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Model experiment and analysis of sandwich earth fill reinforced with geosynthetics

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ABSTRACT: The one of the issues in Japan is to reuse marginal soils such as high water content volcanic cohesive soils and/or construction by-products which have never been used for fill materials. This paper proposes the conventional sandwich method conbined with geosynthetics for obtaining the long term stability due to both drinage and reinforcing functions of gravel sandwich layers and geosynthetics respectively. This paper describes the results from two series of steep earth fill model test. One is for sandwich earth fills without any geosynthetics to obtain the effect of the gravel layer itself. Another is for sandwich earth fills reinforced with geosynthetics varying coarce-grained soil layers to obtain the both fffect of the coarce-grained soil layers and geosynthetics. This paper also conducts Finite Element analysis to develop the simulation model for the proposed sandwich earth filling method conbined with geosynthetics. The results of the model tests have shown the sufficient effects of the proposed filling method.

1 INTRODUCTION

Recently, the increase of the surplus soil from a construction site, especially soft soil and clay, has become the one of the serious issue in Japan. It is difficult to keep the disposal ground for the such surplus soil and the disposal cost of these soils becomes expensive, too. Therefore it is a social demand to utilize such soils effectively. Recently, it has become necessary to use even high water content volcanic soil as earth fill materials. In addition to the shearing resistant force and drainage function in the sandwich method, instant increase of tensile strength by reinforcement materials is expected to provide both short and long term reinforcing functions.

In this study, the authors observed the behavior of the model earth fill made by Sandwich method, in case of different thickness of the coarse-grained soil layer, and several reinforced cases. Also, the authors observed the effect of the thickness of coarse-grained soil layer on to the tensile resistance.

2 MODEL TEST

2.1 *Experimental method*

The authors assumed a case of an earth fill for road. Since usual compaction method can not be applied, the model was made by the handiwork at the medium density. A earth fill model was constructed to height H =0.5m as a base fill, and uniform surcharge by the air control was applied to simulate the load by the fills on top of that. the authors observed the behavior against the surcharge up to 47.5 kPa (equivalent to a 25m high earth fill). Table 1 shows the experimental specification.

Table 1. Model experiment specification.

Name	Contents
Scale	1/10
Experiment box width and shape of earth fill	experiment box width 0.35m width of crest 0.6m width of the base 0.9m height 0.5m
Condition of volcanic cohesive soil	w=47%, ρ=1.7g/cm^3, c=1~2kPa
Condition of sandy soil	w=3%, ρ=1.5g/cm^3, φ=35°
Reinforcement material	Geogrid KZ-2-20, T=2kN/m, E=4.5×10^{2}MPa

2.2 *Test materials*

The material of earth fill model was volcanic cohesive soil with high water content sampled from Mt.Aso, and the material of coarse-grained layer was sand. The material of reinforcement was geogrid.

2.3 *Measurement installments*

Figure 1 shows the arrangement of measurement installments. The authors installed the earth pressure meters and the pore water pressure meters in three places, and set up the displacement gauges in five places to measure vertial and lateral displacement at

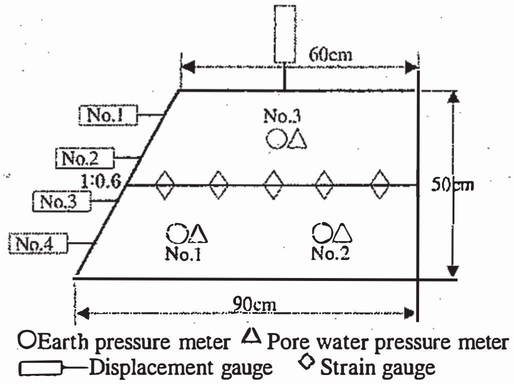


Figure 1. Arrangement of measurement installments.

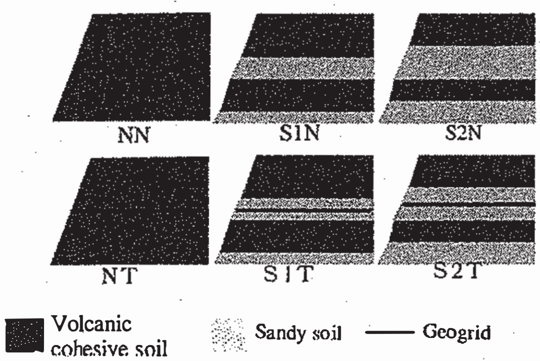


Figure 2. The experimental cases (6 cases).

the top of the earth fill. In case of reinforcement earth fill, the authors installed the strain gauge in reinforcement materials.

