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Reinforcing effect of vetiver (Vetiveria zizanioides) root in geotechnical structures - experiments and analyses

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Abstract. Vetiver grass (Vetiveria zizanioides) is being effectively used in many countries to protect embankment and slopes for their characteristics of having long and strong roots. In this paper, in-situ shear tests of the ground with the vetiver roots have been conducted to investigate the stabilization properties corresponding to the embankment slopes. Numerical analyses have also been performed with the finite element method using elastoplastic subloading t_{ij} model, which can simulate typical soil behavior. It is revealed from field tests that the shear strength of vetiver rooted soil matrix is higher than that of the unreinforced soil. The reinforced soil with vetiver root also shows ductile behavior. The numerical analyses capture well the results of the in-situ shear tests. Effectiveness of vetiver root in geotechnical structures–strip foundation and embankment slope has been evaluated by finite element analyses. It is found that the reinforcement with vetiver root enhances the bearing capacities of the grounds and stabilizes the embankment slopes.

Keywords: eco-engineering; FE analysis; slope stabilization; vegetation; vetiver root

1. Introduction

Recently, many countries are effectively using naturally grown vetiver grass (Vetiveria zizanioides) to protect embankment and slopes. Establishment of vegetation which is accepted all over the world due to its low-cost, longevity and environment friendliness may be a soft bioengineering technique to rigid or hard structures (Islam 2003). Vetiver grass applications also include soil and water conservation systems in agricultural environment, mine rehabilitation, contaminated soil and saline land remediation, as well as wastewater treatment (Erskine 1992, Truong and Baker 1998, Truong and Loch 2004, Verhagen *et al*. 2008).

Vetiver grass plantation has become popular as a bio-engineering technique due to its special attributes. It can grow on sites where annual rainfall ranges from 200 mm to 5000 mm (Rahman *et al*. 1996). It can survive in temperature ranging from -20°C to 60°C (Xu and Zhang 1999). It grows on highly acidic soil types (pH ranges from 3.0 to 10.5) and also tolerant to high content of Al, Mn, As, Cd, Cr, Ni, Pb, Hg, Se and Zn in the soil (Truong and Baker 1998). The saline threshold (EC) of vetiver is 7.8 dSm^{-1}. However, in soil with EC values of 10~20 dSm^{-1}, the yield

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of vetiver is reduced by 10%~50% (Truong and Baker 1998). Its roots are very strong with a diameter of 0.66 ± 0.32 mm having a high tensile strength of around 85.10 ± 31.20 MPa (Hengchaovanich 1998). It is also found that vetiver hedges can survive even for more than 100 years (Verhagen *et al*. 2008).

Many studies (Hengchaovanich 1998, Truong 1999, Ke *et al*. 2003, Dudai *et al*. 2006) have been performed to assess the performance of vetiver grass against climatic change, slope protection, coastal embankment protection and so on. Hengchaovanich (1998) analyzed slope stability based on vetiver root strength. Ke *et al*. (2003) tested vetiver as a bank protection measure on several test sites in Australia, China, Philippines and Vietnam. Their tests showed promising results for the use of vetiver grass as a bank protection measure. Verhagen *et al*. (2008) conducted different laboratory and model tests on vetiver grass to realize the use of it in coastal engineering and showed that vetiver grass is able to establish a full stop to bank erosion caused by rapid draw down. Mickovski *et al*. (2005) investigated uprooting resistance of vetiver grass. Mickovski and Van Beek (2009) determined the root morphology of vetiver and presented the effects of young vetiver on soil reinforcement and slope stability in semi-arid climate. Coppin and Richards (1990) presented the influences of vegetation on the stability of slope segment based on several factors such as enhanced effective soil cohesion due to soil reinforcement by roots, surcharge due to weight of vegetation, vertical height of groundwater table above the slip plane with the vegetation, tensile root force acting at the base of the slip plane, angle between roots and slip plane and wind loading force parallel to the slope etc. They showed that the vegetation increases the factor of safety of slopes by 55%. But, the calculations are purely illustrative and it needs to assume some parameters in their study.

Several studies were conducted to investigate the effectiveness of the vetiver grass plantation in slope stabilization (Van Beek *et al*. 2005, Cazzuffi *et al*. 2006, Mickovski and Van Beek 2009, Islam *et al*. 2013). Different tests were also conducted to determine the effectiveness of vetiver root in slope protection. In this paper, firstly the field test results have been simulated using the finite element analyses. Then, the effectiveness of vetiver root on the stability of slopes and bearing capacity of the reinforced ground is evaluated by finite element analyses, using the elastoplastic subloading t_{ij} model (Nakai and Hinokio 2004, Nakai *et al*. 2011). This model can describe the typical stress, deformation and strength characteristics of soils, such as the influence of the intermediate principal stress, the influence of stress-path dependency of the plastic flow and the influence of the density and/or the confining pressure.

2. Soil properties and parameters for modelling

2.1 Soil properties

As the coastal zone and flood plain zone are most vulnerable to natural hazards, the study areas have been selected in the coastal zone and flood plain zone of Bangladesh. Kuakata (coastal zone) and Pubail (flood prone area) regions of Bangladesh have been selected for this study. The specific gravity (G_s) of the soil samples collected from Pubail region varies from 2.50 to 2.55. Natural moisture content (w_n) ranges from 24 to 25%. Dry unit weight of the soil samples varies from 13.92 to13.96 kN/m^3. Fig. 1(a) shows the particle size distribution of the soil samples collected from Pubail region. Clay, silt, and sand fractions of the soils have been determined according to ASTM D 422. Clay, silt, and sand content of the soils vary between 20 to 30%, 60 to 65% and 2 to

