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# Development of new FRTP-geogrid and its application to test embankment

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ABSTRACT: The authors developed a new type of FRTP-geogrid which combined aramid fiber with geogrid made of high density polyethylene. The aramid fiber is used as lead within geogrid and makes the resulting composite material strong and hard to stretch. Firstly this paper reports the performance of the geogrid developed (composite material), for instance, the overall tensile strength, creep property, durability, and so forth. The results of these tests show that the developed geogrid is highly useful for reinforcing soil embankment. Secondly this paper describes the result of applying the developed geogrid to an actual embankment, where a vertical retaining wall of concrete blocks was used and the backfill was reinforced with the developed geogrid. Monitored were the lateral movement of retaining wall, strains produced on the geogrid, etc. Without any trouble, 40 days have passed since the embankment was completed. The results of measurement suggest that the developed geogrid can be successfully applied to actual soil structures.

## 1 INTRODUCTION

The materials of the geogrid used presently for a reinforced embankment process seem to be mostly of high-density polyethylene. This may possibly stem from the fact that high-density polyethylene is appraised excellent over the characteristics (chemical resistivity, stiffness, etc.) which are sought with such materials as referred above. We have of late developed an FRTP-geogrid via the process of molding wherein high-density polyethylene and aramid fibres are combined together for simultaneous molding into a grid structure (see Fig.1).

It is widely known that the aramid fibres exhibit not only high toughness and a slight elongation percentage but also low creep characteristic as shown in Fig.2. Quoted herein are the results from the performance verification test which we conducted for the FRTP-geogrid (ADEAM: a registered tradename) recently developed, using a composite material comprising the aramid fibres and polyethylene in combination. Also included herein are the data of dynamic observations of a vertically reinforced retaining wall of an actual size (5m high) with large concrete blocks used in combination.

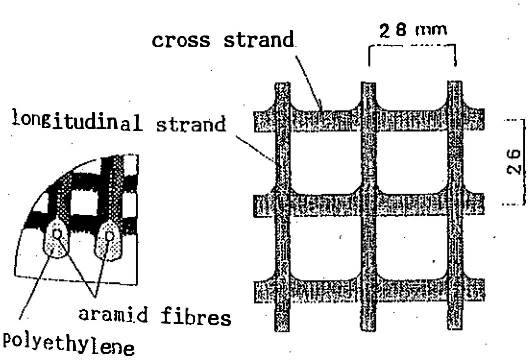


Fig.1 FRTP-geogrid structure

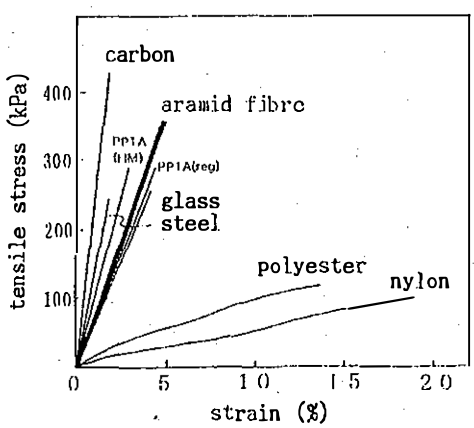


Fig.2 Stress-strain characteristics of individual materials

## 2 FRTP-GEOGRID TENSILE STRUCTURAL PERFORMANCE VERIFICATION TEST

### 2.1 Testing for tensile strength characteristic

1. Tension test: The tension test referred to herein is applied to ascertain the tensile strength/elongation characteristic which ADEAM shows when pulled at a constant rate with a tension tester used. As illustrated in Fig.3, ADEAM exhibits the tensile strength/elongation characteristic approximately the same as found with aramid fibres. Through the test, it was disclosed that the geogrid shows the maximum tensile strength at an elongation of 5 through 7%.