2.4 *Experimental cases*

The purpose of this experiment is inspection of the behavior of earth fill in different thickness of sand layer and combination effect of inserted reinforcement materials. Table 1 shows the six experimental cases, and Figure 2 shows the cross section views of those models.

In this model experiment, the following cases were examined and the results were compared.

Table 2. The experimental cases.

Case division	Cohesive soil single layer	Sandy soil thin layer (t=6cm)	Sandy soil thick layer (t=12cm)
N (non-reinforced)	NN	S1N	S2N
T (reinforced soil)	NT	S1T	S2T

3 EXPERIMENT RESULTS

3.1 *Amount of settlements*

Figure 3 shows the comparison of the settlement of each case and Table 3 shows maximum settlements. The settlements in case NN and NT are especially large. The amount of settlement in case NN is 111 mm at the final surcharge 47.5 kPa.

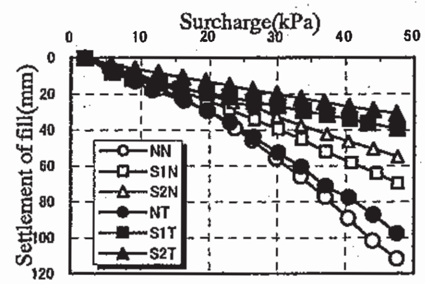


Figure 3. Relationship between surcharge and fill settlement.

Table 3. The comparison of the fill settlement and maximum lateral displacement.

case	Fill settlement (mm)	The ratio	Order	Maximum lateral displacement(mm)	The ratio	Order
NN	111.82	1.00	①	119.04	1.00	①
S1N	69.00	0.61	③	58.22	0.49	③
S2N	54.86	0.49	④	54.78	0.46	④
NT	97.68	0.87	②	99.72	0.83	②
S1T	39.26	0.35	⑥	27.24	0.23	⑥
S2T	31.06	0.27	⑥	28.94	0.24	⑥

The settlement of case S1N is 50~61% of that in case NN. The settlement in case S2T is 27~35% of that in case NN. The authors found the sandwich method with reinforcement materials is effective to restrain settlement. Whereas the case of clay with reinforcement materials inserted, the effect is smaller in restraining settlement.

3.2 *The maximum lateral displacement in the slope (the upper step)*

Figure 4 shows the comparison of each case of maximum lateral displacement (the top). The tendency of maximum lateral displacement is same as the amount of settlement, and those in the case NN and NT are especially large. In case NN, the lateral displacement is 119mm at the load 47.5kPa, showing the upper part the laying earth was transformed greatly.

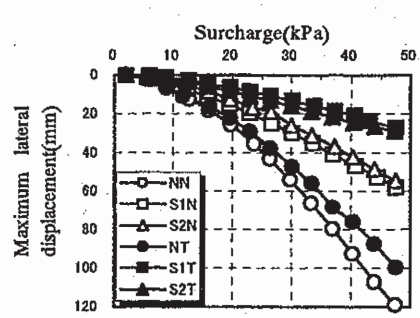


Figure 4. Relationship between suecharge and maximum lateral displacement of slope.

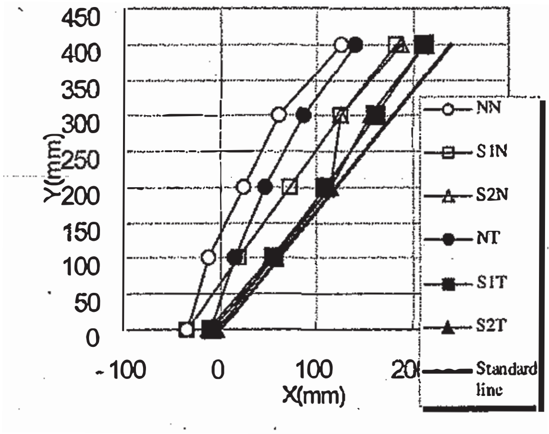


Figure 5. Slope deformation of final state (lateral displacement).

