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A Study on Interaction Behaviors of Soil-PET Mat installed on Dredged Soils

연약한 준설점토상 매립시 포설된 PET 매트와 지반거동에 관한 연구

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요 지

토목섬유를 토목공학에 적용한 이후 토목섬유의 손상에 대하여 많은 관심이 되어왔다. 이 논문에서는 시험시공을 3지점에서 수행했는데 목적은 연약한 점토지반상에서 보강재의 거동을 조사하고 매립시 최적의 시공방법론을 찾기위한 것이다. 이 시험시공에 사용된 폴리에스테르매트(인장강도 15톤)의 봉합부분이 원호활동 사면파괴로 인하여 파괴되었는데 이는 급격한 성토로 인해 과잉간극수압이 발생하여 하중이 증가했기때문이다. 매트의 인장 파열이 진행되는 지반 거동 중 성토고로 유발될 수 있는 간극수압보다 큰 간극수압이 측정되었는데, 특히 지표하 5m 깊이에서는 장기간에 걸쳐 간극수압이 증가하였다. 이러한 결과를 이용하여 성토고와 연약점토의 융기거동의 관계에 대하여 검토하였다.

Abstract

Geosynthetic damage has attracted a major attention since the introduction of geotextiles for civil engineering applications. In this study 3 pilot trial embankments were carried out to investigate the behaviours of reinforced embankments over soft cohesive soils and to find the optimum methodology of embankments over soft soils. As the seamed part of polyester mat (PET, tensile strength 15 ton) used in the first full-scale field test was ruptured under progressing rotational slope failure because of unexpectedly rapid construction of embankments, the excessive pore water pressures were measured. On the soil behavior where tension explosion of mat was continued, pore pressure larger than the one caused by embankment height was measured. Especially, at the depth of 5.0 m under the ground pore pressure increased over long term. It was discussed with respect to the height of embankment and heaving behavior of soft soils.

Keywords : Embankment, Geosynthetics, Heaving, PET mat, Pore water pressure, Soft soils

1. Introduction

The behavior of geosynthetic reinforced embankments over soft soil has attracted considerable attention in both practice and literature (R.K. Rowe, 2002).

For about half a century geosynthetics has served successfully as separators, filters, reinforcement, drainage medium and protection layers in a countless number of projects. Among the large number of successful applications there are, however, examples where geosynthetics has not fulfilled its intended purpose due to damage created during installation.

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Watn et al. (2002) detailed damages during installation such as the abrasion caused by a repeated sliding action from an abrasive material on the geosynthetic, splitting, puncturing, fiber cutting locally by sharp soil materials or equipments, the stress rupture occurring when the geosynthetic is exposed to large loads and deformations.

Pinho-Lopes (2002) emphasized durability of geo-sythetics, and the effect of damage on their mechanical behavior during installation was observed through laboratory and field damage test. Also, the reduction factors for creep and damage during installation were proposed to prevent some overdesign of the materials.

Jewell (1998) proved by numerical analysis that three failure mechanisms were classified for the lateral sliding increasing horizontal displacement, bearing capacity, stability by geosythetic reinforcement, the bearing capacity failure, and the rotational slope failure, in which reinforced materials were ruptured and extracted accompanying large deformation.

Dessicated surface layers were formed by Progressive Trench Method (PTM) on marine ultra soft clay layers with reclaimed and dredging clay in Yulchon 1st industrial construction site, where step banking was performed for site renovation. When shear failure or depression happened by embankment loading and construction equipment, work was difficult and efficiency dropped.

However, it is judged that the specific research and construction data on dry dessicating clay layers and ultra soft clay layers by embankment load and equipment acting, are insufficient. Also, auxiliary method is needed to use the effective operation of heavy equipment while banking.

Full-scale test embankments, where temporary road preferentially was enforced, were carried out using rubble aggregate and EPS concrete block to presume the dessicated thickness and representative shear strength, more important factor in a bearing capacity calculation, and to secure the stress change and effect of load and trafficability.

In this paper, on the case the polyester (PET, tensile strength 15ton) mat rips, which happened in Yulchon 1st industrial construction site during pilot trial, through the analysis of embankment loading, behavior of lower layers, field instumentation etc. the embankment behaviors and roles of PET mat are discussed.

2. Applications of Geosynthetics

2.1 Behaviors of Geosynthetics

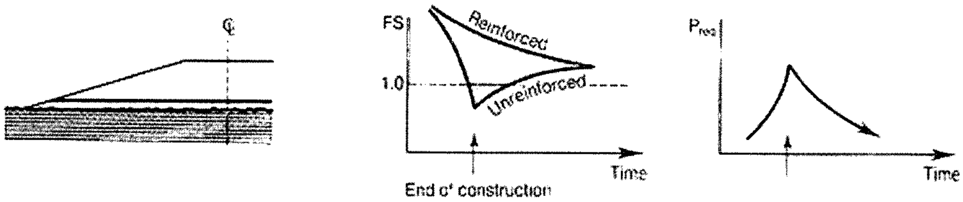
The roles of geotextiles as separators under the condition of intermixing of poor in-situ soils with good quality granular materials subject to surface loading, provide a barrier to migration of particles between the two disimilar soils but allow the free transmission of water. One requirement for a separator is a sufficent tensile strength to maintain continuity and not to rip, puncture or burst under the local stress concentrations (Fig. 1 (a)).