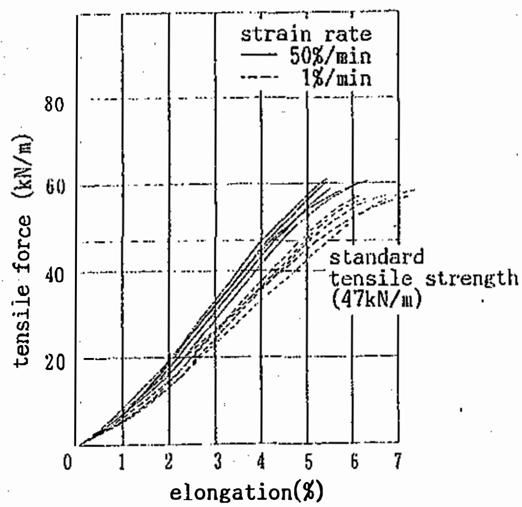


Fig.3 Tension test results of ADEAM(G-6) (8 pieces)

2. Point-of-intersection strength test: The point-of-intersection strength test herein involves a tension test to confirm unitary integrity which is required between the cross and longitudinal strands - respective elements necessary to make up the geogrid, wherein the tension test is done with a cross strand of a test piece kept fixed while a longitudinal strand is pulled, using a hook hung therefrom (see Fig.4). The test disclosed that the cross strand underwent a deformation but experienced neither rupture nor relocation at each point of intersection, with unitary integrity maintained between the cross and longitudinal strands. This coincides with a proof that ADEAM owns sufficient point-of-intersection strength as specified in Table 1, and that 2 to 3 cross strands match a single longitudinal strand over tensile strength.

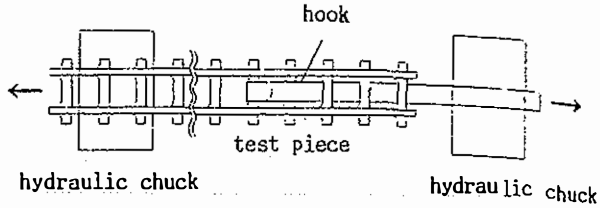


Fig.4 Point-of-intersection strength test

Table 1. Point-of-intersection strength test results (G-6)

| tensile strength (kN) | | | | | |
|---|---|---|---|---|---|
| No.1 | No.2 | No.3 | No.4 | No.5 | av. |
| 0.96 | 0.93 | 0.83 | 0.81 | 0.77 | 0.87 |

Tensile strength of one longitudinal strand in ADEAM G-6: 1.65 through 1.94kN

3. Bonding strength: We conducted a bonding strength test to examine the ADEAM for unitary integrity which gives adhesion between an aramid fibre core and a polyethylene covering. With reference to the outcomes of the test quoted in Table 2, it is concluded that the length of bonding together the aramid fibre core and polyethylene covering which is over 260mm will suffice to not only prevent the core and covering from coming off each other but also prohibit them from loosening.

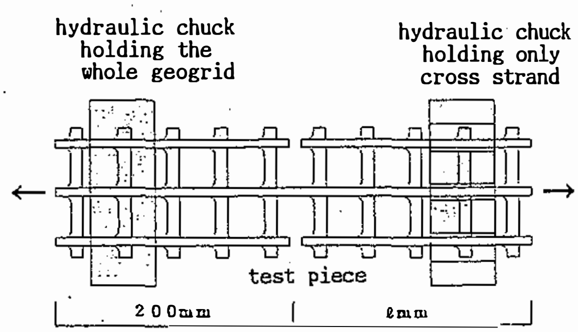


Fig.5 Bonding strength test

Table 2. Bonding strength test results

| test piece No. | | length of bonding (ℓ in Fig.5) 104mm | | 208mm | | 260mm | | 312mm | |
|---|---|---|---|---|---|---|---|---|---|
| bonding strength (kN/one strand) | 1 | 1.14 | × | 1.71 | × | 1.87 | ○ | 1.80 | ○ |
| | 2 | 1.25 | × | 1.81 | ○ | 1.74 | ○ | 1.84 | ○ |
| | 3 | 1.28 | × | 1.69 | × | 1.84 | ○ | 1.79 | ○ |
| | 4 | 1.29 | × | 1.92 | ○ | 1.92 | ○ | 1.89 | ○ |
| | 5 | 1.36 | × | 1.96 | ○ | 1.91 | ○ | 1.84 | ○ |
| | av. | 1.26 | | 1.82 | | 1.86 | | 1.83 | |

