FROM THE EDITOR'S DESK

First of all, I take this opportunity to wish all the members and readers a Very Happy and Prosperous New Year.

With the rapidly growing technological advancement, geotextiles have made inroads into a large variety of domains and have been acclaimed all over the world because of some of their built-in advantages like easiness and flexibility of use, softness (as compared to monolithic and rock constructions), rapidity of installation and long-term efficacy. Geotextiles provide a relatively safe and economically feasible solution in day-to-day engineering demands and construction challenges. As an alternate of natural materials, geotextile products perform a wide range of functions such as erosion control, soil stabilization, filtration, drainage, separation and reinforcement requirements.

Apart from the regular uses, geosystems made with geotextiles like geotextiles tubes, bags containers, etc., are playing a proactive role in hydraulic, coastal, offshore engineering and river protection works as eco-friendly, construction-friendly and cheaper alternatives of the traditional engineering solutions which generally are short lived, expensive and not eco-friendly.

Geosynthetics have exhibited successful applications across the globe in the areas of roads and pavement stabilization, embankment protection, ground stabilization, soil erosion control, landfills and waste management, etc. As our nation is on development mode, it becomes even more important to pay attention to ensure that our initiatives in infrastructure development are carried out in an environment-friendly and sustainable manner.

CBIP took the initiative for creating the awareness about versatility and utility of Geosynthetics including Geomembranes, etc., in various engineering applications as early as in 1985 by organizing a Workshop on Geomembranes and Geofabrics which brought together on a common platform the academicians, manufacturers and representatives of user organizations for discussions and mutual exchange of ideas to facilitate extensive use of these materials in various engineering applications. Central Board of Irrigation and Power (CBIP), in association with Indian Chapter of IGS, is doing its best in promoting the use of Geosynthetics, including Geotextiles.

CBIP is going to complete 35 years of its service to the Geosynthetics Community. On behalf of CBIP and Indian Chapter of IGS, I thank International Geosynthetics Society (IGS) for the continued support in our endevour.

On this occasion, we fondly remember the Past Presidents of the Indian Chapter, namely Dr. R.K. Katti, Mr. H.V. Eswaraiah, Dr. G.V. Rao, Dr. D.G. Kadkade, Dr. K. Rajagopal and Dr. G.V.S. Raju, and all the past members of the Executive Board of the Indian Chapter, for their guidance and support in the journey so far. I thank all the manufacturers, researchers and practitioners, who have supported the activities of the Chapter.

V.K. Kanjlia
Member Secretary
Indian Chapter of IGS

USE OF INDUSTRIAL BULK WASTE MATERIALS IN INDIAN INFRASTRUCTURE DEVELOPMENT - A CRITICAL APPRAISAL

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INTRODUCTION

The quantities of bulk waste being produced major industries in the country are humongous. As brought out in the foregoing, not all them all toxic/hazardous. The non-toxic material can be safely used in our projects in many ways. For some of them the techniques of use are clearly defined by IRC, some are also being used, nevertheless the mountains of these wastes continue to grow occupying considerable area too. At the same time, we are unable to procure the right kind earthen material for our civil engineering projects; sand and aggregates have become premium and even earth is not available in the restrictions being imposed by the MoEF. Along with that the notifications by the concerned ministries state that we will not be permitted to use virgin earth, sand etc., but only recycled materials or bulk waste materials should be used, beyond 2030. In addition as good construction sites is rarely available, construction on soft and weak soils also becoming increasingly challenging?

The paper deals with the bulk industrial waste materials which are commonly designated as marginal materials in the geotechnical parlance.

