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Applications of Geosynthetics in Embankments- A Review

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Abstract: Construction and maintenance of embankments form an integral part of every highway or railway projects which are keystones of the economy of our nation. In many locations good soil will not be available for the foundation and also for the construction of embankment. In such cases reinforcing soil with geosynthetic is a very effective and economic option. This paper thoroughly reviews the improvements in load settlement behaviour of embankments attained by inclusion of various forms of natural and polymeric geosynthetics. It is seen from literature that natural geosynthetics made with coir or jute can also be effectively used for improving the durability and stability of embankments.

Keywords: Embankments, Natural geosynthetics, Polymeric geosynthetics, Stability of embankments.

I. INTRODUCTION

Geosynthetics have become well established construction materials for geotechnical and environmental applications in most parts of the world. Geosynthetic-reinforced structures have been used worldwide due to their successful performance and economic efficiency. Over the years, these products have helped designers and contractors to solve several types of engineering problems where the use of conventional construction materials would be restricted or considerably more expensive.

Common types of geosynthetics used for soil reinforcement include geotextiles (particularly woven geotextiles), geogrids and geocells. Geotextiles are continuous sheets of woven, nonwoven, knitted or stitch-bonded fibers or yarns. The sheets are flexible and permeable and generally have the appearance of a fabric. Geogrids have a uniformly distributed array of apertures between their longitudinal and transverse elements. These apertures allow direct contact between soil particles on either side of the sheet. Geocells are relatively thick, three-dimensional networks constructed from strips of polymeric sheet. The strips are joined together to form interconnected cells that are filled with soil and sometimes concrete. In some cases 0.5 m to 1 m wide strips of polyolefin geogrids have been linked together with vertical polymeric rods used to form deep geocell layers called geomattresses.

When it happens, embankment collapse can be disastrous causing serious loss of life, money and time. Reconstructing collapsed embankments can be very costly and from a purely economic standpoint, it would be more beneficial to reinforce the embankment so that it does not fail rather than reconstruct.Geosynthetic materials are used widely in embankments to increase stability. Geotextile layers increase the embankment stability by virtue of two primary functions: tensile reinforcement and as a drainage element reducing pore pressures.

II. NSTRUCTION OF EMBANKMENTS USING GEOSYNTHETICS

Geosynthetic-reinforced soil structures are widely used to support bridge abutments and approach roads in place of traditional pile supports and techniques. In such situations, foundation conditions have been shown to adversely affect the stability and deformation behaviour of overlying geosynthetic-reinforced slopes and walls. In the last two decades the use of geosynthetic materials for reinforcing slopes and retaining walls in fill has increased significantly throughout the world. In wall and slope applications the function of the geosynthetic layers is to provide resistance to driving forces or moments caused by the self-weight of the soil and applied surcharges. Generally, the behaviour of geosynthetic-reinforced soil structures is being studied with respect to: (i) self-weight loading and surface loads; (ii) seepage forces; (iii) earthquake loading; and (iv) subsoil conditions. It is highly desirable to understand the behaviour of geosynthetic reinforced slopes both at pre-failure and at failure.

The I-15 Reconstruction Project in Salt Lake City, Utah required rapid embankment construction in an urban environment atop soft lacustrine soils. These soils are compressible, have low shear strength, and require significant

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time to complete primary consolidation settlement. Because of this, innovative embankment systems and foundation treatments were employed to complete construction within the approved budget and demanding schedule constraints. In this paper we are discussing one of the methods used for the construction of the embankment.

Construction of large walls and embankments are challenging in soft soils in urban areas. Special care is taken in such a way that primary consolidation and post construction settlement should not affect the adjacent structures or areas. This means that methods should be adopted to minimize settlement and ensure safety to the embankments.This can be accomplished either by using a smaller loading condition or by altering the foundation conditions to withstand the required load. In either case, the net goal is to reduce the potential settlements to an acceptable magnitude. One such method is by the use of geofoams. Geofoamseliminate any potential foundation settlement by acting as a light-weight fill embankment and, thus, greatly minimize the loading condition imposed on the foundation soils.

Expanded polystyrene geofoam has been used as a light weight embankment fill since at least 1972, where it was used for a roadway project in Norway. Subsequently, use continued throughout Scandinavia and began to spread to the rest of Europe and

Japan. In Japan, the first lightweight fill project using geofoam occurred in about 1985, but after 10 years, Japan's use comprised approximately 50% of worldwide usage.

Construction with geofoam blocks is fairly a straightforward process. The site is first levelled and a layer of bedding sand is placed. Geofoam blocks are then stacked with additional bedding sand filling the gap between the geofoam and the back slope. A load distribution slab consisting of reinforced concrete is constructed atop the geofoam, followed by a small layer of fill, and finally the pavement section. A tilt-up panel wall is placed to cover and protect the exposed face.