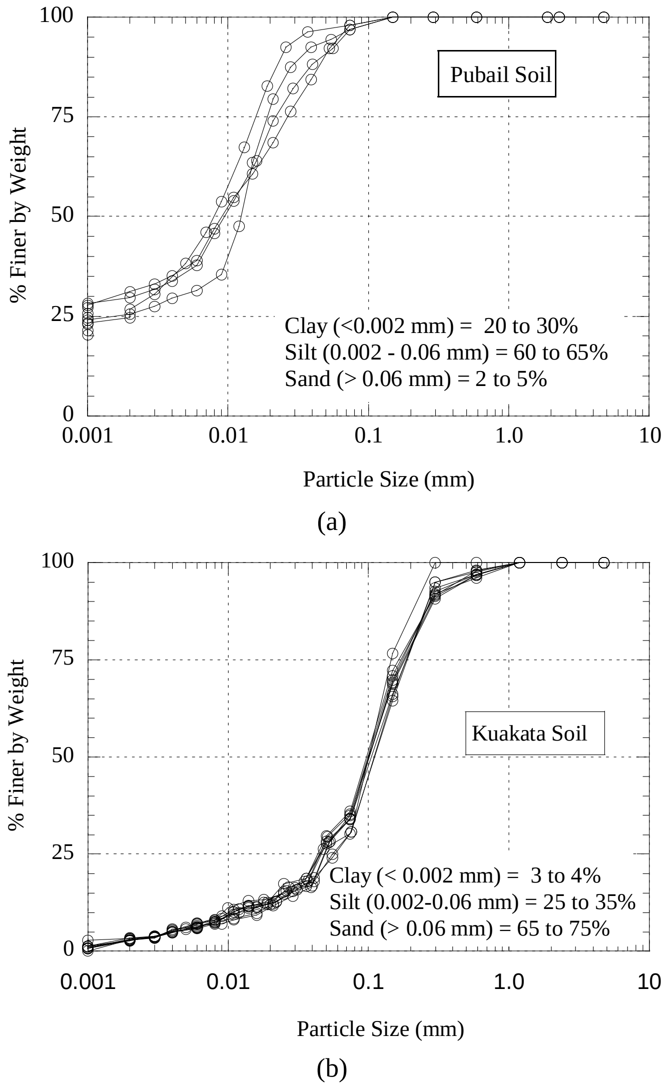


Fig. 1 Particle size distribution: (a) Pubail soil; and (b) Kuakata soil

5%, respectively. The liquid limit of the soils varies from 41 to 44%, plastic limit varies from 22 to 24% and plasticity index varies from 18 to 20%. It is found that the soil samples collected from Pubail are silty clay and the designated group symbol according to ASTM D 2487 is CL (Inorganic clays of low to medium plasticity or silty clay).

The specific gravity (G_s) of the soil samples collected from Kuakata region varies from 2.60 to 2.71. Natural moisture content (w_n) ranges from 10 to 11%. Dry unit weight of the soil samples varies from 14.40 to14.70 kN/m^3. Fig. 1(b) shows the particle size distribution of the soil samples collected from Kuakata region. Clay, silt, and sand content of the soils vary between 3 to 4%, 25 to 35% and 65 to 75%, respectively. These soils are found to be non-plastic. It is found that the soil samples collected from Kuakata are silty sand and the designated group symbol according to

ASTM D 2487 is SM (silty sand).

2.2 Parameters for numerical modelling

An elastoplastic constitutive model for soils, called the subloading t_{ij}-model (Nakai and Hinokio 2004), is used in the finite element analyses. This model requires only a few unified material parameters, but can accurately describe the following typical characteristics of soils: (1) the influence of the intermediate principal stress on the deformation and strength of soil is considered by using the t_{ij} concept; (2) the influence of the stress path on the direction of the plastic flow is considered by splitting the plastic strain increment into two components; (3) the influence of the density and/or the confining pressure is considered by adopting the subloading surface concept by Hashiguchi (1980). The parameters of subloading t_{ij} model are fundamentally the same as those of the Cam clay model (Roscoe and Burland 1968), except for the parameter a, which is responsible for the influence of the density and the confining pressure. Parameter β controls the shape of the yield surface. The performance of the constitutive model has already been checked in numerical simulations (Shahin *et al*. 2004, 2011, Nakai *et al*. 2010).

For getting parameters of the constitutive model, consolidation tests for both Pubail and Kuakata soils have been carried out in laboratory. Figs. 2(a) and (b) show the relations between void ratios and mean effective stress in logarithmic scale for Pubail and Kuakata soils, respectively. From these curves, compression index λ, swelling index κ and void ratio at 98 kPa, N are obtained for both soils. Using these values and fitting the computed curve parameter a (density parameter) of subloading t_{ij} model is obtained.

Drained triaxial test for both Pubail and Kuakata soils without root have also been conducted in laboratory to capture the properties of the soils in terms of the stress-strain relations. Figs. 3(a) and (b) represent the results of the triaxial compression drained test for Pubail and Kuakata soils, respectively. The Pubail soil shows compressive volumetric strain the same as clay soil. In contrast, the Kuakata soil shows the positive and negative dilatancy, though the stress-strain curve shows no significant peak strength, i.e., no softening behavior is seen for this specific sample. In numerical