The lateral displacement in case S1N is 49% of that in case NN. The lateral displacement of case S2N is 83% of that in case NN, and the lateral displacement of case S1T is 23% of that in case NN, the lateral displacement of S2T is 24% of that in case NN.

So the lower clay layer of S2N, S1T, and S2T, did not move much in lateral.

3.3 *Lateral displacement distribution in the slope (final states)*

Figure 5 shows final states of the fill slope deformation in each case.

In no reinforcement cases, the deformation of the slope in case NN is significantly. On the other hand, the slope in case S1N (Sandwich) becomes flat, and the maximum lateral displacement is about half of that. The maximum lateral displacement in case S2N, where sand layer is thick, is about same as that in case S1N, but it is characteristic that displacement of lower clay is extremely small. The maximum lateral displacements in case S1N and S2N are about 50% of that in case NN. These results show the sandwich method of construction is effective for restraining slope deformation.

On the other hand, in reinforcement cases, the lateral displacement in case NT is smaller than that in case NN (about 85%). The displacements in case S1T and S2T are extremely small. The authors found the transformation restraint effect was small by laying reinforcement materials in a single clay layer, but transformation was remarkably restrained by laying reinforcement materials in the sand layer of sandwich method. And, in case of using the reinforcement, the authors found the difference of displacement by thickness of the sand layer was small. Accordingly, the sandwich method of construction with reinforcement materials in thin sand layers was found effective for transformation restraint and stability of high earth fill.

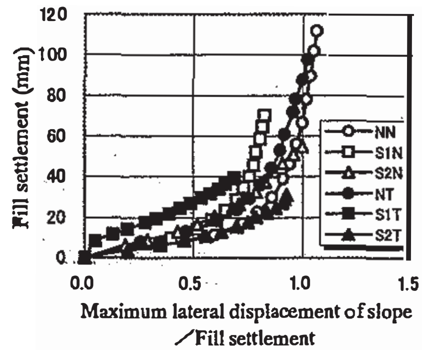


Figure 6. Relationship between fill settlement and maximum slope lateral displacement /fill settlement.

3.4 *Evaluation of the slope stability*

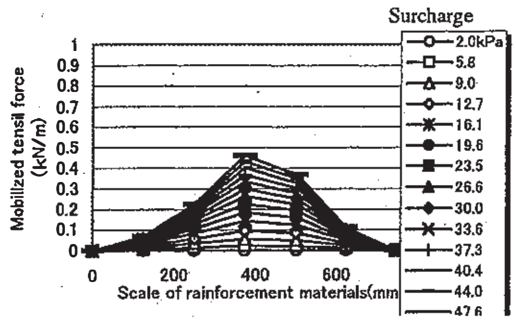
Figure 6 shows the relationship between the fill settlement and maximum slope lateral displacement per fill settlement at each 3.5 kPa increment up to 47.5 kPa. In case NN and NT, the curve incline changes steep at low surcharge of less than 20 kPa, there is a sign of the destruction in early stage. In case S1N, the curve incline change steep at 25 kPa, that is the sign of destruction. However, in case S2N, increase of incline is slow even over 30 kPa, and the settlement increases slow (in this case, there is the destruction tendency only upper part sand layer). On the other hand, in case S1T and S2T with reinforced sandwich method, the curve incline does not change and there is no sign of destruction. In Figure 8, the reinforced Sandwich method (case S1T and S2T) shows the large effect on the increase of stability.

3.5 *Mobilized tensile force in reinforcement materials*

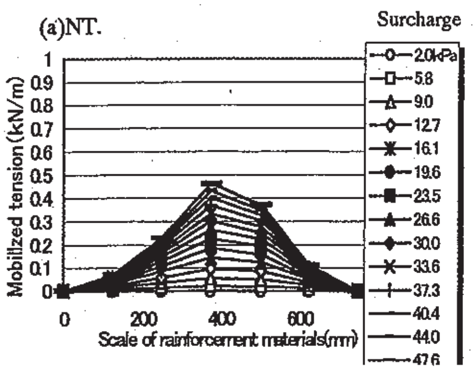
Figure 7 shows tensile force in reinforcement materials. The authors measured tensile force in reinforcement materials by strain gauge in case NT, S1T, and S2T, which are the case that reinforcement materials are inserted. Tensile force almost increases in a constant ratio in all cases at the surcharge up to 47.5 kPa. In case NT, the largest tensile force 0.9kN/m is measured at the distance of 50cm from the edge of the slope, which is significant at the center of loading plate. This means the tensile force is about 50% of reinforcement strength.