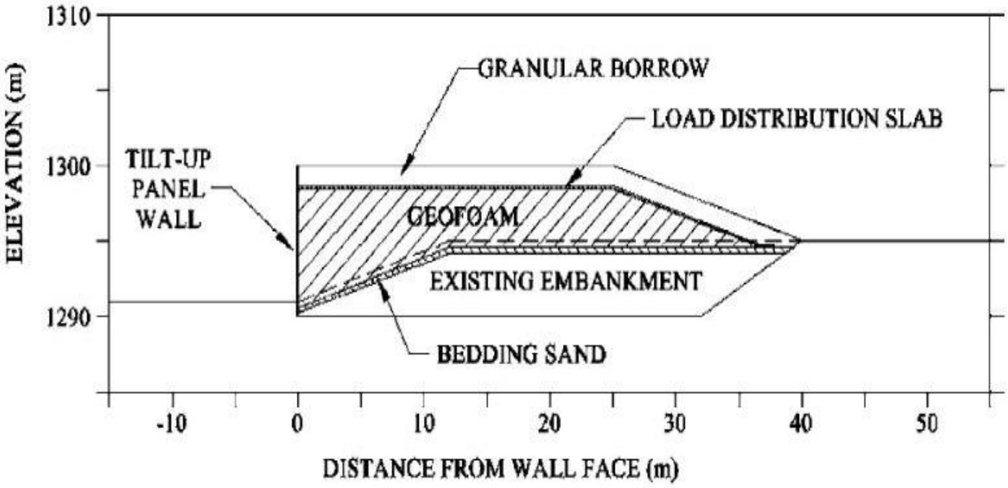

Fig1. Construction using Geofoam Blocks

EPS geofoam with a nominal density of 20 kg/m3 was used for the lightweight embankment construction on the I-15 Reconstruction Project. The contract specifications did not require trimming of the geofoam block by the manufacturer. As necessary, individual geofoam blocks were cut on site to desired shapes and sizes. The average unconfined compressive strength at 10% strain of standard 50 mm cube samples was 110 kPa. A working stress of 40% of the average strength at 10% strain was allowed for the overlying fill, pavement pressure, and transient loading.Approximately 100,000 m3 of geofoam embankment was placed on the I-15 Project at several localities.The primary use of geofoam on the I-15 Project was as lightweight embankment over existing buried utilities to minimize settlements.Geofoam embankment was also used to expedite the construction in a few critical locations where the project schedule did not allow for conventional embankment construction and the requisite 6 to 12 month waiting period for accelerated primary consolidation settlement with PV drains. The use of geofoam at these locations completely eliminated the settlement time associated with placement of conventional embankment.

A. Numerical Modelling of Geofoam Embankments

In 2001, the Utah Department of Transportation completed a 4-year $1.4 billion I-15 reconstruction project in Salt Lake City, Utah. That project included widespread use of expanded polystyrene geofoam as lightweight embankment at important utility crossings and where close proximity to existing buildings necessitated minimizing consolidation

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settlement. The writers used a bilinear elastic model. The placement of this extremely lightweight material, with a density of 18 kg/m3, allowed rapid construction of full-height embankments in a short period of time. Fig. 1 shows the partially completed construction of a typical geofoam embankment for the I-15 reconstruction project.

Fig2. Typical geofoam embankment construction on the I-1 reconstruction project in Salt Lake City

The modelling was performed using fast Lagrangian Analysis of Continua FLAC, a general finite-difference program for geomaterials. Results of this study will improve the understanding and estimation of the stress distribution and vertical displacement of geofoam embankments. Vertical displacement in a typical geofoam embankment is due to a combination of gap closure, seating, and elastic compression of the geofoam blocks.

They generalized the typical construction cross section for a geofoam embankment to five layers, starting from the bottom, the first layer consisted of a minimum of 0.3 m of base sand that was graded and levelled for the placement. Two materials made up the second layer: the pre-existing granular embankment, graded at a 1.5H: 1V 33.7° backslope, and the adjacent geofoam, which abuts the existing embankment. Layer three consisted of a 0.150-m-thick reinforced concrete load distribution slab LDS, used to protect the geofoam from local overstressing. Layer four was an untreated pavement base course UTBC, about 0.610 m thick and layer five was an unreinforced Portland cement concrete pavement PCCP, which was generally 0.356 m thick. After placement of the geofoam embankment, a full-height tilt-up prefabricated concrete panel wall was placed in a slotted strip footing.

1. Modelling Approach:
First, the particular geometry for the entire embankment was inputted using construction cross section drawings at the instrumentation array locations. Then, the PCCP, UTBC, LDS, geofoam, and backfill layers were given null properties in the FLAC model and the model was allowed to come to equilibrium. The entire geofoam mass was placed at once in the model. After placing the geofoam, FLAC again was allowed to come to static equilibrium. This represented the placement and resulting compression of the geofoam and backfill due to their self-weights. This step also produced additional displacement in the base sand from the placement of the geofoam and backfill. The stresses produced in this step resulted from the weights of the geofoam, backfill, and base sand. Similarly, each subsequent layer in the model was incrementally added to the model and the model was allowed to come to static equilibrium. To create numerical stability, we fixed the side boundaries of the FLACmodels in the horizontal direction and fixed the bottom of the FLAC models in the vertical direction. The base of the model was set at 10 m below the first geofoam block layer.