The soil reinforcement function of geotextiles is the mechanical improvement of embankment to resist horizontal earth pressure and displacement (Fig. 1 (b)). The inclusion of reinforcement can allow a steeper slope or embankment to be built and also allow greater load to be applied by external loads, such as in an unpaved road or working platform.

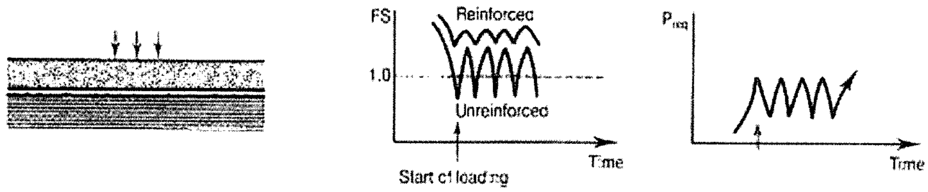
The reinforcement resists loading which comes mainly from the self-weight of the soil and the geometry of the structure applied to slope. In an embankment on soft soil, the reinforcement is only required to boost stability during the

(a) separation (b) reinforcement

Fig. 1. Geotextile functions



(a) embankments on soft soil



(b) unpaved roads

Fig. 2. Variation of required reinforcement force with time

critical period of construction and subsequent foundation consolidation. Once the foundation soil has increased in strength the reinforcement is no longer required to ensure stability.

Fig. 2 (a) describes variation with time of the factor of safety, with and without reinforcement, and correspondingly requiring reinforcement force. Unpaved roads and the working platforms are the main applications where geotextile reinforcement boosts the resistance to external loads, and the required tensile force in the reinforcement responds to the repeated loads.

When geotextiles loaded is torn during installation or construction, it has close connection with initial tensile strength of geotextiles. The criteria about survivability of geosynthetics may make up strain energy and residual stress and failure strain from installation to service life. It is proposed under the assumptions that geosythetics transformed as soft lower layer. Principle on energy and strain is presented in Fig. 3.

When geotextiles are under repeated live loading on the process of the opening and closing of tensile cracks, tensile reinforcement can reduce or disperse localized cracking (Fig. 4).

2.2 Shear Characteristics of Geosynthetics

The shear characteristics of geotextiles may approach to rupture after yield point in constant rate of elongation

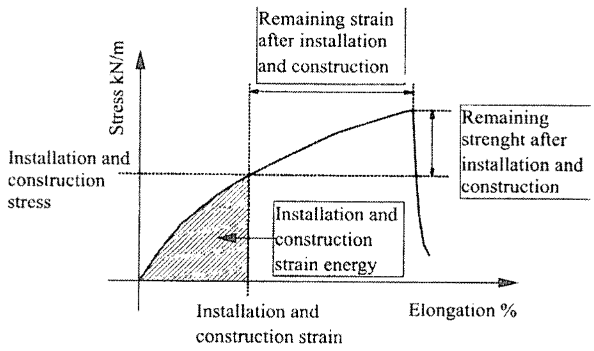
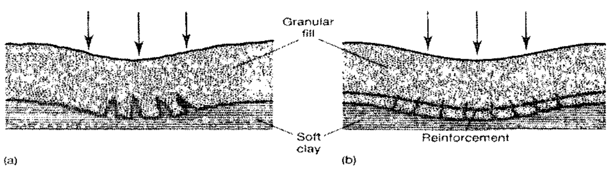
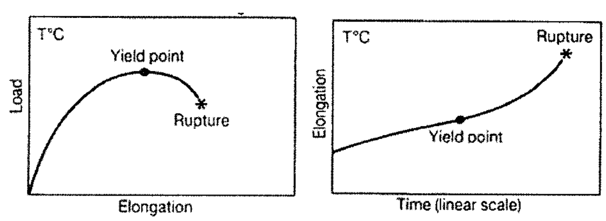


Fig. 3. Survivability criterion principle



(a) unreinforced case (b) reinforced case

Fig. 4. Local tensile cracking at the base of a loaded granular layer



(a) constant rate of elongation test (b) sustained load test

Fig. 5. Yield and rupture points

test as shown in Fig. 5 (a). Under a sustained load the yield point is defined where the rate of elongation reaches a minimum (Fig. 5 (b)). Allowable design load of geosythetics is determined under allowable maximum elongation condition in which reinforced earth structures is not over-deformed.