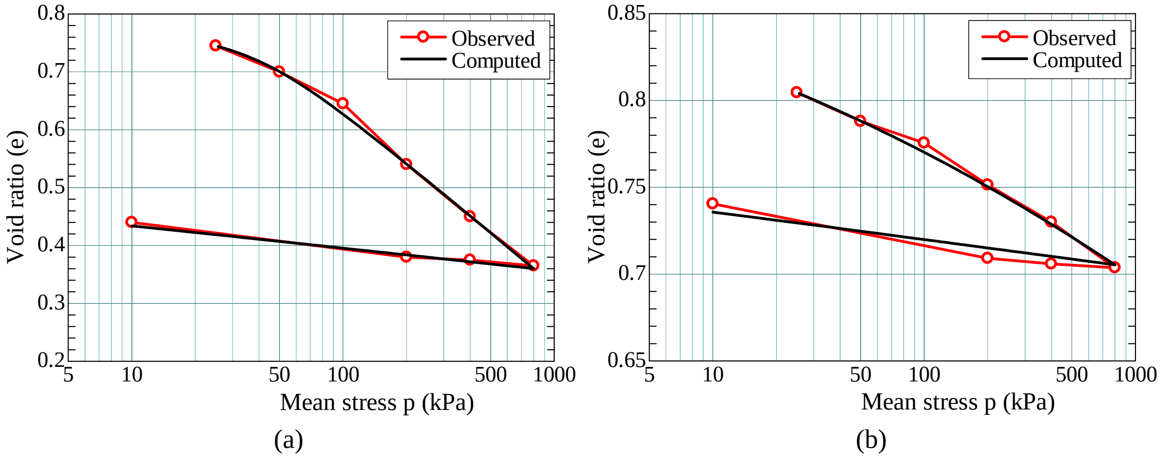
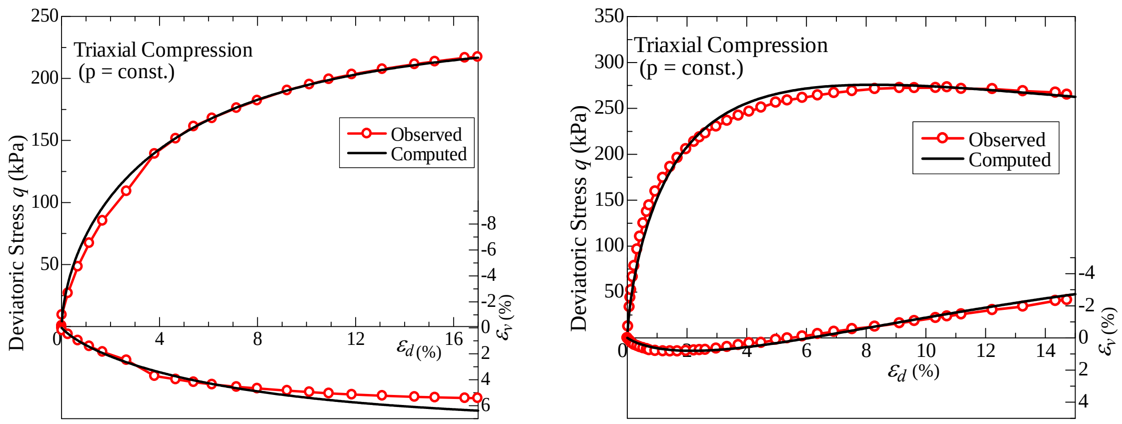


Fig. 2 e-lnp curve obtained from 1-D consolidation tests: (a) Pubail soil and (b) Kuakata soil



(a) (b)

Fig. 3 (a) Observed and computed results of drained triaxial compression tests: (a) Pubail soil; and (b) Kuakata soil

Table 1 Parameters used in subloading tij model for Pubail soil and Kuakata soil

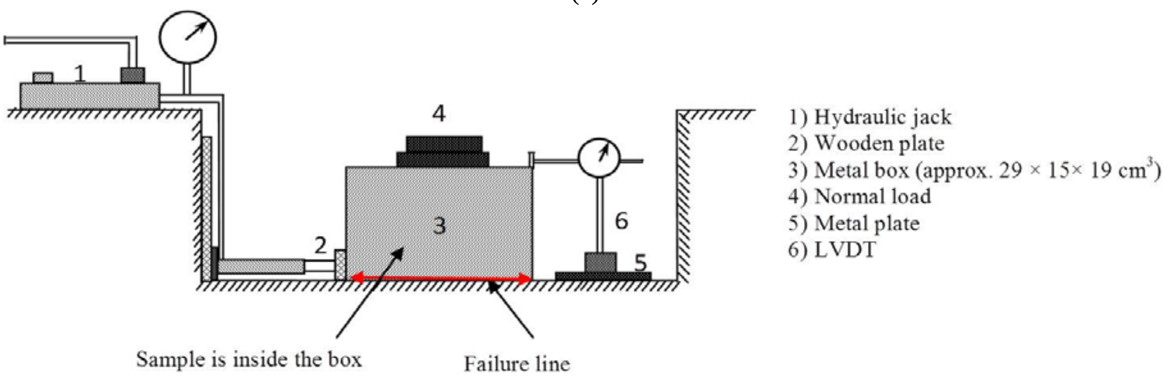
Parameter	Notation	Value		Remarks
		Pubail soil	Kuakata soil	
Compression index	λ	0.134	0.050	
Swelling index	κ	0.017	0.007	
Reference void ratio on normally consolidation line at p = 98 kPa and q = 0 kPa	e_{NC}	0.645	0.90	Same parameters as Cam clay model
Critical state stress ratio $R_{cs} = (\sigma_1/\sigma_3)_{cs(comp.)}$	R_{cs}	3.00	2.72	
Poisson's ratio	ν_e	0.2	0.2	
Shape of yield surface (same as original Cam clay at $\beta = 1$)	β	1.55	1.38	
Influence of confining pressure	a	800	115	

analysis, triaxial simulation is carried out considering one element in axisymmetric condition the same way as the laboratory tests. The same parameters of consolidation tests are used for the triaxial tests. The shape parameter β of the yield surface for the constitutive model is obtained fitting the computed results with the laboratory triaxial tests. The parameters of the constitutive model for Pubail soils and Kuakata soils are listed in Table 1. The parameters are fundamentally the same as those of the Cam-clay model except the parameter a, which is responsible for the influence of density and confining pressure.

2.3 Modeling of in-situ direct shear test results

Field tests were conducted to determine the in-situ shear strength and horizontal deformation failure of vetiver rooted soil matrix and bared soil at Kuakata and Pubail regions of Bangladesh. The in-situ test results were simulated using the subloading t_{ij} model. During the field test, it is found that roots of the vetiver grass grow up to 500 mm depth from the existing ground level (EGL). The density of the roots decreased with the increase of depth from the existing ground level. In both locations, field tests were conducted at a same depth (250 mm from EGL) for both unreinforced and rooted soils. For each location, field tests were conducted with three arbitrarily selected normal stresses (i.e., 11, 15 and 20 kPa). For each case, two tests were conducted to check the consistency of the test results. As a result, a total of forty eight block samples (0.29 × 0.15 × 0.19 m^3) were tested in the field under different normal stresses (i.e., 11 to 20 kPa). Out of forty eight block samples, twenty four samples were vetiver rooted and remaining twenty four samples were unreinforced soil. Details of the test results are available in Islam *et al*. (2010, 2013). In this paper, the test results obtained for 20 kPa for both Pubail and Kuakata sites has been explained.