2. Results:
The differential displacements between layers, were measured shortly after the final dead loads i.e., LDS, UTBC, and PCCP were placed.

FLAC produced reasonable estimates of the field measurements, both in terms of vertical and horizontal stress distributions and vertical displacement. We believe that numerical modelling provides valuable insight into the behaviour and design of these complex multi-layered embankment systems. The results of this study and additional FLAC modelling was subsequently employed to evaluate the sliding stability of geofoam embankments subjected to strong ground motion resulting from major earthquakes.

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III. PILED EMBANKMENTS IN SOFT SOILS

One of the major obstacles to construction of embankments over soft soil is settlement. A number of techniques have been developed in order to deal with these anticipated settlements. A popular and widely used option is to build the embankment on a grid of piles or columns that are driven or constructed to a more competent underlying layer, such as bedrock. Piled embankments provide an economic solution to the problem of constructing embankments over soft soils. The piles and geosynthetic combination can alleviate the uneven surface settlements that sometimes occur in embankments supported by piles without reinforcement. The embankment load is transferred almost entirely to the piles, and then to the competent layer. Thus the problems associated with soft soils can be avoided. It has been found that the addition of one or more geogrid layers at the base of the embankment, just above the piles, facilitates the transfer of the embankment load to the piles, and allows for greater pile spacing. The mechanism that allows load transfer in this technique is soil arching.

Arching occurs when there is a difference of the stiffness between the installed structure and the surrounding soil. If the structure is stiffer than the soil then load arches onto the structure. Otherwise, if the structure is less stiff than the soil then load arches away from the structure. If part of a rigid support of soil mass yields, the adjoining particles move with respect to the remainder of the soil mass. This movement is resisted by shearing stresses which reduce the pressure on the yielding portion of the support while increasing the pressure on the adjacent rigid zones. This phenomenon is called the arching effect. The load from the embankment must be effectively transferred to the piles and to prevent punching of the piles through the embankment fill creating differential settlement at the surface of the embankment. If the piles are placed close enough together, soil arching will occur and the load will be transferred to the piles more effectively. In the theoretical study of the above technique, the following simplifications are used:

a) The embankment fill is homogeneous, isotropic, and cohesion less.
b) The soft soil ground is also homogeneous, isotropic, and cohesive.
c) The soft soil and the embankment fill deform only vertically.
d) Piles are sufficiently rigid and undergo insignificant deformation.
e) There is no friction between a pile and the surrounding soft soil.
f) The ratio of the embankment fill height to the center-to-center spacing is greater than 0.5.

In the analysis of this technique, a uniform surcharge load over the embankment was also included.

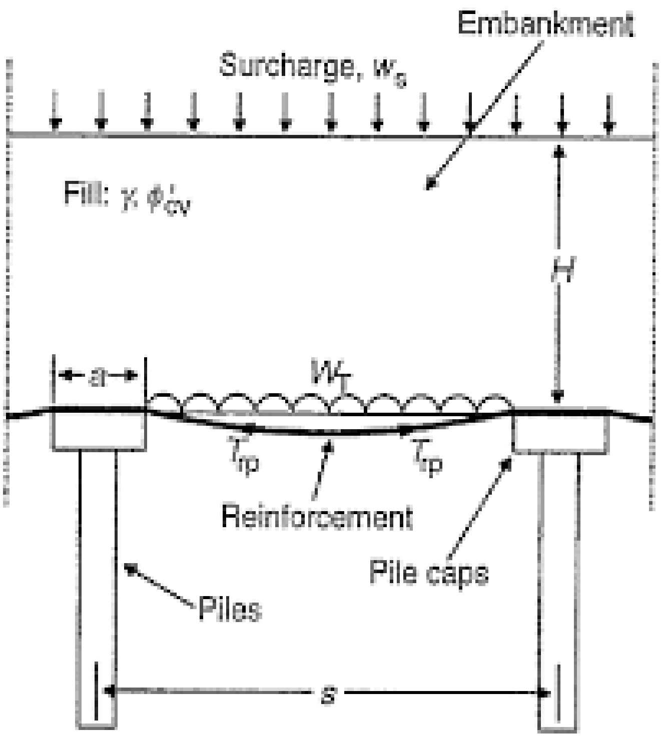

Fig.3 Geosynthetic overlying pile caps.