2.3 Analysis of Geosyntheics

Due to their fibrous structure, geosynthetics have significant tensile axial stiffness but very low resistance to bending. They can sustain, by mobilization of their tensile strength, either directly applied tension along their axis (Fig. 6 (a)), or through membrane effect, indirectly applied tension to their surface (Fig. 6 (b)). More rarely, their rigidity under compression leads generally to the formation of folds (Fig. 6 (c)).

Villard etc. (2002) compared the results of 3-dimensional modelling analysis and 2-dimensional plane strain condition with laboratory model test results, but did not represent complex behaviors of full-scale field.

3. Pilot Trial and Analysis of Instrumentation Results

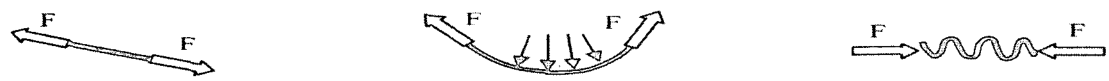
3.1 Outlines of Pilot Trial

Pilot construction was done at the Yulchon 1st industrial construction complex of Jeollanamdo, where marine clay layers (undrained shear strength (Su) ranged 1-2 t/m^2) are distributed to 6-10 m depth.

Temporary dock was set up, where dredging clay was reclaimed by 6 m depth, and formed. The scope of construction is that dry dessicated layer of about 60 cm thickness by applying PTM was formed and banking was performed for ground improvement. So, the objective of pilot trial is to select effective and economical banking method. Area of test construction site is 90 x 100 m, which were composed of about 10,000 m^3 mountain soil and 15 ton PET mat of 14,000 m^2. Position and overview of site are shown in Fig. 7.

Fig. 8 shows the staged construction height and instrumentation layout, installation locations. Doble wrikle pipes were set up in two rows (Line 1,2) and one rank (Line 3) and while banking, deformation of original clay layer was monitored periodically, where profile gauge should go through on the upper part of PET mat. Piezometers were installed in 1 m, 3 m, 5 m depth each, on a twofold row (Line 1).

Average shear stength distribution at 60 cm depth of dry dessicated layer in pilot trial site is as shown in Fig. 11 (a), undrained shear strength measured to 1.37 t/m^2,



(a) Traction (b) Membrane effect (c) Compression

Fig. 6. Types of behaviour of a geosynthetic reinforcement sheet

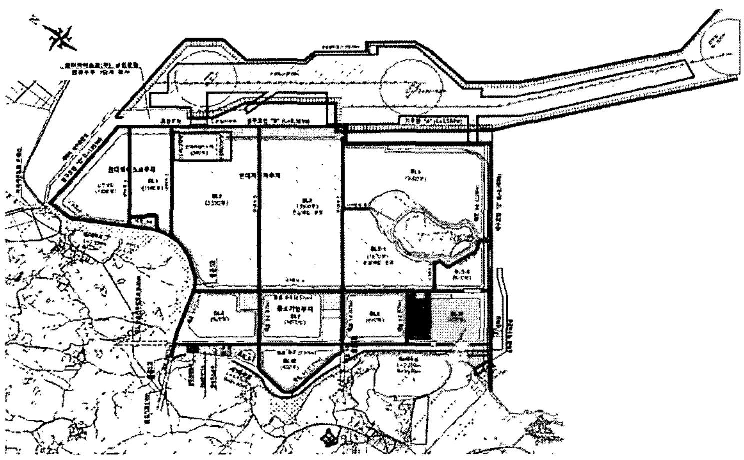


Fig. 7. Location of pilot trial

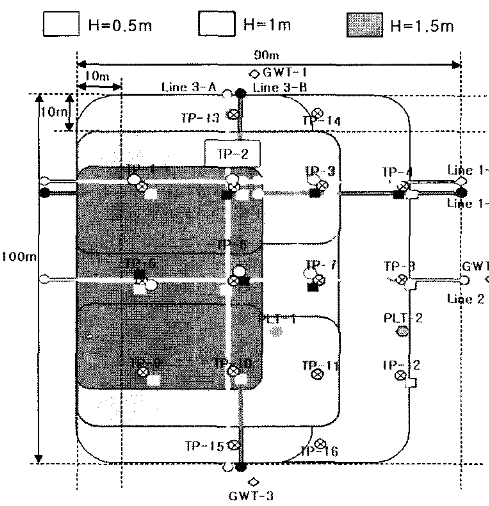


Fig. 8. Plan of pilot trial

maximum value at first part of upper line of banking (line 1). As shown in Fig. 11 (b), Average water content at surface layer of 60 cm ranged 60-80%, value is smallest in the center. Embankment materials used in pilot trial are about 8% of #200 passing, classified as GW-GM by USCS, 22° of internal friction angle, 0.55 t/m^2 of cohesion (Table 1).