TYPES OF MARGINAL MATERIALS / WASTE MATERIALS

There are many types of wastes that available whose chemical and physical properties widely vary along with their location in different parts of the country. Notably they can be listed as below:

- Building Waste - Construction and Demolition Waste (CDM)
- Fly Ash (Pond Ash)
- Plastic Waste
- Waste Marble Powder (WMP)
- Jarosite – Jarofix
- Copper slag, Blast Furnace slag
- Red Mud

1. BUILDING WASTE - CONSTRUCTION AND DEMOLITION WASTE (CDW)

A building site is a place of origin of different reuses and their significant part is building waste. According to the communiqué of the Commission of the European Parliament of July 1, 2014, building waste is a refuse of building processes, repairs or disassembly of building objects. This definition is identical to the understanding of a building waste as stated in the decree of the Minister of Environment according to which building wastes are. ”wastes originating from building repairs, disassembly of building objects and road infrastructure (including ground and earth from polluted areas”). They includes subgroups of various wastes – from concrete, glass, plastics, wood, asphalts and tar products, metal scrap, metal alloy, soil and ground, insulation and structural materials, also these containing asbestos, gypsum, mercury, PCB, up to the wastes containing dangerous substances. In the case of the earth being excavated, it is considered as building waste when this earth or rock ruble is removed (or moved) the ways of their usage. In the years of 2004-2006 the total recycling of building and disassembly wastes in Poland amounted to 28.3%, whereas in other countries stood at in: the Netherlands 98.1%, Denmark 94.9%, Germany 86.3%.

Large quantities of mineral waste from excavation, construction or demolition works are generated in Germany annually. In order to protect natural resources, it is required by German law to recycle and reuse as much of these materials as possible. While recycling of non-hazardous construction and demolition waste has already progressed to a certain point, a considerable amount of (natural) excavation materials is still being landfilled and deposited. Fine-grained soils with in situ water contents above the optimum compaction water content generally show poor engineering properties, therefore their properties need to be improved by soil treatment or they have to be discarded. At the Technische Universität München the use of demolition waste for the improvement of soft, fine-grained soils is currently being investigated, combining both types of waste materials.

In India, the IRC has brought out Guidelines for the use of Construction and Demolition waste in Road Sector (IRC 121 – 2017).

2. FLY ASH / POND ASH

Fly ash is defined as “Finely divided residue resulting from the combustion of ground or powdered coal which is transported from the fire box through the boiler by flue gases". Fly ash is captured by mechanical separators, electrostatic precipitators or bag fillers. Later, it is transported through slurry pipelines to ash ponds

Present generation of fly ash from coal based thermal power plants in India is 131 t/year and it is expected to increase to 300-400 t/year by 2016-17.

Government of India Initiatives on Fly Ash

According to the Gazette of India dated January 27, 2016, Ministry of Environment, Forests and Climate Change Notification, New Delhi, the 25th January, 2016 the coal or lignite based thermal power plants shall comply with the above provision in addition to 100% utilization of fly ash generated by them before 31st December, 2017. The coal or lignite based thermal power plants shall within a radius of three hundred kilometers bear the entire cost of transportation of ash to the site of road construction projects under Pradhan Mantri Gramin Sadak Yojna and asset creation programmes of the Government involving construction of buildings, road, dams and embankments. Ministry of Environment, Forests and Climate Change (MoEFCC) has revised norms for fly ash usage and disposal by granting permission to use it for agriculture. The ministry has also made it mandatory for power plants to give fly ash free of cost to users within 300-kilometre-radius.

According to MoEFCC, the fly ash utilization in the country was 57.6 % in 2014 as against 13.5 % in 1999. About 20,000 hectares of land resources can be saved annually by effectively utilization of fly ash in India.

This may be summarized as;

1. Utilization of fly ash is 56.04% in the first half of 2015-2016, which is very much behind the required target.
2. Areas having large prospective of fly ash utilization needs to be discovered for increasing the overall utilization of fly ash in India.
3. Technological advancement is required for collection, storage and disposal facilities of fly ash so that fly ash in dry form could be made available to its users.
4. The states and districts where TPPs are located needs to promote fly ash utilization; construction of building/highways/roads/flyovers and other infrastructure projects.
5. The use of fly ash in the projects within a radius of 300 km of any TPP as mandated in MoEF and CC's notification of 25th Jan 2016 has to be ensured right from project formulation stage.
6. Utilization of fly ash in agriculture is below expectation because of presence of heavy metal and radioactive elements in fly ash. These apprehensions are mandatory to be addressed for increasing fly ash utilization.
7. There is need to encourage industry-institute interaction for entrepreneur development, creating awareness and organizing training workshops.