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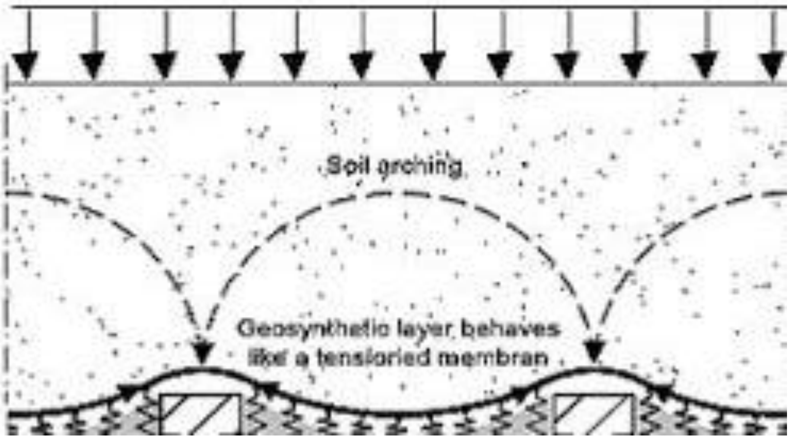

Fig.4 Soil arching

Fig.5 shows the influence of soft ground depth on efficiency. It can be seen that for embankments of smaller depths, the efficiency can be drastically increased with the usage of geotextiles. Also, efficiency increases with increasing area ratio. The efficiency for reinforced case is higher than that for unreinforced case because geosynthetic enhances the load transfer from the soil to the pile caps.

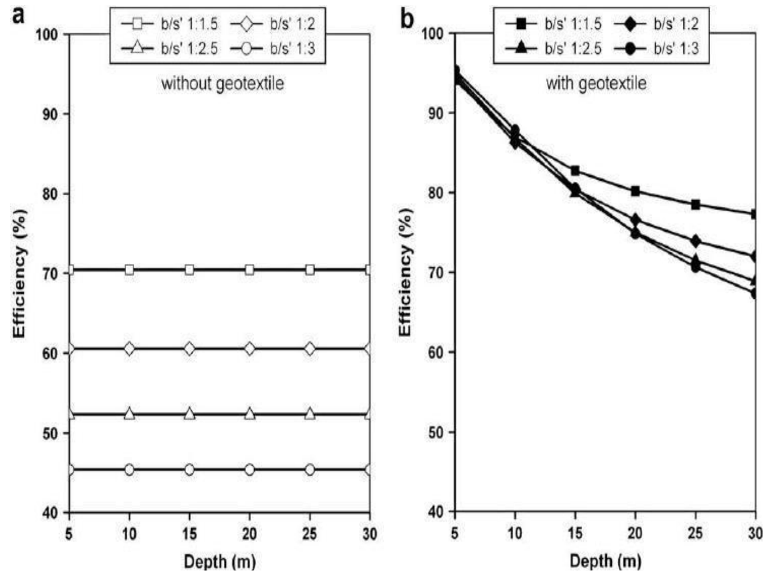

Fig.5 Effect of embankment depth on efficiency

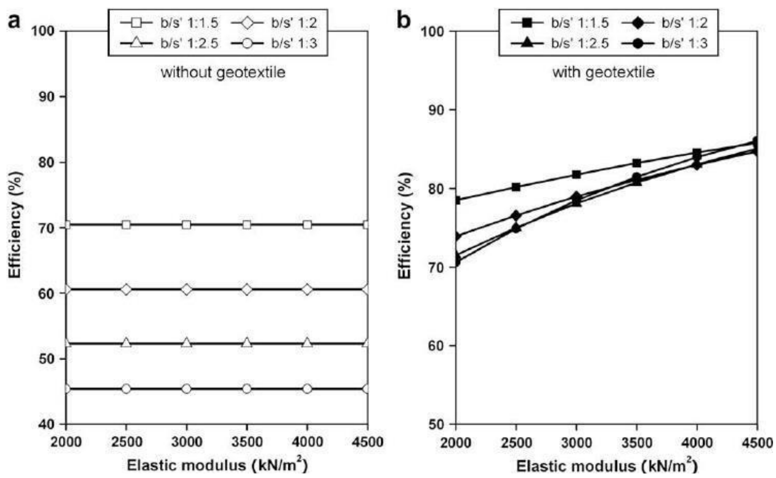

Fig.6 Effect of soft ground elastic modulus on Efficiency

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Fig.6shows the influence of soft ground elastic modulus on Efficiency. It can be seen that efficiency increases with increasing area ratio. It also can be seen that efficiency increases with increasing soft ground elastic modulus. The efficiency for reinforced case is higher than that for unreinforced case due to that the geosynthetic enhances the load transfer from the soil to the pile caps.

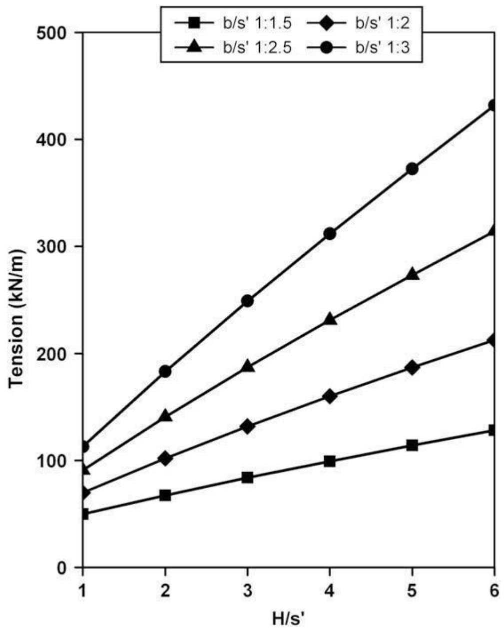

Fig.7 Effect of embankment height on tension of Geosynthetic

Fig.7shows the influence of embankment fill height on tension of geosynthetic. The tension of geosynthetic decreases with increasing area ratio. Also, tension of geosynthetic increases with increasing embankment fills height.