○: breakage
×: coming-off of a counterpart element

## 2.2 Creep characteristic test

Through the creep characteristic test, each test piece underwent measurement the magnitude of a deformation which it had suffered under continuous application of some load over long hours, with the creep characteristic of the ADEAM thereby ascertained. Presented in Figs.6 and 7 respectively are the test data of a relation between the magnitude of creep-originated strain and the lapse of time, and another relation between creep rupture strength and the time consumed before the start of rupture. Referring to Fig.6, it is clear that ADEAM (G-6) incurs rupture when the magnitude of strain grows up to approximately 4%. It is further estimated that the strain corresponding to a live load of 34.5kN/m (equivalent to 73% the standard tensile strength of each ADEAM) would not outgrow a limit of rupture even after the lapse of $10^6$ hours (approx. 100 years). As given in Fig.7, the creep rupture strength which each ADEAM shows following the lapse of $10^6$ hours (approx. 100 years) is estimated to be of such a magnitude defined around the dotted line in the figure, and large for its standard tensile strength.

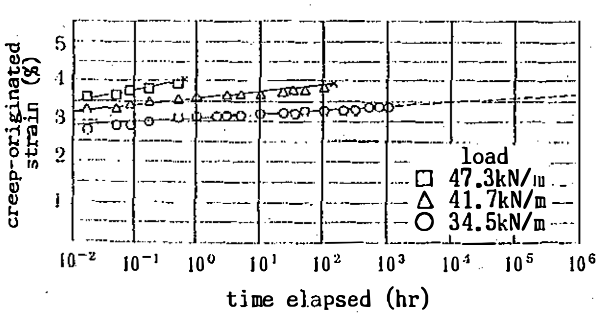


Fig.6 Magnitude of creep-originated strain and the lapse of time (G-6)

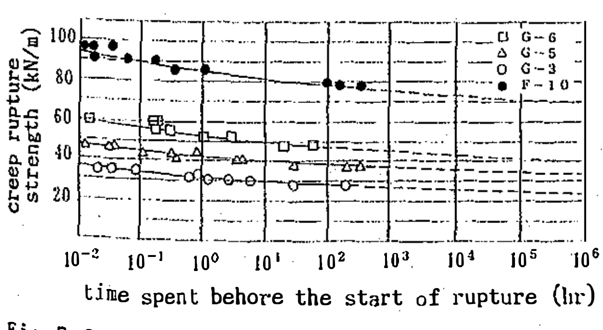


Fig.7 Creep rupture strength and time of rupture

## 2.3 Durability test

1. On-site installation shock resisitance test: To quantitatively grasp how much is the intensity of rupture which each ADEAM would undergo, and how much is the decline of tensile strength that the ADEAM would suffer in case where the soil for use in banking is found with a large gravel content, we implemented an on-site installation shock resistance test. ADEAM was burried 10cm (test 1) or 20cm (test 2) below the ground surface. The shock load was given by driving a wheel loader (test 1) or hydrauric shovel (test 2). No ADEAM exhibited a trace of rupture in the post-test inspection following removal from a shock resistance tester. It was further noted that as shown in Table 3, the strength maintainability of each ADEAM is as high as over 90%.

Table 3. Strength maintainability of ADEAM during the installation

| banking materials | | type | (1) | (2) | (3) |
|---|---|---|---|---|---|
| test 1 | gravel | ADEAM G-5 | 62.7 | 59.4 | 94.7 |
| | size 10cm | ADEAM F-10 | 105.8 | 97.6 | 92.2 |
| test 2 | gravel 93%<br>sand 6%<br>silt 1%<br>max. size 2.54cm | ADEAM G-6 | 64.2 | 62.2 | 96.9 |

(1) initial strength (kN/m)
(2) post-test strength (kN/m)
(3) strength maintainability (%)

2. Weather proofness test: The weather proofness test which we have implemented, using a sunshine weather meter clarified that long exposure to sunshine has a very slight effect upon the reduction of tensile strength (see Fig.8).

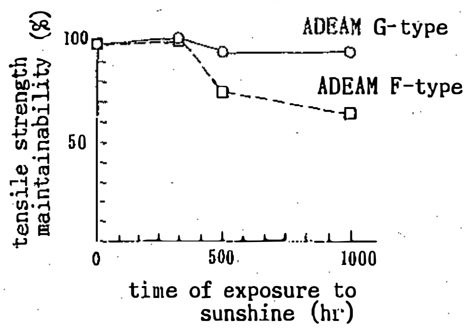


Fig.8 Weather proofness test results

## 3 TEST EMBANKMENT

### 3.1 Outline of the geogrid installation

Figs.9 and 10 show a cross section, respectively of A and B zones.