The largest tensile force in case S1T is 0.45 kN/m at 45cm distance from the edge of the slope. The authors found that stresses are more spread and peak plateau is wider compared with case NT.

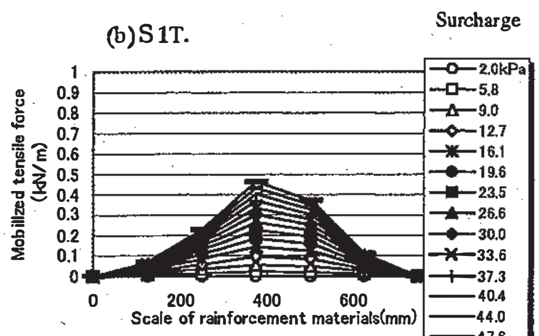
In case S2T, the largest tensile force is 0.45 kN/m at 37.5cm distance from the edge of the slope. The authors found that the peak plateau is more spread than case S1T, and the peak is lower toward edge. Thicker the sand layer is, the nearer the maximum tensile force is to the slope edge, shallower the deformation zone becomes.



(a)NT.



(b)S1T.



(c)S2T

Figure 7. Mobilized tensile force in geogrid.

Therefore, the thickness of the sandwiched sand layer has no effect the tensile force and the tensile force is relatively small in case of the thin (6cm) sand layer. So, the authors found reinforcement materials inserted in thin sand layer have enough reinforcing effect.

3.6 *Earth pressure*

Figure 8 shows the relationsip between surcharge and earth pressure curve. In case of no reinforcement in the lower layer No.1, earth pressure in case S1N increases. The authors think vertical earth pressure by sand layer restraints. The authors think, the reason that earth pressure does not change in case NN is due to the large horizontal deformation by earth fill-material made from clay only.

On the other hand, in cases of reinforcement, NT, S1T, and S2T, earth pressure increase linearly, in spite of different gradients. The authors think earth pressure declines in case S2N, because of circular slip of internal sand layer.

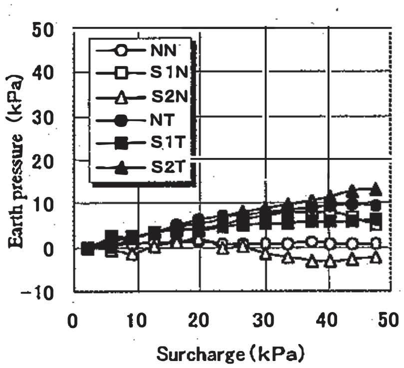
Comparing with and without reinforcement, earth pressure is larger with reinforcement. The authors think vertical earth pressure decrease due to lateral flow in case of no reinforcement. In case of lower layer No.2, earth pressures increase linearly in all cases. Comparing with and without reinforcement, earth pressure in reinforced case is larger. The authors think this is due to the confining effect of the reinforcement materials.

In case of upper layer No.3, earth pressure decrease after the peak in all cases, except case S1T. The authors think earth pressure decreases due to circular slip in the case of NN and NT with only clay.

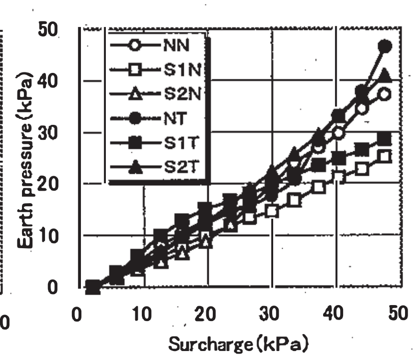
3.7 *Pore water pressure*

Figure 9 shows surcharge-pore water pressure curve. In case of lower layer No.1, the pore water pressure decreases after a peak in all cases. The authors think this is due to the drain effect generally.