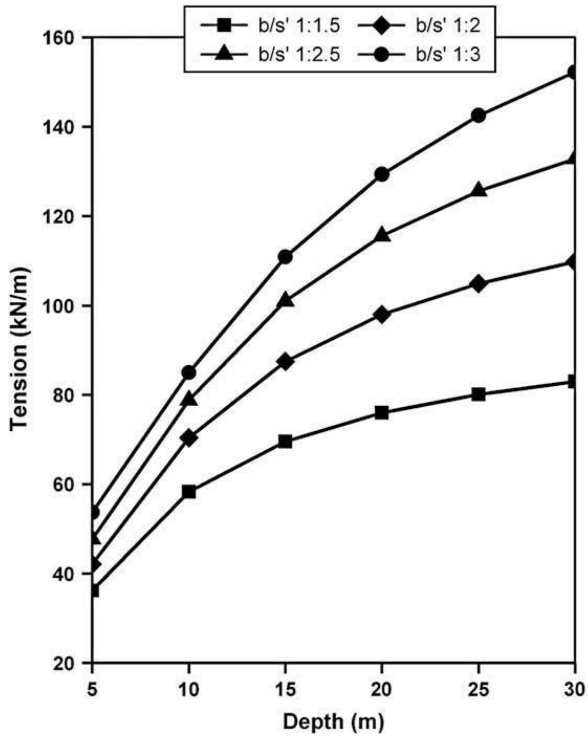

Fig.8 Effect of soft ground depth on tension of Geosynthetic.

Fig.8shows the influence of soft ground depth on tension of geosynthetic. It is seen that geosynthetic tensionincreases with increasing soft ground depth.

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The above method of analysis shows that inclusion of a geosynthetic membrane can increase the fill load carried by piles, as indicated by efficiency. The method also shows that the portion of the fill load carried by piles increases with the area ratio of pile caps. For a given area ratio, the efficiency reaches a maximum value when the ratio of the thickness of the fill to the spacing of the pile caps is large.

IV. COUPLED MECHANICAL AND HYDRAULIC MODELLING OF GEOSYNTHETIC-REINFORCED COLUMN-SUPPORTED EMBANKMENTS

Geosynthetic-reinforced column-supported GRCS embankments have increasingly been used in the recent years for accelerated construction. To investigate the time-dependent behavior, coupled two-dimensional mechanical and hydraulic numerical modelling was conducted in this study to analyze a well-instrumented geotextile-reinforced deep mixed column-supported embankment in Hertsby, Finland. In the mechanical modelling, soils and DM columns were modelled as elastic-plastic materials and a geotextile layer was modelled using cable elements. In the hydraulic modelling, water flow was modelled to simulate generation and dissipation of excess pore water pressures during and after the construction of the embankment.

In this system, columns such as deep mixed columns, vibroconcrete columns, stone columns, and aggregate piers provide major support for the embankment over soft soils and reduce its settlement. One high-strength geosynthetic layer or multiple low-strength geosynthetic layers mostly geogrid are used above the columns to enhance the load transfer from soft soils to columns.

A. Description of Selected Case Study

The soils from the ground surface consisted of 1–1.5-m of crust, 10–14-m of soft clay, 1–6-m of silt, and 1–5-m of glacial till. The soft clay, considered as the problematic layer, had an un-drained shear strength of 10–15 kPa and the effective cohesion and friction angle of 8 kPa and 13°, respectively, determined from drained tri-axial tests. The tangential elastic moduli under drained and un-drained conditions were 300–600 kPa and 3,000–8,000 kPa, respectively, also determined from the tri-axial tests at the confining stresses corresponding to the in situ stresses. The determined Poisson’s ratio under a drained condition was 0.1– 0.2. The embankment section consisted of a 0.05-m-thick asphalt layer, 0.20-m-thick crushed stone base course, 1.05-m-thick gravel sub-base, and 0.50- m-thick sand working platform above the existing ground. The ground water table was close to the existing ground surface. The ultimate strength of this geotextile was 200 kN/m in both longitudinal and transverse directions. The secant stiffness of this geotextile layer was 1,790 and 2,120 kN/m at strains of 2% and 6%, respectively. They were alternately installed into two patterns walls and isolated columns parallel to the centreline of the embankment. For the convenience of construction, the column walls were constructed parallel to the centreline of the embankment instead of perpendicular to the centreline.