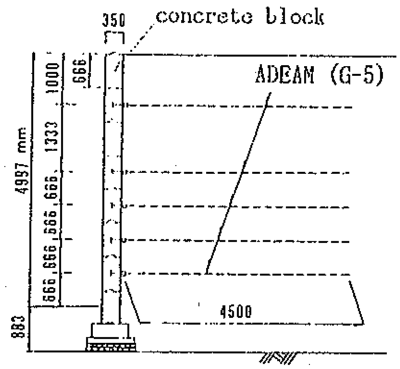


Fig.9 A-cross section

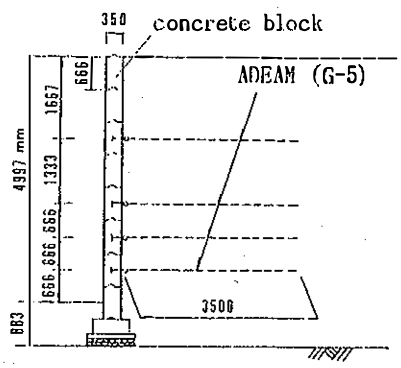


Fig.10 B-cross section

Covered in this paragraph are the particulars of the geogrid test installation done at A and B zones, using large concrete blocks. To unite together a plurality of large concrete blocks, each being 35cm thick, 150cm wide, and 66.6cm high, and measureing a weight of approx. 6.86kN, a 22mm diameter reinforcing steel bar was inserted into each filler space provided at an every pitch of 75cm, with concrete pouring therein to follow. The banking material involved was pit sand, and in preparation for the test embankment, pit sand was laid down and spread to a thickness of 30cm. Then, the pit sand went through roll-pressing and the on-site density thereof was measured according to the sand filling method so that the pit sand layer would be handled in a state of given compactness. Table 4 shows the physical characteristics of the banking material (pit sand). The shear strength of this banking material was determined, depending on the consequences of a tri-axial compression test. Taking into account the fact that the water contention per cent of dry weight (approx. 8.5%) found with the banking material used for the tri-axial compression test is considerably lesser compared with that (approx. 13.8%) of the soil at some work site, the shear strength of the former would have possibly been estimated rather large. The foundation ground was soft clayey soil. Presented in Fig.11 are the results of the in-situ cone-penetration test done prior to test banking. A consequent comparison was made between two cases of A and B zones which aredifferent with regard to the quantity of installed ADEAM, the space between mutually adjacent ADEAM installed at respective positions, and the length of ADEAM serial installation, as specified in Figs.9 and 10.

Table 4. Physical characteristics of the banking material

| | | |
|---|---|---|
| wet denesity | $\rho_t$ | 1.8Mg/m³ |
| cohesion | $c_d$ | 39.2kPa |
| friction angle | $\phi_d$ | 37.0° |
| optimum moisture content | $\omega_{opt}$ | 20% |
| maximum dry density | $\rho_{dmax}$ | 1.59Mg/m³ |

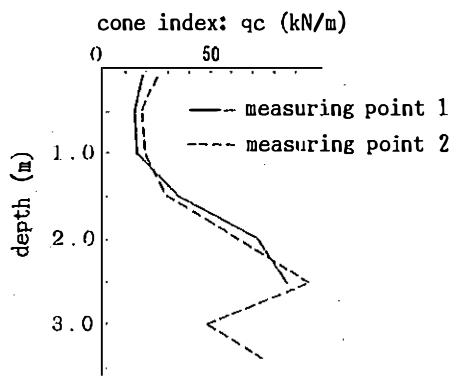


Fig.11 Results of cone-penetration test

### 3.2 Contents of instrumentation

Each of the walls of banked soil respectively in A and B zones was rendered cross-sectional instrumentation, followed by a consequent comparison there between and a post-comparison instrumentation data analysis. Enumerated below are the instrumentation content particulars:

1. Horizontal displacement of a wall face
2. Settlement of a concrete foundation
3. Settlement of banked soil
4. Displacement of the ground adjacent to banked soil
5. Vertical earth pressure of banked soil
6. Strain in the ADEAM, and so forth

Figs.12 and 13 illustrate the cross sections of the respective walls provided in A and B zones for due instrumentations (refer to Table 5).