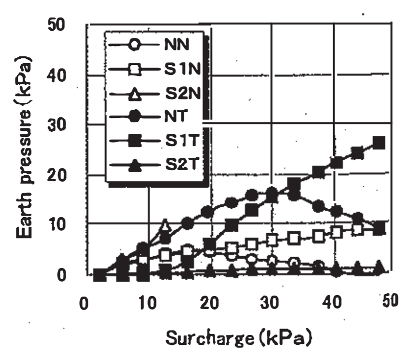
In case of lower layer No.2, it is difficult to drain because measurement position is lower right in the structure. In cases except S2N and S2T, the authors think the pore pressure increases linearly. In case



a) Earth pressure meter No.1

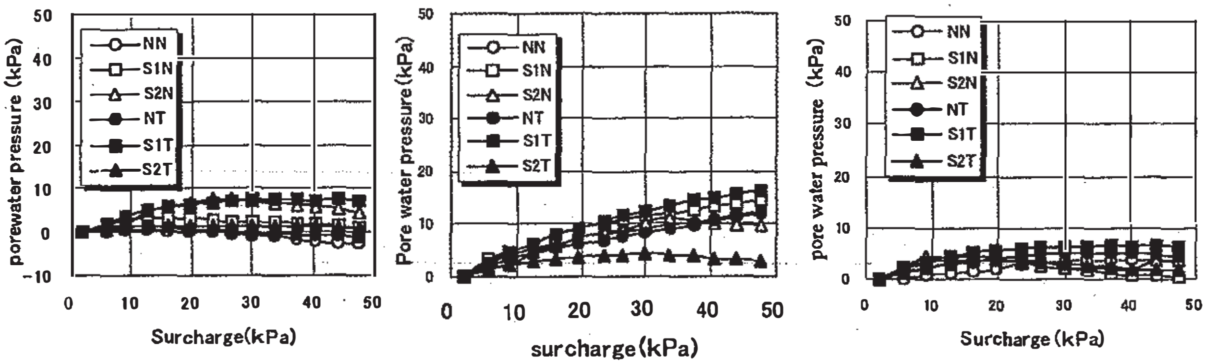


b) Earth pressure meter No.2



c) Earth pressure meter No.3

Figure 8. Surcharge - Earth pressure curve.



a) Pore water pressure meter No.1 b) Pore water pressure meter No.2 c) Pore waterpressure meter No.3

Figure 9. Surcharge - Pore water pressure curve.

S2N and S2T, there is a tendency that pore pressure decreases after a peak due to effect of the thick sand layer.

In case upper layer No. 3, the pore pressure decreases after a peak in all cases. It is considered that this cause is the drainage effect.

3.8 *Summary of experimental result*

① Amounts of deformation are in the following order.

NN>NT>S1N>S2N>S1T>S2T

The deformation in case NN made from clay only is extremely large. However, it is possible to decrease the deformation to some extent if the sandwich method adopted (the S series) (the deformation is 50~60% of that in case NN).

Furthermore, it is possible to decrease the (S2T deformation extremely by inserting reinforcement materials (the T series) in the sand layer (the deformation is 27~35% of that in case NN). However, the effect in case NT, which is not sandwich method, is small and the deformation in case NT is 85% of that in case NN.

② The effect of reinforcement by inserting reinforcement materials in clay is small (the deformation is 85% of case NN). In case S1N, sandwich method in thin layer is rather effective (the deformation is 50% of that in case NN).

NT>S1N

③ Scales of circular slip are in the following order.

NN>NT>S1N>S2N

There is no evidence of sliding in case S1T and S2T.

④ In case of no reinforcement, the restraining effect of sliding deformation is smaller in S1N (the deformation is 60% of that in case NN). When they are reinforced by the sandwich method, the restraining effect of sliding deformation is larger in thin sandwich method (case S1T, deformation is 35% of that in case NN) than thick sandwich method (case S2T, deformation is 50% of case that in NN).

S1N>S2N>S1T

⑤ The maximum tensile force in reinforced materials is largest in case NT with 0.9kN/m, and is same in case S1T and S2T with 0.45kN/m. The thicker the sand layer becomes, the nearer to the slope edge the maximum position is.