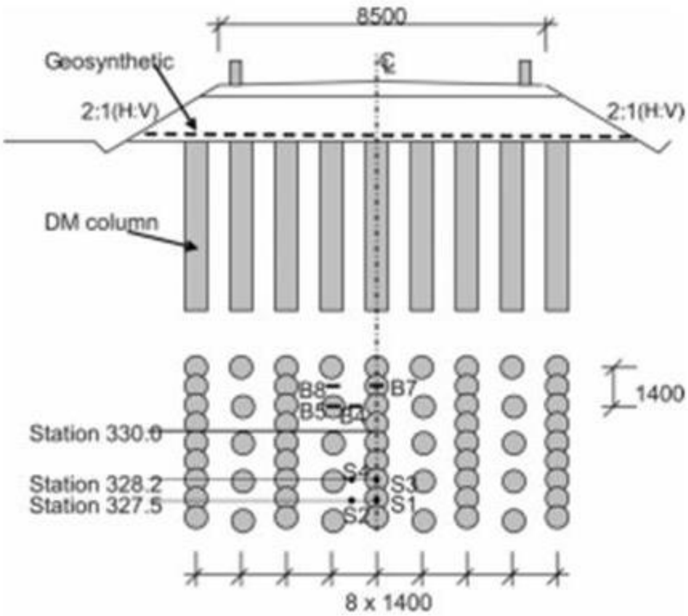

Fig9. Embankment cross-section and column layout

B. Numerical Modelling

The soil layers and the DM columns were extended to the depth of the firm glacial till. No deformation below the silt layer was assumed. The right side i.e., the centreline of the embankment boundary was assumed impervious considering no water flow into and out the symmetry plane. The left-side boundary was assumed impervious because it is quite far from the embankment and expected to have an insignificant effect on the computed results. The bottom was also assumed impervious to account for low permeability of the glacial till. Cable elements were used to simulate the geotextile layer, which was located 0.3 m above the top of the DM columns. The construction of the embankment was modelled in three stages. The traffic was simulated by applying an equivalent static, distributed load of 12 kPa on the crest of the embankment, which was assumed starting right after the placement of the asphalt layer in Stage 3, i.e., the traffic loading was included in Stage 3. After the completion of Stage 3 the modelling was extended to the end of the designed service life assuming 30 years in this study to yield the final post construction settlement. In Fast Lagrangian Analysis of Continua FLAC, the deformation-diffusion process of coupled mechanical and hydraulic modelling includes two loops: mechanical and hydraulic loops. Starting with a hydraulic loop, pore water pressure change is calculated. The volumetric strain increment from the mechanical loop is then conveyed back to the hydraulic loop. As the two loops keep exchanging data, the coupled modelling is implemented. In this study, each mechanical loop was determined to cycle 100 times or reach the predetermined equilibrium ratio limit 1×10-4, whichever came first to bring the system into a quasistatic condition, while a hydraulic loop was determined to cycle once. The coupled modelling was terminated when the specified consolidation time was reached. Pore water pressures, effective stresses, total stresses, consolidation time, and settlements at specified locations were recorded during the numerical run. Almost no difference in the calculated settlement was found between the boundary distances of 15 and 20 m. Therefore, the left boundary was determined to be at 15 m from the center of the leftmost column. After the left boundary was determined, the mesh sizes were reduced to half of the original sizes in both directions and less than 2% difference in the calculated settlement was found.

C. Results and Comparison

There was an immediate settlement right after the application of each load. Then the settlement became relatively stable and continued increasing at a slow rate after 2 years since the construction. This fast process of consolidation can be explained that the excess pore water pressure dissipates hydraulically and mechanically, i.e., through drainage and load transfer. On the other hand, due to the columns having a higher modulus than the soft soil, the soft soil tended to settle more than the columns and more load was transferred from the soft soil to the columns. Therefore, the stress on the soft soil decreased with time, which induced the dissipation of excess pore water pressure. This dissipation was caused by unloading on the soft soil. With the inclusion of geosynthetic reinforcement, the load transfer effect in reducing excess pore water pressure is even more efficient.

It is shown that the excess pore water pressure suddenly increased after each load and then dissipated gradually with time. This gradual dissipation of the excess pore water pressure represented a consolidation process.

It can be seen at Stages 1, 2, and 3 that only a small portion of the soil near the bottom slightly heaved. However, as the consolidation proceeded, all the displacements became negative, i.e., compression.

The influence of the geosynthetic tensile stiffness on the maximum settlement became less significant when the tensile stiffness was higher than J= 1,700 kN/m. The higher tensile stiffness of the geosynthetic reinforcement yielded a larger maximum tension. Another interesting finding is that as the consolidation proceeded from 1.5 to 30 years, the tension was slightly reduced.

This result can be explained by the larger increase of the settlement on the DM walls than that on the soft soil under the full height of the embankment from 1.5 to 30 years. The change of the tension in the geosynthetic reinforcement depends on the combined effect of more or less settlement increase in DM walls and lateral movement of the soft soil.