Polyester mat (tensile strength 15 ton) used in pilot trial was tested by KS K 0210, 0530, 0743, whose tensile strength is 470.6 kgf (cross machine direction, tensile strain 20%). Fig. 12 (a) shows specimen of Grab method and dimension of clamp. Fig. 12 (b) shows tensile test shapes, and Fig. 12 (b) shows ruptured features of specimen when strain was concentrated on clamp after tensile test finishes. PET mats were set up in a banking direction with overlapped needlework suture (J-SSn6) in field.

3.2 Results and Analysis of Instrumentation

Figs. 13 (a) and (b) show behaviors of embankment thickness and lower ground layer measured by profile

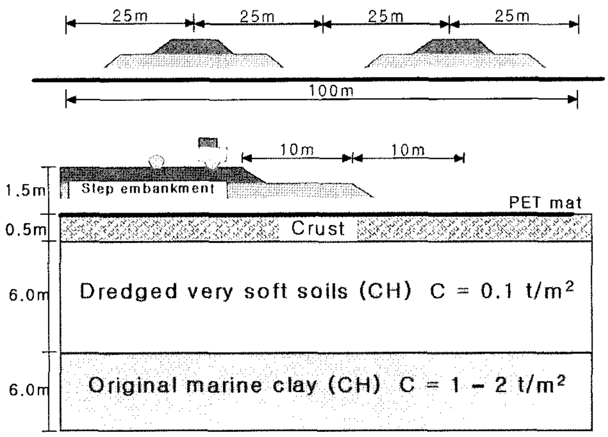


Fig. 9. Soil profiles of pilot trial

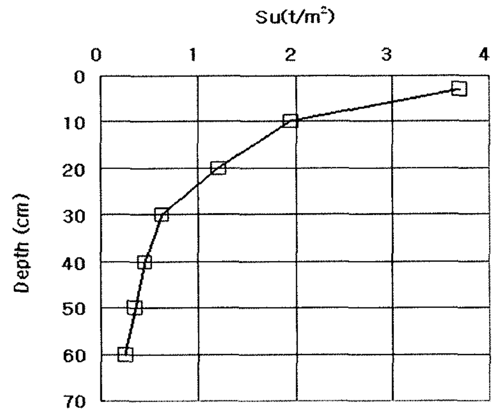
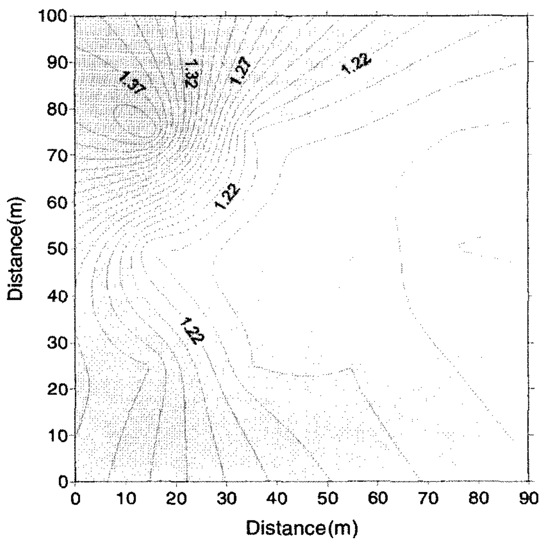
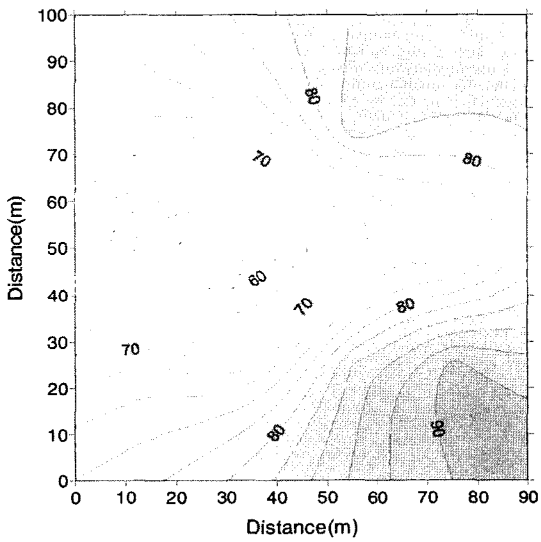


Fig. 10. Depth-shear strength (dessicated layer)