(a)



(b)

Fig. 4 Experimental set-up for field test: (a) photograph showing the experimental set-up; and (b) schematic diagram showing the experimental set-up

2.3.1 Preparation of block samples

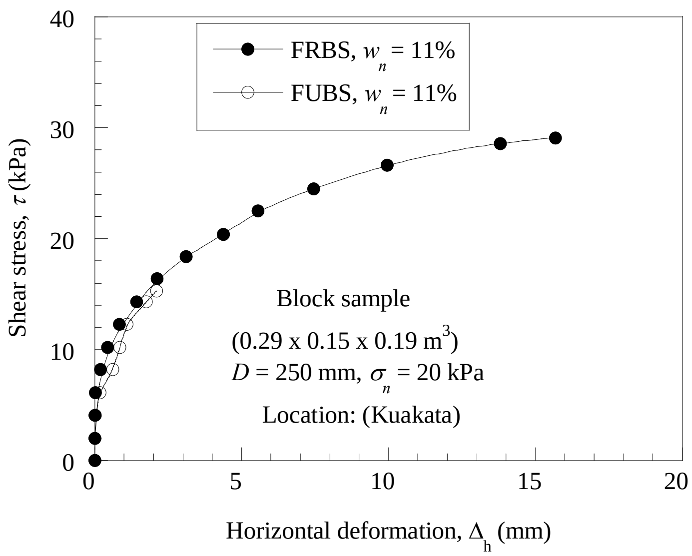
Firstly, the existing clump of vetiver grass was cut at the ground level with a sharp knife. Keeping the root position undisturbed, a trench of the size of 1 m × 1 m was made up to a desired depth. At first, slightly bigger size of the desired block is taken with wider rooted area then the rooted area is prepared in desired block sample shape by a sharp knife.

2.3.2 Test set-up

Metal box (having bottom face open) was smoothly pushed from the top of the block sample carefully ensuring that the bottom edge of the metal box does not touch the ground level. Then normal load was placed on the metal box. After that the shear force was applied in a particular direction from one side by a hydraulic jack. Calibrated pressure gauge having the capacity of 250 kN was used to measure the shear force. The block sample failed at the bottom. The deformation of the sample was measured by a Linear Variable Displacement Transducer (LVDT) having the capacity of 50 mm. The LVDT was fixed to the ground surface by a metal plate. Figs. 4(a) and (b) show the photographs and schematic diagram of the in-situ test set-up, respectively.

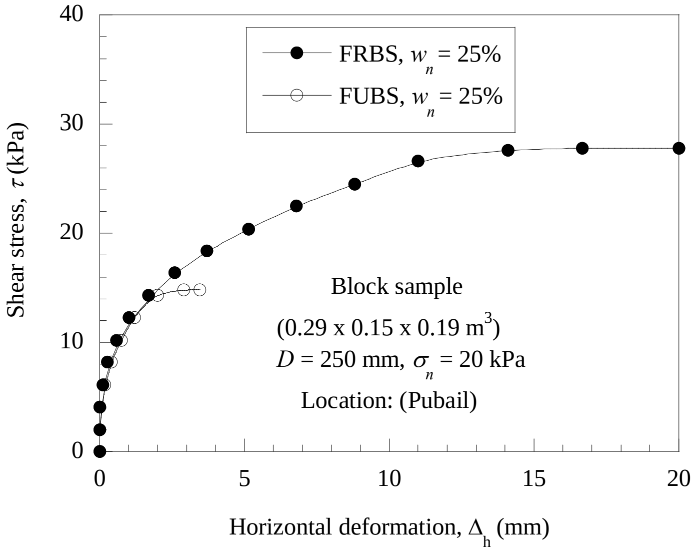
2.3.3 In-situ test results

Figs. 5(a) and (b) show the shear stress versus horizontal deformation graph of the field unreinforced block samples (FUBS) and field rooted block samples (FRBS) under 20 kPa normal stress at 250 mm depth for Kuakata and Pubail soils, respectively. It is observed that the strength of the vetiver rooted soil matrix is about 2 times higher than that of the unreinforced soil for both Kuakata and Pubail regions. The field rooted block samples show ductile behavior in both the Kuakata and Pubail soils. It means that vetiver roots are able to sustain more energy which may contribute to the stability of a slope.



(a)

Fig. 5 Shear stress vs. horizontal deformation graphs for: (a) Kuakata soil; and (b) Pubial soil from field tests



(b)

Fig. 5 Continued

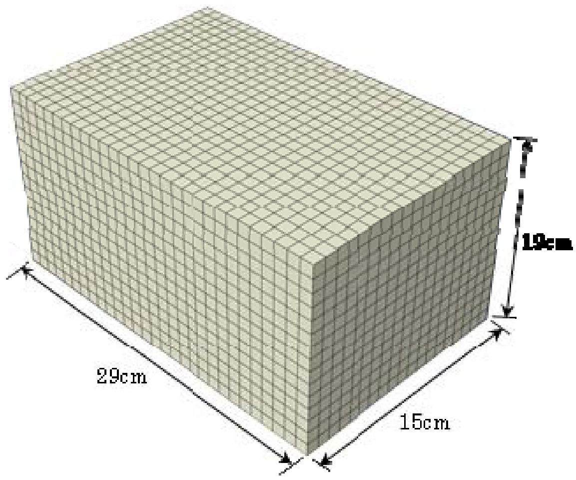


Fig. 6 3D finite element mesh for simulating field shear test

2.3.4 Simulation of in-situ test results

The three-dimensional analyses have been carried out for simulating the field test results with the program FEMt_{ij}-3D. The analyses have been conducted with the same scale of the field tests. Fig. 6 represents 3D finite element mesh for simulating the tests. Isoperimetric 8 noded brick elements are used to model the ground in the 3D analyses. The parameters shown in Table 1 are used in the analyses for the Pubail ground and Kuakata ground. The initial stress levels of the ground were calculated by applying the body forces due to the self-weight (for Pubail soil γ =

13.92 kN/m^3 and for Kuakata soil γ = 14.7 kN/m^3), starting from a negligible confining pressure ($p_0 = 9.8\times10^{-6}$ kPa). After making the initial ground, the weight of the over-burden (depth of 250 mm) is applied and then it is removed to get the same stress condition of the field block sample. Finally, confining pressures of 11 kPa, 15 kPa and 20 kPa are applied at the top surface of the mesh the same as the field before shearing the samples. In the reinforced ground, the vetiver roots are modeled with beam element having negligible bending stiffness. The average diameter of the vetiver root is considered as 0.75 mm (Hengchaovanich 1998), and average length of the root is assumed as 2.5 m (Erskine 1992, Truong 1999, Hellin and Haigh 2002, Ke *et al*. 2003). Young modulus E of 2.65 GPa is used for the vetiver root (Dunn *et al*. 1996).