Table 5. A list of instruments

| symbols | instruments |
|---|---|
| | strain gauge (for geogrid) |
| | horizontal rod |
| | settlement plate |
| △ | earth pressure gauge |
| | displacement gauge pile |

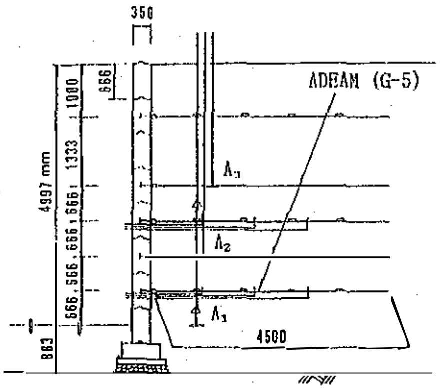


Fig.12 A wall cross section (A zone) through with instrumentations

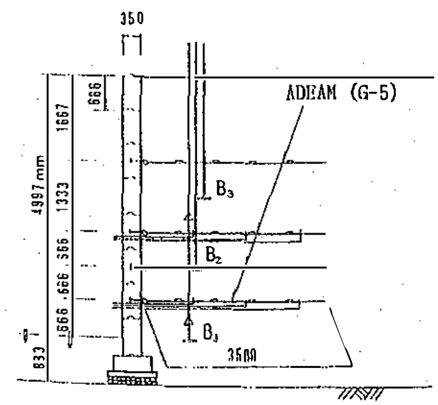


Fig.13 A wall cross section (8 zone) through with instrumentations

### 3.3 Results of measurement

The interim measurement results obtained approximately 40 days after banking to a certain elevation are such as given below:

1. Horizontal displacement of a wall face: Given in Fig.14 are the findings from the measurement of a wall face horizontal displacement. Total displacement - inclusive of a horizontal relocation of the concrete foundation - which the wall face had incurred approximately 40 days after banking with the ADEAM applied, was 9.8cm outward in A zone while B zone marked a total displacement of 10.7cm with the wall face. As is clear from the above, the total displacement of the wall face in A zone where banking was done with the inter-ADEAM space set lesser than in B zone, is smaller as compared with that recorded in B zone. Further, noting that the wall face displacement after banking to a certain elevation was 1.3cm outward in A zone while the displacement in B zone was 1.7cm, it is considered that the wall of banked soil would almost be stabilized with the serial block inclination angle getting fixed, becoming free from undergoing additional displacement.

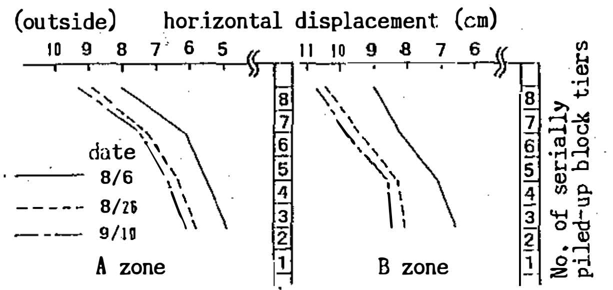


Fig.14 Wall face horizontal displacement measurements

2. Settlements of the concrete foundation and banked soil: Fig.15 gives the outcomes of settlement measurements respectively with the concrete foundation and banked soil. Since the foundation ground was of soft clayey soil commonly with the concrete foundation and banked soil, both settled approximately to the same extent. To be more precise, the settlement amounts observed at the time of our measurement were about 25cm with the concrete foundation, and around 28cm with the banked soil.

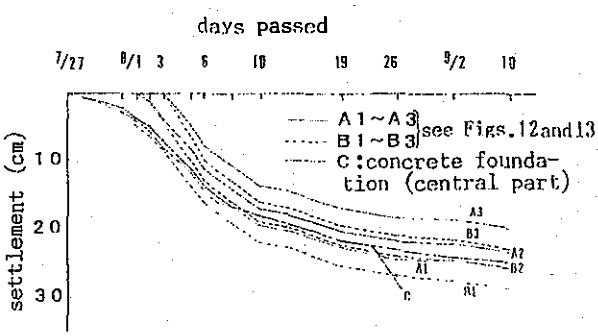


Fig.15 Settlements respectively of the concrete foundation and banked soil

3. Displacement of the ground adjacent to banked soil: Presented in Fig.16 is the vertical displacement witnessed with the ground adjacent to banked soil. It is noted that there is a propensity wherein the foundation ground in the neighbourhood of banked soil is subject to subsidence along with the settlement of the banked soil.