(a) Average shear stength at surface layer 60 cm



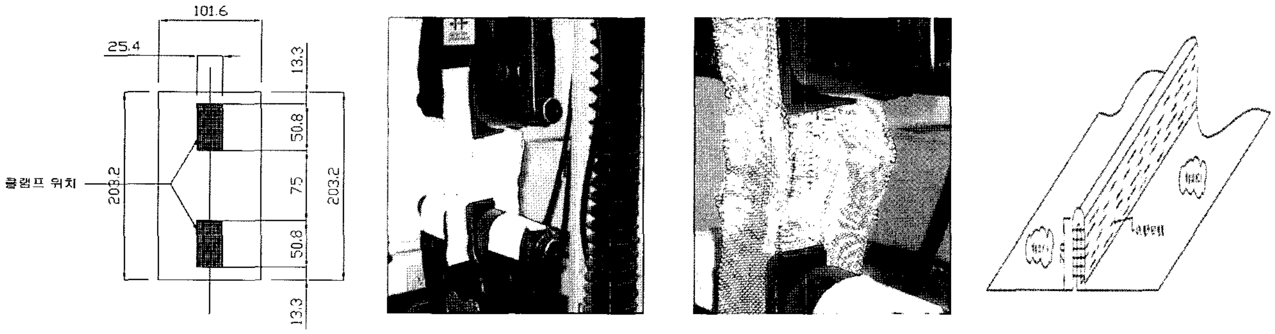
(b) Average water content at surface layer 60 cm

Fig. 11. Average shear stength and water content distribution of dry dessicated layer in pilot trial site

Table 1. soil properties

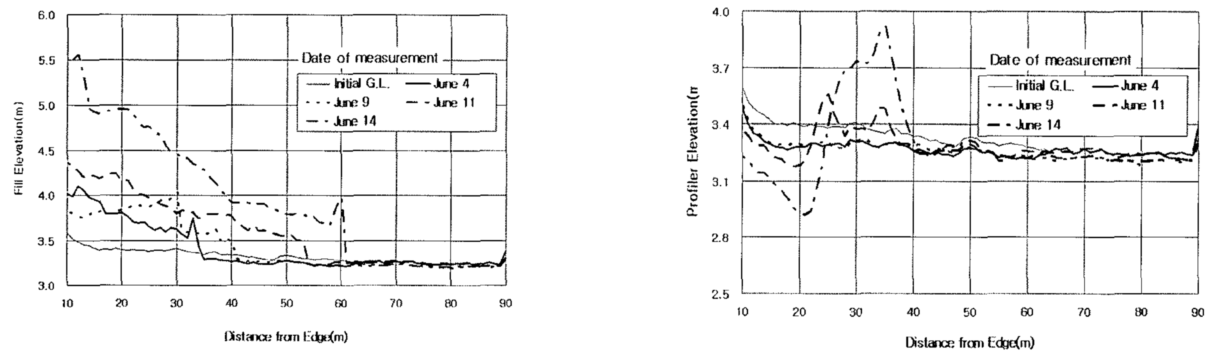
Gs	wn (%)	#200 passing (%)	LL (%)	PL (%)	∅ (°)	c (t/m^2)	USCS
2.698	14.5	7.8	N.P	N.P	22	0.55	GW-GM

gauge from surcharge start to June 14. While banking progressed uniformly by designed thickness at initial time of embankment, the management of equipment did not match the supply the of banking materials after banking duration of 10 days. Finally height at start point of embankment increased by 2 m. On June 14, banking has rapidly performed to 20 m at that point. Large heaving took place after one day at TP-2. It is judged that considering ground height, mat was ruptured for heaving. So, construction was stopped to remove and investigate embankment soil. From the center heaved, the seamed part of mat has torn and got wider by about 15 cm, then



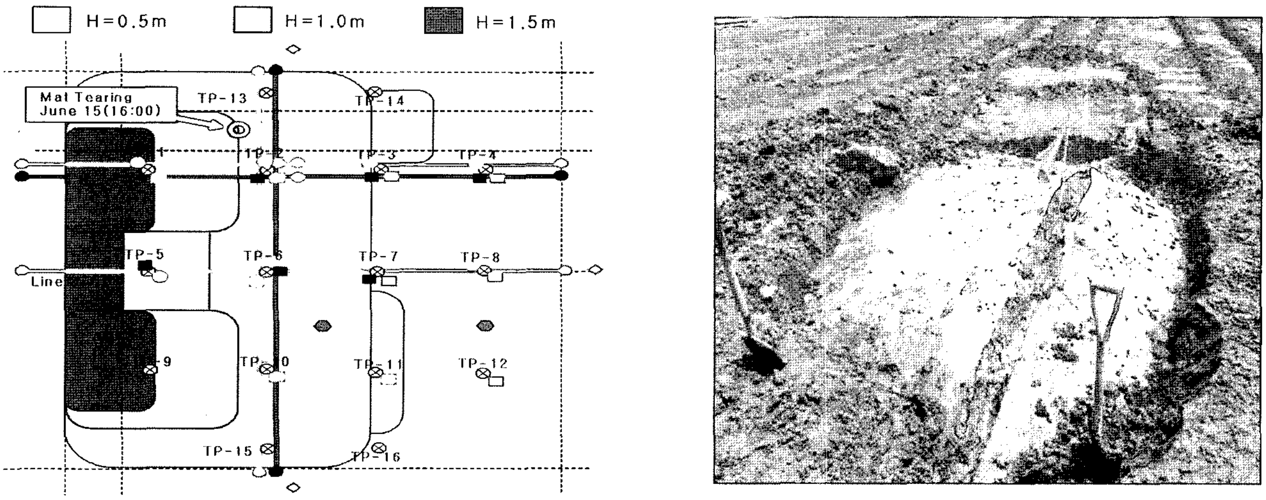
(a) mat test specimen (b) tensile tester (c) specimen after tensile test (d) needlework suture (SSn6)