Fig. 7 shows the computed relations of shear stress and horizontal displacement for both Kuakata and Pubail soils. Here, the results of computed unreinforced block samples (UBS) and rooted block samples (RBS) are compared in each figure. In the case of unreinforced block samples tension develops at the peak strength or just beyond the peak strength. In the formulation of modified stress t_{ij}, square roots of stress invariants are used. Therefore, the modified stress t_{ij} cannot be obtained in the case of negative stress condition hence the calculation stops when tension develops in the soils. This is one of the advantages of the elastoplastic subloading t_{ij}-model. In the case of reinforced block samples (RBS), the shear strength is higher than that of the unreinforced block samples (UBS). Similar trend was observed in case of field shear tests. It is seen that, the larger the value of the confining pressures the higher the shear strength of the block is as can be expected. The vetiver roots enhance both strength and ductility of the ground. The finite element analyses capture well the results of the field tests in both Kuakata and Pubail soils.

3. Analyses of geotechnical structures

3.1 Bearing capacity analyses

Bearing capacity of both Pubail and Kuakata grounds are simulated with finite element analyses. The simulations are carried out considering plane strain drained conditions using FEMt_{ij}-2D. Fig. 8 shows a typical mesh for two dimensional analyses, where the bottom boundary is fixed, and the lateral boundaries are free in the vertical direction. Roots (beam) are placed vertically from top to the bottom connecting two end nodes. The spacing of the beam is the same as the dimension of the elements in the mesh. Isoperimetric 4-noded quadrilateral elements are used to model the ground in the analysis. The same parameters shown in Table 1 are used in the analyses for the Pubail ground and Kuakata ground. The 2D model ground in every analysis is made by self-consolidating Pubail soil (unit weight = 13.92 kN/m^3) and Kuakata soil (unit weight = 14.7 kN/m^3) at a very low confining stress $p_0 = 9.8 \times 10^{-6}$ kPa. For both Pubail and Kuakata ground conditions, a strip foundation of 4m is used which is assumed to be an elastic material with large stiffness, and the frictional behavior between the foundation and the ground is simulated by an elastoplastic joint element (Nakai 1985). The friction angle between the foundation and the soils is assumed to be 15°. At first, the initial ground was prepared and then displacement is applied at central node on the strip foundation. The reinforced ground with vetiver root is modeled as beam element the same as the 3D analyses explained in the previous section. The average diameter of the vetiver root is considered as 0.75 mm and average length of the root is assumed as 2.5 m. Young modulus E of 2.65 GPa is used for the vetiver root as described in earlier sections.

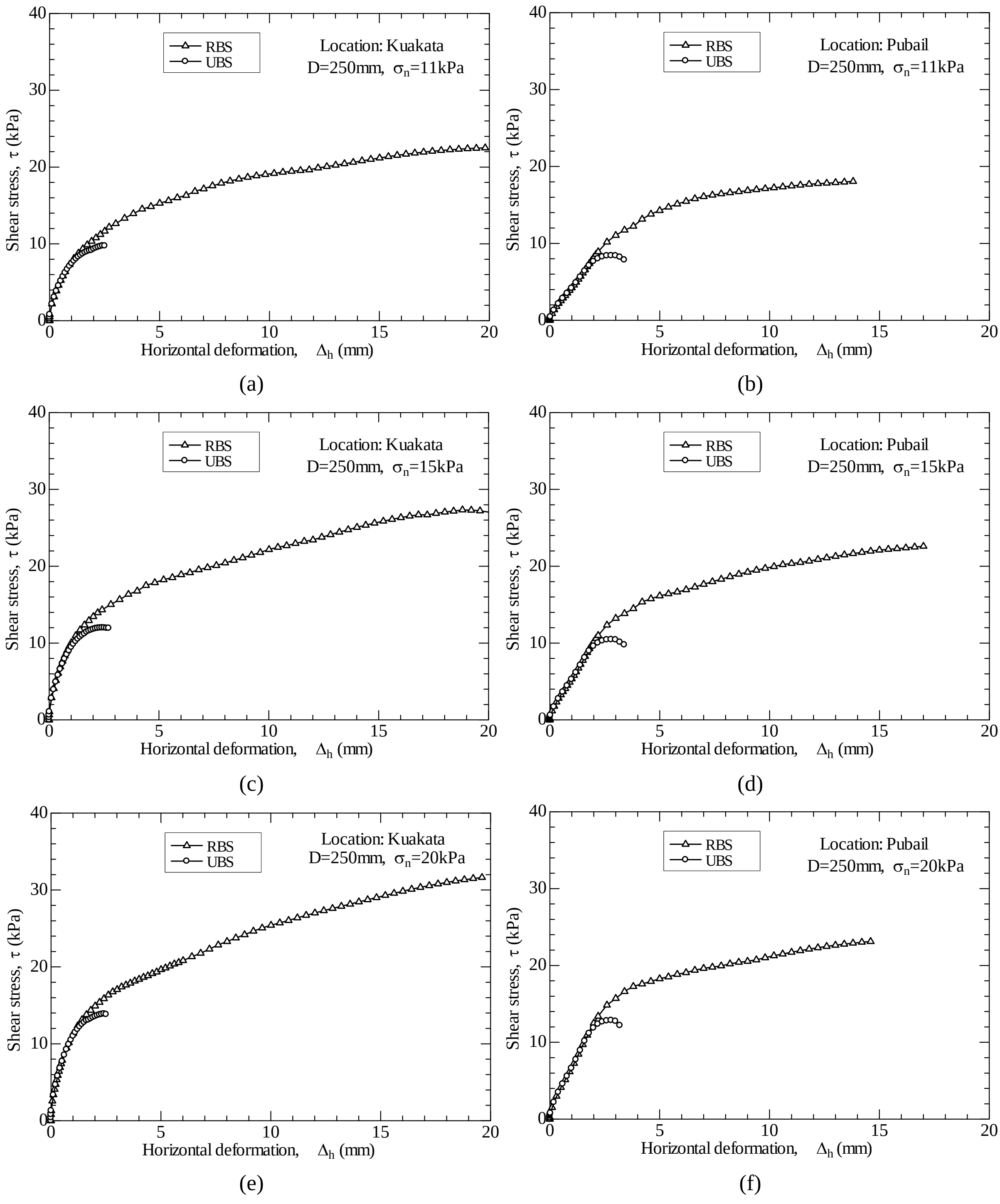


Fig. 7 Simulation of field shear stress vs. horizontal deformation results: (a) Kuakata soil (σ_n = 11 kPa); (b) Kuakata soil (σ_n = 15 kPa); (c) Kuakata soil (σ_n = 20 kPa); (d) Pubail soil (σ_n = 11 kPa); (e) Pubail soil (σ_n = 15 kPa); (f) Pubail soil (σ_n = 20 kPa)