Uses in Road Sector

The Indian Road Congress have issued Guidelines long ago on utilizing in sub-base base stabilized soil courses along with lime. Also in lime based concrete layers fly ash was recommended.

More recently, IRC SP 58, Use of fly ash in Road Embankments was developed (1999). Based on this, a number of embankments have been built in the country using fly ash. Also, it has been successfully as fill material in Geosynthetic Reinforced Soil Walls, the earliest being the one at Okhla fly over, later the Grade Separator at Saritha Vihar (both in Delhi). Another notable one was on the soft soil at Gangavaram port (Visakhapatnam) wherein the Geosynthetic Reinforced Soil Wall of the ROB was built entirely with pond ash. Figures 1, 2, and 3 depict three Geosynthetic Reinforced Soil Structures in India, built with pond ash as the reinforced fill material.

Fig. 1 : Road over bridge at Gangavaram Port, Visakhapatnam, A.P.

Fig. 2 : Full height panel Geosynthetic Reinforced Soil Wall, at Korba, Chattisgarh

Fig. 3 : RoB at Khaperkheda Thermal Power Station, Maha Genco (Photo courtesy : Satish Naik)

3. WASTE PLASTIC

The amount of plastic waste being generated in India is so large that it is one of the most hazardous environmental concern.

On the use of such plastic waste, the IRC SP 98-2013 deals with the guide lines for use of Waste Plastic in Hot Bituminous mixes (Dry Process) in Wearing courses.

As mixing such flaky waste material is difficult in soils, this has not been taken up earnestly in road embankments, despite the fact that, when uniformly mixed plastic waste enhances the engineering properties of the soil.

4. WASTE MARBLE POWDER

One of the most important parts of the mining industry is the marble industry. The use of marbles in the structures dates back to ancient times. However, especially in the last decades, the use of marble in the structures has increased too much. As the production in the marble factories increases, the amount of waste has reached a volume that cannot be stored. Marble wastes are formed as a result of cutting and polishing processes in marble factories. These by-products, which cannot be stored, should be utilized in other sectors. So, both economic losses can be reduced and environmental pollution can be prevented.

Wherever marble stone processing is done, in areas like Ajmer, the processes yield marble slurry and marble powder in larger quantities, This material is commonly designated as waste marble powder (WMP).

The fresh, strength, and durability properties of conventional and self-compacting concrete (SCC) produced by replacing with WMP were investigated in comparison with the literature. It has been observed that the use of WMP by replacing with cement or sand in both conventional and SCC positively affects fresh and hardened properties of these concretes. The use of WMP instead of raw materials (e.g., sand, coarse aggregate) obtained from the natural resources reduces the energy consumption and the use of these natural resources. As a result, it is possible to obtain economical and eco-efficient concretes using WMP.

The bulk WMP can also be used in embankments (similar to fly ash) and also in pavement layers. It is understood that the MoRTH has recommended its use in various projects of Rajasthan, but unfortunately no Case histories are available, as of now.

It is hence necessary to undertake both laboratory and field studies (with case histories) in order that proper guidelines may be formulated.

5. JAROSITE – JAROFIX

Jarosite is a basic hydrous sulphate containing potassium and iron. Jarosite is often produced as a by-product during the purification and refining of zinc. The Jarosite mainly contains iron, sulphur, zinc, calcium, lead, cadmium and aluminum. This paper will study the potential application of Jarosite in various engineering works such as road construction, airfields, dams, bricks and tiles. Due to the increasing of annual production of Jarosite, it is a major source of pollution for surrounding environment including soil, vegetation and aquatic life.

Zinc ore concentrate contains about 50% zinc. This concentration is roasted at 900 deg C and subjected to leaching where Jarosite is formed as a waste material. Jarosite waste is a group of wide variety of particles ranging from clay to fine silty sand (Typical values silt 63%, Clay 32%). At present, the annual production of Jarosite is about 0.4 million metric tons while its accumulation is very less because it is converted into new stable material called Jarofix by addition of 2% lime and 10% cement.