4 PULLING TEST OF REINFORCEMENT MATERIALS

The authors conduced the tensile strength test (pull out speed is 1mm/min), in order to determine the friction between reinforcement materials and soil, and calculated the apparent cohesion c^*, the coefficient of reduction α, and the gradient of straight line β. The maximum tensile shear strength can be calculated by the following equations.

$$\tau_{max} = \sigma N \cdot \tan\beta + c^*$$
$$= \alpha \cdot \sigma N \cdot \tan\phi + c^*$$

By these equations, the maximum tensile shear strength in volcanic ash soil is τ max =3.6 (kPa), and, that of sand soil is τ max = 0.366 σ_N tan35.05(°) +0.82 (kPa).

5 VANE TEST

Natural water content of volcanic ash soil used in this model test was about 45%, and this shear strength was about c=10 kPa. It is desirable to reduce strength around 1/10.

Therefore, in order to make consistent samples, the authors set up the target water content (47%), and strength around τ=1~2 kPa.

As a result the authors found this cohesion is c =0.5-2.5 kPa.

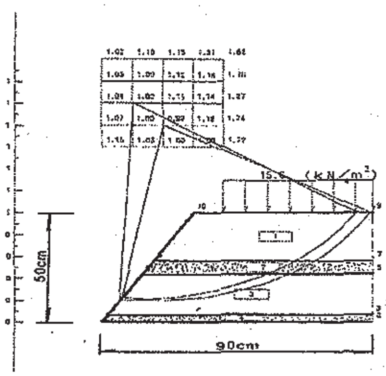


Figure 10. Defoımational analysis.

6 ANALYSIS

6.1 *Stability analysis*

The authors conducted stability analyses assuming a circular slip surface for three cases NN, S1N, and S2N in accordance to C =5 kPa. The safety factor of each case is roughly equal to 1, so It is considered that Analysis result is consistent with the deformation behavior by experiment (figure 10).

6.2 *FEM analysis parameters*

The authors assumed the constitutive model of soil is linear elastic body. Table 4 shows analysis parameters. Modulus of elasticity of volcanic cohesive soil is determined based on the results from parametric study.

Table 4. Analysis parameters.

The earth fill materials	Volcanic cohesive soil V	Sandy soil S
Young's modulus E(kPa)	150	5,000
Density of soil γ (kN/m³)	17.0	15.0
The poisson's ratio ν	0.45	0.35

Reinforcement materials (Geogrid)		
Align ment girder	Tensile stiffnes EA (kN)	45
	Flexural rigidity EI(kN・m²)	45
	Shearing rigidity GA(kPa)	4,500
Joint	Tangential rigidity E_s(kPa)	100
	Alignment rigidity E_n(MPa)	10
	Adhesion (kPa) 0.8, Frictional angle (°)	14.4

6.3 *Comparison of the experimental values and FEM analytical results.*

Table 5 shows the comparison of the experimental values and FEM analytical results (Linear elasticity analysis). Also Figure 11 shows the example of deformation analysis results. The settlement is close to the experimental value, except the cohesive soil with a single layer (case NN and NT). But, the analytical values of lateral displacement are all smaller than experimental values, except sandwich method by reinforcement (case S1T and S2T). However, the settlement clearly shows the effect of reinforcement. On the other hand, lateral displacement proves the effectiveness of sandwich method, but it does not prove the effectiveness of reinforcement.

Table 5. The comparison of the experimental values and FEM analytical results.

	Fill settlement (mm)		A maximum lateral displacement(mm)	
case	Experimental values	FEM	Experimental values	FEM
NN	111	80	119	70
NT	97	60	99	60
S1N	69	50	58	25
S2N	55	35	55	25
S1T	39	45	27	25
S2T	31	30	28	25

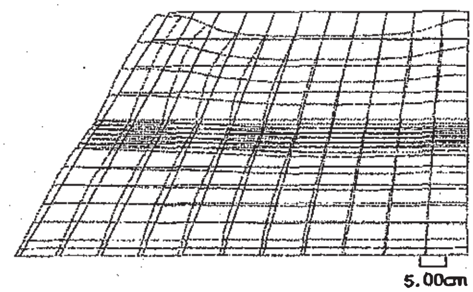


Figure 11. The example of deformation analysis results.

7 CONCLUSION

The authors found following results in this test

① The construction of high earth fill with steep slope is possible by using reinforcement materials to the traditional sandwich method.

② The sandwich method with reinforcement materials in thin sand layers is very effective in decreasing the deformation.

③ Tensile strength in reinforced materials of sand layer is not very mach influenced by the sand thick.

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