Fig. 12. PET mat tensile test (15t) and 'J'-type field needlework suture



(a) Surcharge height (b) Ground deformation

Fig. 13. Surcharge height and ground deformation measured by profile gauge (Line 1)



(a) Location of shear rupture (b) Photograph of torn PET mat

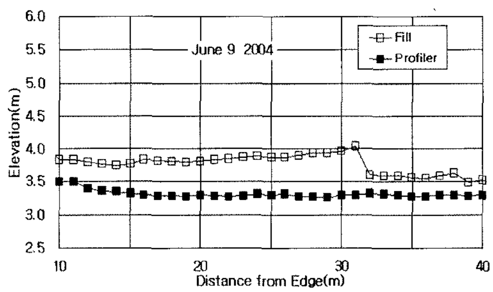
Fig. 14. Torn mat while construction after ground heaving (2004 June 15 (surcharge duration=9days))

reclaimed clay was displayed (FIg. 14).

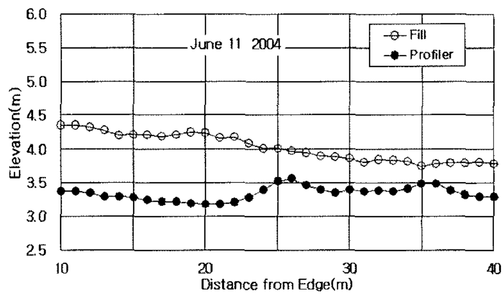
Fig. 15 shows comparisons of surcharge height and ground deformation measured by profile gauge while banking was progressed for 7days from June 9. About 1 m thick banking at start point of embankment on June 11 caused irregular settlement and heaving at the front of 50 cm banking. It is judged that on June 14, dumping was progressed thicker than design height of about 22 m thick inducing ground deformation of about 1 m heaved at the sector of 50 cm banking thickness. Finally on June 15 shear failure happened. On June 15, nearly damaged mat was excavated, and banked again with the width of 1 m after PET mat was installed with overlapping.

Fig. 16 shows results of groundwater level measured in 3 spots of outer pilot trial sector. It is shown that groundwater level at each location kept constant even for 2days (from June 14 to June 15) when rupture happened. Fig. 17 shows results of surface settlement plate at 2 locations: the one is at TP-2 where failure happened, another is TP-7 where rupture did not happen. At TP-2, heaving of about 10 cm was monitored just on the day shear failure took place, but, at TP-7 it did not happen, settlement kept constant.

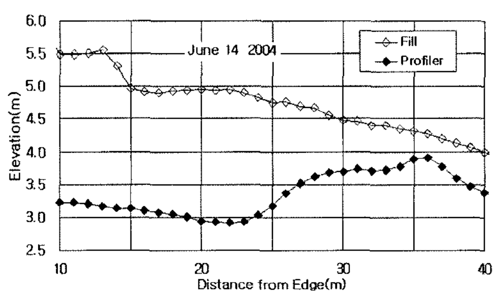
Fig. 18 shows results of pore pressure measured from underground depth of 1 m, 3 m, 5 m at TP-2 and TP-7. While observing changes of pore pressure from June 14 to June 15, rupture may happen. At TP-2, pore pressure value increased very insecurely. Pore pressure at the depth of 5 m at TP-2 increased sharply. It is judged that the unsettled increase of pore pressure from June 14 to June