V. PERFORMANCE OF A GEOGRID-REINFORCED AND PILE-SUPPORTED HIGHWAY EMBANKMENT

OVER SOFT CLAY: CASE STUDY

Conventional piled embankment construction i.e., piled embankments without geogrid reinforcements requires closely spaced piles or large pile caps to transfer most embankment loads to the piles through soil arching. In order to place the relatively expensive piles as far apart as possible, a relatively inexpensive geogrid material is included at the base of the fill. This geogrid reinforced and pile-supported (GRPS) system has been used in several applications. Maddison et al. (1996) described an innovative system of ground improvement comprising vibroconcrete columns and a load transfer

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platform incorporating low-strength geogrids. The system was used to support a 6.0 m high embankment constructed over highly compressible peat and clay soils. Vibro-concrete columns and geogrids were used for widening an existing roadway. Based on the performance investigation of13 pile-supported and geogrid-reinforced earth platforms, Han and Gabr (2002) recommended that area ratio could be reduced to 10–20%, in comparison with the relative high area ratio of conventional piled embankments (50–70%). This is a case history of a GRPS highway embankment project in which a low improvement area ratio of 8.7%.

A. Site Conditions

The site is located in a northern suburb of Shanghai, China. The profile of the soil is as follows: there is a 1.5 m thick coarse grained fill overlying a 2.3 m thick deposit of silty clay; this deposit overlies soft silty clay that is approximately 10.2 m thick. Underneath the soft silty clay is a medium silty clay layer that is about 2 m thick followed by a sandy silt layer. The ground water level was at a depth of 1.5 m. The soft silty clay layer has a low to medium plasticity, and a liquidity index IL of 1.2. Its water content ranges between 40 and 50% and is generally close to the liquid limit. The uppermost coarse-grained fill layer has a relatively high preconsolidation pressure, in comparison with the underlying soft silty clay, which is normally consolidated or lightly over consolidated.

B. Geogrid-Reinforced and Pile-SupportedEmbankment

The embankment was 5.6 m high and 120 m long with a crown width of 35 m. The side slope was 1 V to 1.5 H. The fill material consisted mainly of pulverized fuel ash with cohesion of 10 kPa. The embankment was supported by cast-in-place annulus concrete piles that were formed from a low-slump concrete which were 16 m in length and were founded on a relatively stiffer sandy silt layer. The outer diameter of each pile was 1.008 m and the thickness of the concrete annulus was 120 mm. The design capacity of the pile was 600 kN and the key installation procedures of the pile are summarized as follows:

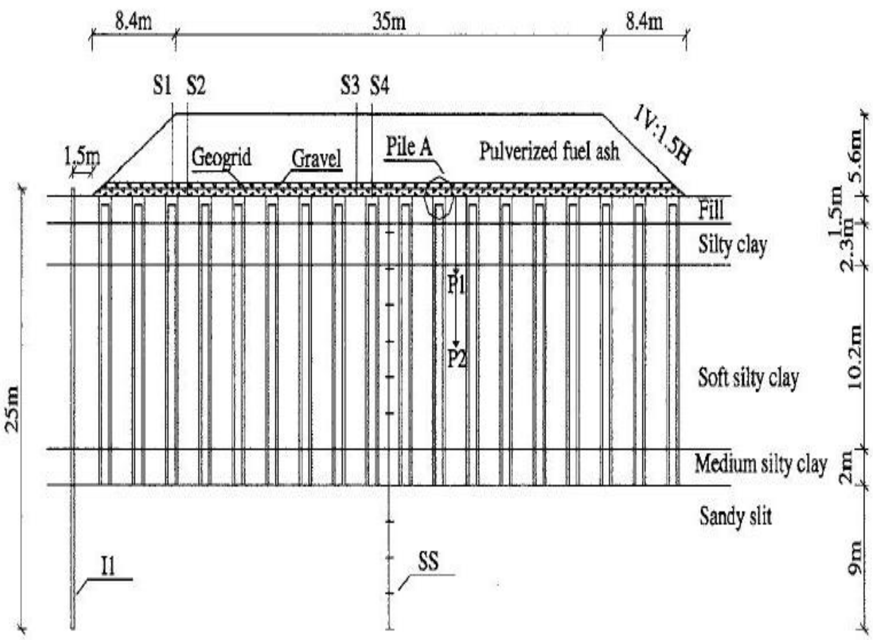

Fig10. Cross section of instrumented test embankment

1. A temporary double-wall casing is driven into the ground by a vertically vibrating driving machine. The double-wall casing consists of two concentric 8 mm thick steel pipes with different diameters. The outer and inner diameters of pipes are 1.016 and 0.76 m, respectively. This creates a 120 mm thick annulus between the outer and inner pipes for concreting. The inner pipe is open-ended whereas the annulus is fitted with a temporary conical-shaped driving shoe, which can be detached by wet concreting pressure during concreting. During driving the casing, soil is displaced into the inner pipe and outside the outer pipe. This creates a 120 mm thick annulus between the outer and inner pipes for in situ concreting.

2. During in situ concreting the annulus, the casing is withdrawn at a steady rate of 0.8–1.2 m/min. An appropriate concrete head varying from 0.3 to 0.5 m is always maintained within the annulus to provide stability, whereas the casing is withdrawn.