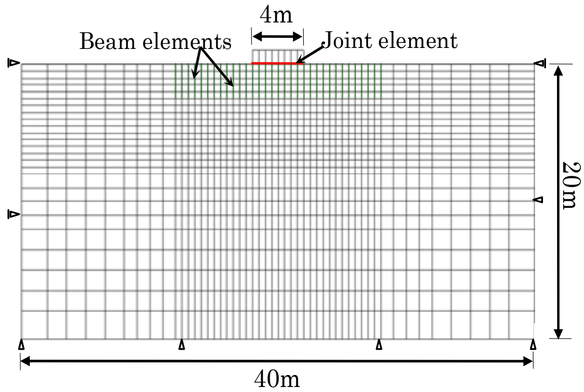


Fig. 8 Finite element mesh for bearing capacity analyses

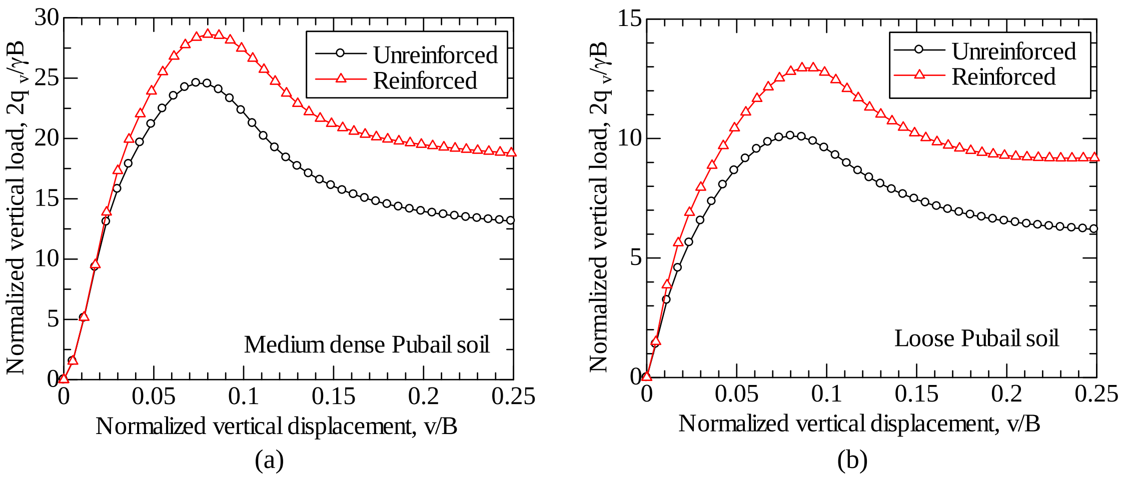


Fig. 9 Normalized vertical load ($2q_v/\gamma B$) vs. normalized vertical displacement (v/B) for Pubail grounds: (a) medium dense soils; and (b) loose soils

Fig. 9 shows the normalized load-displacement curves of Pubail grounds subjected to vertical concentric loading. Fig. 10 represents the same in the case of Kuakata soils. Fig. 9(a) represents the results of the ground with medium density, while Fig. 9(b) illustrates the results for loose soils. The vertical axis indicates the load normalized with γB (γ: unit weight of soil, B: width of the strip foundation) that corresponds to the coefficient of bearing capacity, and the horizontal axis is the displacement normalized with the width of the foundation, B. It is seen that the load carrying capacity of dense ground is larger than that of loose ground as can be expected. Comparing the results between the reinforced and unreinforced grounds, it is revealed that the reinforced ground with the reinforcement of vetiver root shows larger bearing capacity for both dense and loose grounds. As Kuakata soil contains much sand particles the load bearing capacity of this ground is significantly higher than that of the Pubail soil where the sand contents are lesser than clay and silts. However, the displacement in the Pubail ground is larger than that of the Kuakata ground at the ultimate bearing capacity (peak value of the vertical load). Even in sandy ground (Kuakata soils) the vetiver roots significantly increase the bearing capacity of the ground.

3.2 Slope stability analyses

Slope stability analyses are carried out to investigate the effectiveness of vetiver root in stability problem in a slope ground. Here, soil-water coupling analysis is performed considering the construction process of real ground embankment for both Pubail and Kuakata soils.

Fig. 11 illustrates the mesh for the finite element analyses, and the dimensions of the embankment with base ground. The levee crown width is 10.5 m, the bottom width is 31.5 m, and

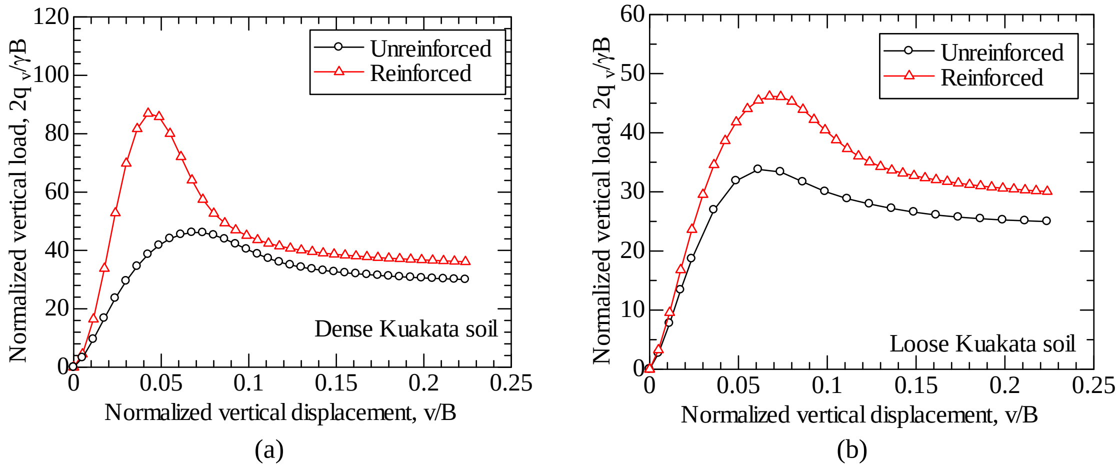


Fig. 10 Normalized vertical load ($2q_v/\gamma B$) vs. normalized vertical displacement (v/B) for Kuakata grounds: (a) medium dense soils; and (b) loose soils