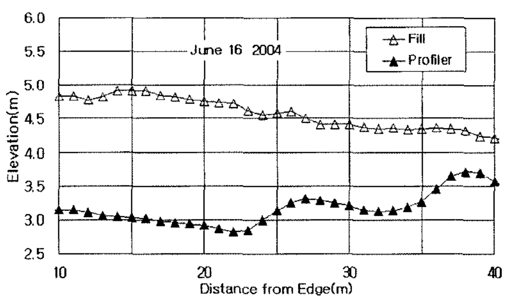
(a) June 9



(b) June 11



(c) June 14



(d) June 16

Fig. 15. Surcharge height and ground strain as time depending

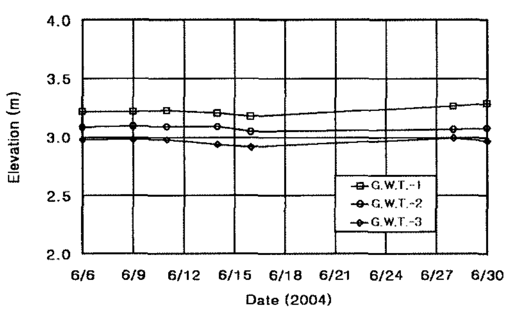


Fig. 16. Results of groundwater level

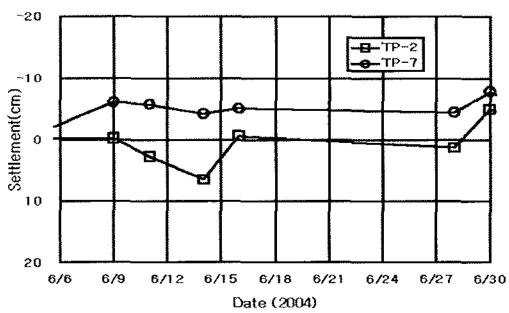
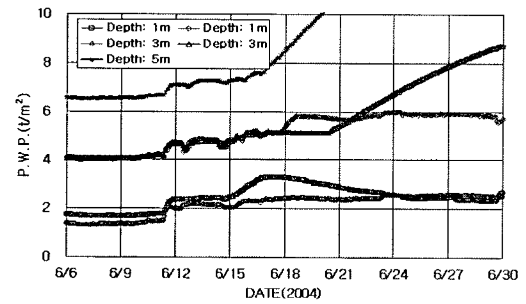


Fig. 17. Results of settlement plate

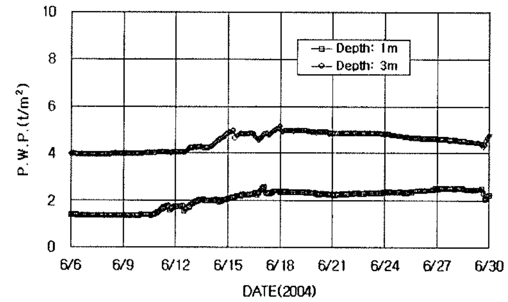
15 was converted to be stable and decreased slowly after June 16 embankment was recontrolled. Even at TP-7, the pressure finally kept to be 0.5 t/m^2 which is induced from surcharge.

Fig. 19 shows the surcharge height and ground deformation measured from June 16 to July 30 when last surcharge was completed. By measurement results of profile gauge until July 30 under banking, surcharge height had the close relation with settlement and heaving happened repeatedly.

Until embankment completion when the process of ground deformation from 50 m of height is observed, complex behavior of ground was measured. From near 60 m of surcharge height to upper part of embankment, just a little ground deformation value of about 10 cm was monitored. After June 16, when both design embankment height and embankment velocity were controlled and managed, shear failure did not happen any more.

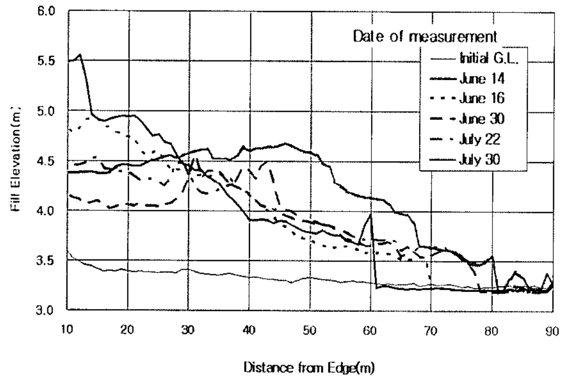


(a) TP-2

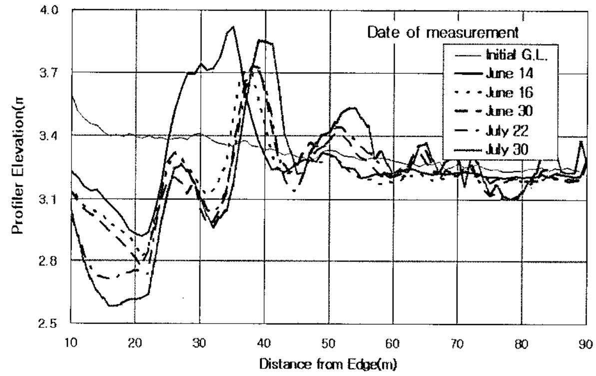


(b) TP-7

Fig. 18. Changes of pore pressure at 1 m, 3 m, 5 m depth of TP-2 and TP-7 location



(a) Surcharge height



(b) Ground deformation

Fig. 19. Results of Profile Gauge measurements of pilot trial (Line 1)