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3. After withdrawing the double wall pipe pile, a concrete plug is constructed by replacing the top 0.5 m of soil column inside the original inner pipe with concrete. This is to repair any possible damage to the top part of the annulus pile caused by withdrawing the double-wall steel casing. The annulus concrete piles were placed in a square pattern at a distance of three times the pile diameter (3 m) from the centre to the center of the adjacent piles. The area ratio, defined as the percent coverage of the pile caps over the total foundation area, was 8.7%. One layer of a biaxial polypropylene grid was sandwiched between two 0.25 m thick gravel layers to form a 0.5 m thick composite-reinforced bearing layer. The tensile strength in both directions of the geogrid is 90 kN/m and the maximum allowable tensile strain is 8%.

C. Details of Embankment Construction

In total, 740 piles were installed with tolerances of 150 mm in plane and 1% vertical alignment. Following the installation of the piles, a 0.25 m thick cushion layer of well-graded compacted gravel was laid over the piles to provide a working layer and to prevent the lower geogrid from mechanical damage above the pile heads. This layer was compacted using a light weight road roller. Vibration was not used in the compacting to minimize the risk of damaging the heads of the unreinforced piles. One layer of the TGGS90-90 biaxial geogrid was placed as an interlock with the granular fill. Another 0.25 m thick cushion layer was placed on the top of the geogrid. Thus, the height of the whole geogrid reinforced bearing layer was 50 cm. At the edges of the embankment, the geogrid was wrapped up and anchored back into the embankment over a 5 m length. Fig. shows the embankment construction history. The embankment was constructed to a height of 5.6 m over a period of about 55 day.

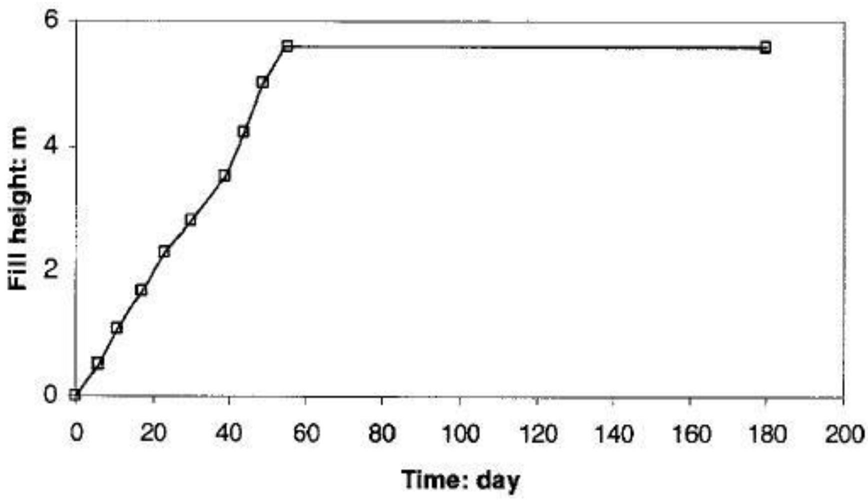

Fig11. Embankment height versus time

VI. COIR GEOTEXTILE

Unpaved roads are usually used for low volume traffic and serve as access roads. Being basically an agricultural country low volume roads play a very important role in the rural economy and resource industries in India. When unpaved roads are built on soft foundation soils, large deformations can occur, which increase maintenance cost and lead to interruption of traffic service. The use of geosynthetic products as an inclusion in flexible pavements for reinforcement has been demonstrated to be a viable technology through studies conducted over the last three decades which results in increased service life of the pavement or reduced base thickness to carry the same number of load repetitions. Benefits of reducing base course thickness are realized if the cost of the geosynthetic is less than the cost of the reduced base course material. In developing countries like India cost and availability of geosynthetics are the major constraining factors for the construction of reinforced soil structures. Hence alternative natural products are used to make the constructions cost efficient and eco friendly. But deterioration over time limits the use of natural geotextiles to temporary applications only. One of such applications can be in unpaved road over soft subgrade where the rate of plastic deformation (rut development) due to repeated traffic loads is faster during the initial stage and gets stabilized. In this case, it is expected that consolidation of the soft subgrade soil will make reinforcement unnecessary in the long-term. Natural fibre geotextiles can be a feasible solution in such applications where these products are meant to serve only during the initial stage and final strength is attained by soil consolidation due to passage of vehicles. These natural materials include coir, which is the husk of coconut, a common waste material where coconuts are grown and subsequently processed. Coir fibre is strong and degrades slowly compared to other natural fibres due to high lignin content. The degradation of coir depends on the medium of embedment and climatic conditions and is found to retain 80% of its tensile strength after 6 months of embedment in clay. Coir geotextiles are presently available with wide ranges of properties. Closely woven coir geotextiles possess high tensile strength and pull out resistance which can be economically utilized for temporary reinforcement purposes.

Coir Geotextiles offer a major solution for subgrade improvement and soil structure protection.

VII. CONCLUSION

The above paper deals with the applications of geosynthetics in embankments. The various aspects discussed are as follows:

1. Construction and numerical modelling of highway embankments at Salt Lake, U.S, using geofoam blocks.
2. Analysis and modelling of pile supported embankments reinforced with Geomembranes in soft soil.
3. Construction of geogrid-reinforced pile-supported highway embankment in Shanghai, China.
4. Usage of coir geotextiles as reinforcements in embankments.

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