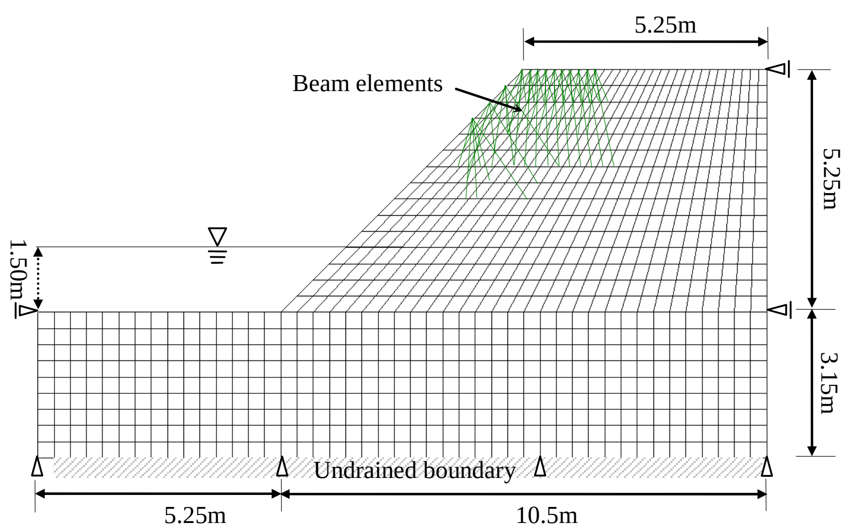


Fig. 11 Finite element mesh for slope stability analyses

the height is 5.25 m of the embankment with an inclination of 1:1. Considering the symmetric ground half section is taken for the analyses. The width of the half ground is 15.75 m having 8.35 m in depth. The bottom face is assumed as fixed boundary condition. The vertical faces are kept fixed in lateral directions. The left, bottom and the right drainage boundaries are considered as undrained boundaries. Initially water table is assumed as 3.75 m below the top surface of the embankment. Here, the effect of raising the water table up to the top of the embankment is simulated for unreinforced ground and the ground with vetiver root. The water table is raised to the top by one week which is very common during flooding time in Bangladesh. A worst situation has been considered raising the water table to the top of the embankment.

The same parameters of the Pubail soils and Kuakata soils shown in Table 1 are also used in these analyses. The initial stresses of the ground are calculated by applying the body forces due to self-weight (γ' = 7.84 kN/m^3), starting from a negligible confining pressure (p_0 = 9.8 × 10^{-6} kPa) and an initial void ratio e = 0.60 and 0.70 for Pubail and Kuakata soils, respectively. After self-weight consolidation the void ratio of the Pubail ground was 0.382 at the bottom and 0.437 at the top, and for the Kuakata ground it was 0.595 at the bottom and 0.595 at the top of the ground.

Fig. 12 illustrates the vertical stress distribution in the initial ground consisting Pubail soils. It can be observed that the self-weight consolidation, by the finite element analysis with the constitutive model (subloading t_{ij} model) used in this analysis, can reproduce a real ground. In the case of slope stability analysis, the strain occurred during the process of the initial ground formation has not been taken into account as the effect of flood water on the embankment is the main concern in this research. The coefficient of permeability for the ground of the Pubail soils is estimated from 1-D consolidation test as 10^{-8} m/sec. And that for Kuakata soils is estimated from 1-D consolidation test is 10^{-7} m/sec.

3.2.1 Displacement vectors: Pubail soil

Fig. 13 represents displacement vectors of the Pubail ground without reinforcement. The unit of the displacement is meter. In this case, the upper part of slope ground flows outwardly due to the generation of excess pore water pressure when the water table gradually goes up. For the excessive displacement complete failure of the embankment may occur. Fig. 14 presents the displacement vectors of the Pubail ground with vetiver root. It can be seen that for the same type of the

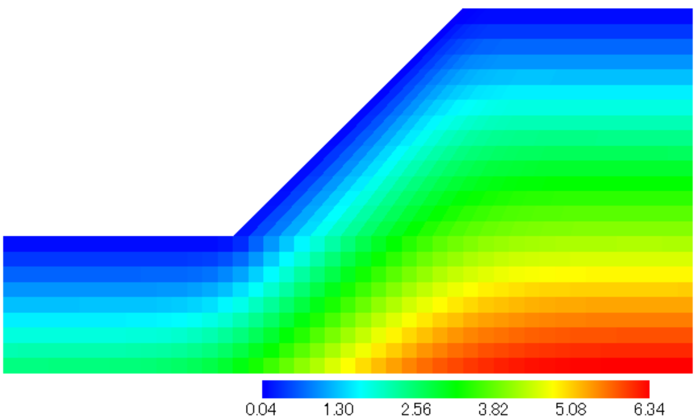


Fig. 12 Effective vertical stress of initial ground: Pubail Soil - unit in (× 9.8 kPa)