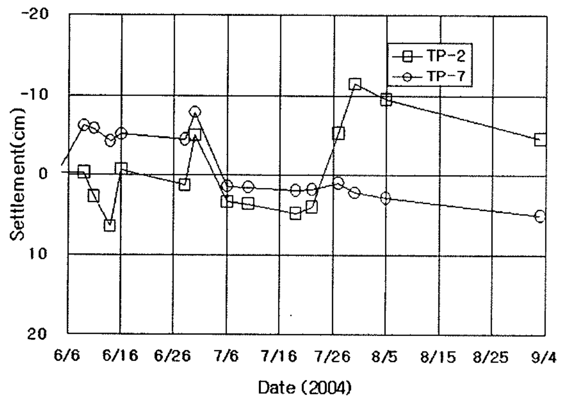


Fig. 20. Results of settlement plate

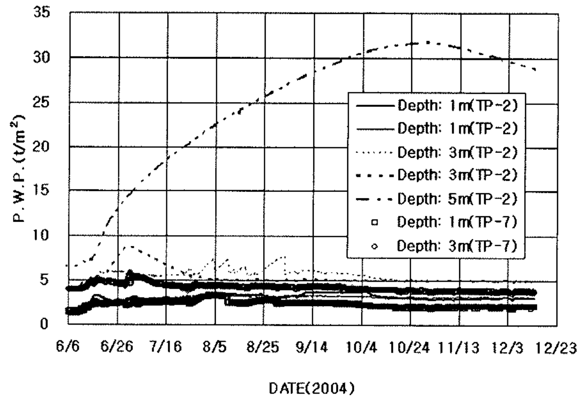


Fig. 21. Changes of pore pressure

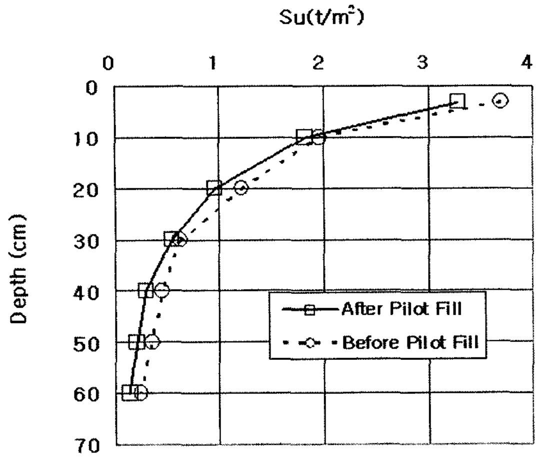


Fig. 22. Shear strength of dry dessicated surface layer after banking completion

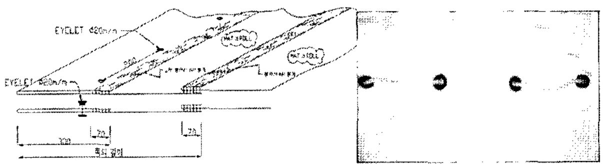


Fig. 23. Indoor overlapped needlework suture using rope

Fig. 20 shows results of settlement plate measured from surcharge start to surcharge duration of 30days at TP-2,TP-7. At TP-7, behaviors of settlement totally were measured, but just a little heaving was monitored smaller than those monitored on June 15, July 25, and July 30 when last sucharge is completed at TP-2. After the completion of last sucharge, settlements happened at all locations of TP-2, TP-7.

Fig. 20 shows results of excess pore pressure measured from surcharge start to mid-December at TP-2, TP-7. At TP-7. At TP-7, pore pressure during surcharge slowly decreased as time goes by, but at TP-2 it was different, where pore pressures at the depth of 1 m and 3 m dissipated, but that at the depth of 5 m increased for 4 months and slowly dissipated.

Fig. 22 describes the relationship between undrained shear strength and depth of dry dessicated surface layer after banking completion. When undrained shear strength of dry dessicated surface layer decreases after banking, it is judged by effects of disturbance that as embankments progressed, water content of dry dessicated surface layer, and loads of banking and equipments repeated.

Main causes of polyester mats, which were torn at suture parts, come from field overlapped needlework suture that did not confirm suture strength. Indoor overlapped needlework suture in Fig. 23 would be used. When it comes to the same polyester mat used in pilot trial, applied load is 504.9 ksf larger than tensile strength.

4. Conclusions

In this paper, on the case the polyester (PET, tensile strength of 15ton) mat rips happened in Yulchon 1st Industrial construction site during pilot trial, through the analysis of embankment height, soil behavior, site instumentation etc.. Conclusions are as follows.

While soil embankment progressed, settlement and heaving repeated nearly in the lower side of embankment according to height. When embankment progressed more rapidly than design planned, tension exploded spot needlework suture department, on the way of polyester mat's restricting soil behavior of heaving.

On the soil behavior which tension explosion of mat continued, pore pressure values larger than the one caused from embankment height were measured. Especially, depth of 5.0 m under the ground pore pressure increased over long term.

As behaviors of pore water pressure increase, effects of polyester mat become great.

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