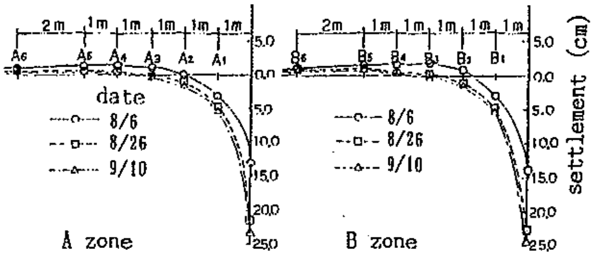


Fig.16 Vertical displacement measurements of the ground adjacent to banked soil

4. Vertical earth pressure of banked soil: Fig.17 shows the measurements of vertical earth pressure of banked soil. Taking a look at the figure, it is found that commonly in A and B zones, the vertical earth pressures of respective banked soils are approximated to the gravity of each banked soil.

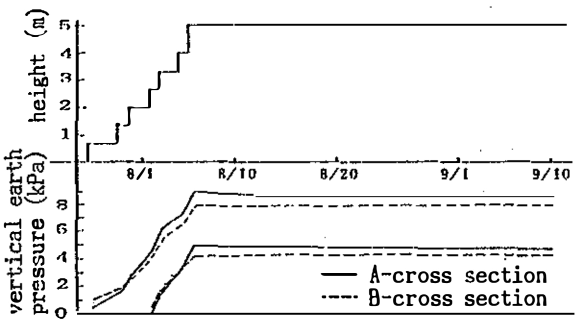


Fig.17 Vertical earth pressure measurements at respective points of banked soil

5. Strain out with ADEAM: Presented in Fig.18 are the measurements of strain out with each ADEAM, which were obtained by using a strain gauge and a horizontal rod. Referring to the figure, it is clarified that the measurements of those strains developed in the ADEAM used in banking respectively in A and B zones are approximately the same, wherein the maximum magnitude of strain is around 1%. Converting the magnitude of strain out with the ADEAM reversely into the tensile force per 1m wide ADEAM, with reference to Fig.3 discloses that the tensile force is rather small, ranging approximately from 4.9 to 5.88kN/m.

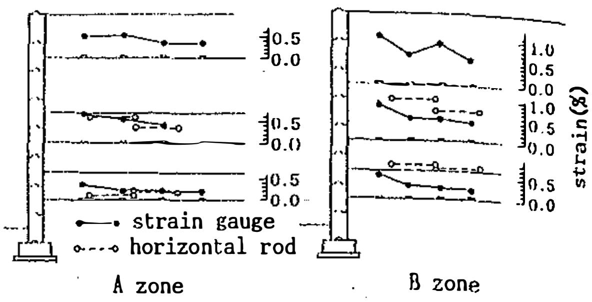


Fig.18 Strain distribution across the ADEAM

## 4 CONCLUSION

By way of the performance verification test done for the FRTP geogrid (ADEAM), we could draw a conclusion that the ADEAM with the tensile strength characteristic, creep characteristic, resistivity to impacts rendered during banking, and durability, each turned out greatly favorable proves capable of providing due serviceability as a banking reinforcement. Likewise via the point-of-intersection strength test designed to make sure unitary integrity between the cross and longitudinal strands, the respective elements making up a composite reinforcement for banking, and the core bonding strength test, it was further found that the FRTP-geogrid is duly serviceable, particularly with availability of the above-remarked unitary integrity, and excellent core bonding strength. We are contemplating to improve from now on the bonding technique thereby to provide a banking reinforcement with more of unitary integrity ensured between the cross and longitudinal strands. The test embankment which we have undertaken of late, left behind no problem over the application of ADEAM, for it is proven capable of providing greatly excellent on-site installation feasibility. The instrumentation data presented herein are neither final nor decisive at the present stage whereat no more data than those in the interim report are available but considering the horizontal displacement which the vertical wall of soil banked with blocks piled up undergoes, and the magnitude of strain out with each ADEAM, and so forth, the subject civil engineering process seems to find sufficient applicability with banked soil structures.

## REFERENCE


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