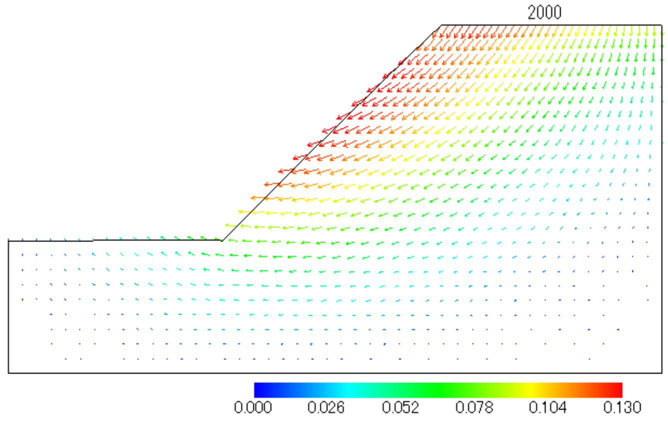


Fig. 13 Displacement vectors of Pubail soil without reinforcement (unit in meter)

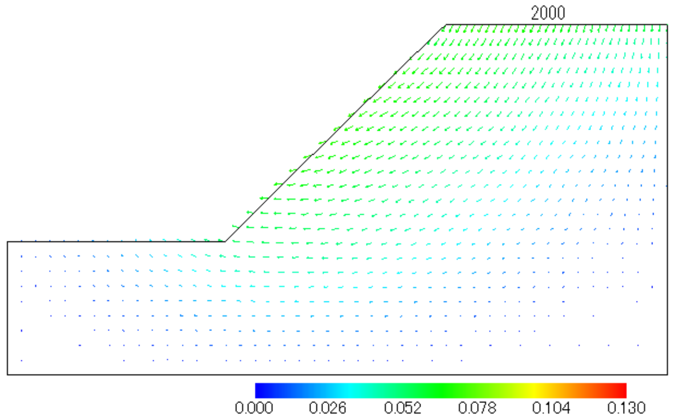


Fig. 14 Displacement vectors of Pubail soil with reinforcement (unit in meter)

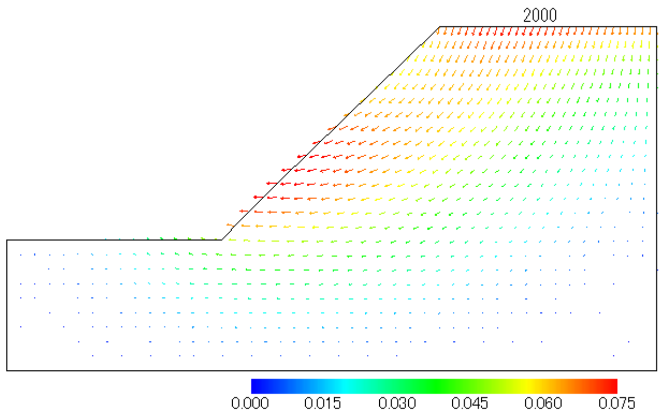


Fig. 15 Displacement vectors of Kuakata soil without reinforcement (unit in meter)

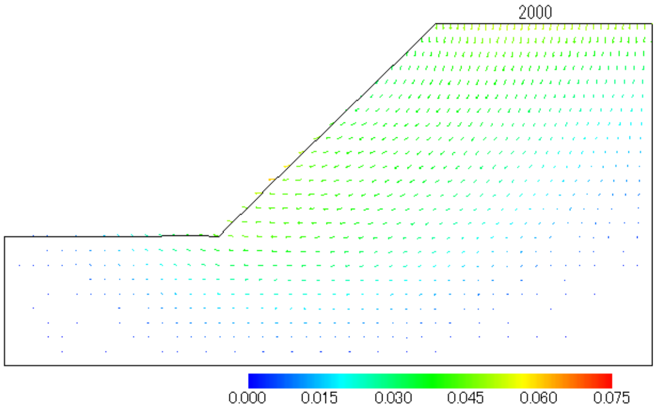


Fig. 16 Displacement vectors of Kuakata soil with vetiver roots (unit in meter)

Pubail ground the displacement is curbed in the reinforced ground where reinforcing is done with vetiver roots (Fig. 14). It is because the reinforcement restricts the lateral flow of the slope ground.

3.2.2 Displacement vectors: Kuakata soil

Fig. 15 represents displacement vectors of the Kuakata ground without reinforcement. The unit of the displacement is represented in meter. In this case, the upper part of slope ground flows outwardly due to the generation of excess pore water pressure the same as the Pubail soils. This excessive displacement may trigger the failure of the embankment. Displacement vectors of Kuakata ground with vetiver root is presented in Fig. 16. The reinforcements with vetiver root curb the displacement of the ground similar to the Pubail soil (Fig. 14).

In general, the magnitude of lateral displacement is smaller in the case of the reinforced ground in both Pubail and Kuakata soils. Therefore, it can be said that the vetiver roots can be used in a slope ground to prevent it from failure during rapid rise in water table during flooding season.

4. Conclusions

The soils in the coastal zone considered in this study are generally silty sand (SM) and the soils in the flood plain area are silty clay (CL). The shear strength parameters, cohesion and angle of internal friction for Kuakata soils are 9 kPa and 25°, respectively. For Pubail soil, the cohesion varies from 16 to 18 kPa and the angle of internal friction varies from 27° to 31°. Hence, the shear strength of such soils is very low. So it can be presumed that embankment slope made from such soil will be dislodged easily due to storm surge and rain-cut erosion.

In-situ shear tests of the ground with the vetiver roots have been conducted to investigate the stabilization properties corresponding to the embankment slopes. Effectiveness of vetiver root in geotechnical structures–strip foundation and embankment slope has been evaluated by finite element analyses. From the model tests and numerical analyses, the following points can be concluded;

(1) The shear strength of vetiver rooted soil matrix is higher than that of the unreinforced soil. The reinforced soil with vetiver root also shows ductile behavior. The numerical analyses

capture well the results of the in-situ shear tests.

(2) The reinforcement with vetiver roots enhances the bearing capacities for both dense and loose grounds. Even in sandy ground (Kuakata soils) the vetiver roots significantly increase the bearing capacity of the ground.

(3) During flooding (rising water level), the displacement of the ground is curbed in the reinforced ground where reinforcing is done with vetiver roots, because the reinforcement restricts the lateral flow of the slope ground.

Reinforcement with vetiver roots causes a significant reinforcing effect in the Pubail and Kuakata grounds. The vetiver root enhances the bearing capacities of the grounds and stabilizes the embankment slopes. Hence, vetiver plantation can be a low-cost, eco-friendly and sustainable technology for embankment protection.

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