element analysis. Korean Geosynthetics Society. 10(2):55-66.

Vinod J.S., Nagaraja S., Sitharam T.G., and Dinesh S.V. (2011). Numerical simulation of reinforced granular soils using DEM. Proc. Geo-Frontiers. Dallas,U.S.A: 4242-4251.

White D.J. and Take W.A. (2002) GeoPIV: Particle Image Velocimetry (PIV) Software for Use in Geotechnical Testing, CUED/D-SOILS/TR322.

Zhang L. and Thornton C. (2007). A numerical examination of the direct shear test. Geotechnique. 57(4):343-354.

Zhao X. and Evans T.M. (2009). Discrete simulation of laboratory loading conditions. Intl. Geomechanics. ASCE. 9(4):169-178.