This material has the great potential to be utilized for the construction of road embankment (Havanagi et al., 2012). While the other significant mix like Jarofix-soil mix (50-75%) and Jarofix-bottom ash mix (50-75%) can also be utilized for the construction of embankment and may be used for construction of sub-grade layer of road pavement (Sinha et al. 2012b & 2013b). The possibility of replacing the natural gypsum, used in cement production, by a Jarosite/alunite chemical precipitate was also, in investigated in literature.

6. COPPER SLAG,GRANULATED BLAST FURNACE SLAG

Copper slag is a by-product obtained from the matte smelting and refining of copper and about 2.2-3.0 tons of copper slag is generated as waste for every ton of copper produced, (Maria Mavroulidou, 2017) [14]. The world copper production is currently about 20.47 million tons (International Copper Study Group, 2018) and hence more than 50 million tons of copper slag is being generated. The copper slag generation in India is estimated to be about 1.63 million tons. Based on the U.S. environmental protection agency regulations, copper slag may be classified as non-hazardous material. Sterlite Industries, Hindalco and Hindustan copper are the major contributors in the production of copper in India. Laboratory studies reveal that it is angular sand size particles with high angle of shearing resistance. Further, it is reported by the studies conducted by Professor Raman that slag is non-hazardous in terms of ground water pollution. Copper slag used in the present work is collected from Sterlite Copper, Thoothukudi, Tamil Nadu. These are materials available in bulk form from the furnaces of iron and steel and copper manufacture. Usually they have larger particle sizes and in general chemically inactive. Once they are powdered they can be an excellent source of fill materials. CRRI has done considerable work on the suitability of these materials. (Havanagi, Mathur, Prasad and Kamaraj, (2007)).

7. RED MUD

This is a byproduct of Aluminum industry and available in bulk wherever Aluminum industry exists. But the material is classified as hazardous. Hence it is yet to find use in civil engineering applications.

RECOMMENDATIONS

The above information gives an idea of the immense potential of these industrial bulk wastes in the road sector. Over the years the Central Road Research Institute, New Delhi and Indian Institute of Technology, Delhi, has conducted significant studies on the subject.

At the University of Wisconsin, USA, the Recycled Materials Resource Center conducts research and outreach on environmental and material properties of recycled materials and catalyzes their wise and appropriate use in the marketplace. Sponsored by the U.S. Department of Transportation and the Federal Highway Administration, the Sustainable Pavements Program has advanced the knowledge and practice of sustainability related to pavements since 2010. The goal of the program is to increase awareness, visibility, and the body of knowledge of sustainability considerations in all stages of the pavement life cycle.

But for the confident use of these materials at the outset, documents should be got prepared which give the following information.

- Available Source (location, quantity etc.)
- Physical and chemical characteristics (including leachate)
- Engineering Characteristics
- Review of actual use i.e. Case Histories
- Potential Uses
- Design Methodology
- Testing and Quality Control
- Construction Methodology
- Special Precautions, if any.

This task is being undertaken by the Indian Road Congress, Committee H4.

REFERENCES

Havanagi V.G., Mathur, S., Prasad P.S. and Kamaraj, C.(2007). Feasibility of copper slag-fly ash-soil mix as a road construction material. Journal of Transportation Research Board, Vol 2. No.1989, 13-20.

IRC SP 58(1999) Use of fly ash in Road Embankments.

IRC 121-2017Guidelines for the use of Construction and Demolition waste in Road Sector.

IRC SP 98-2013 Guide lines for use of Waste Plastic in Hot Bituminous mixes (Dry Process) in Wearing courses.

Sinha A.K, Havanagi V.G. and Shahu J.T (2019). Construction and performance of jarofix waste material embankment. Proceedings of the Institution of Civil Engineers- Construction materials https://doi.org/10.1680/jcoma.18.00003

Sinha A.K, Havanagi V.G, Arora, V.K., Ranjan A., and Mathur S. (2012) Recycling Jarofix waste as a construction material for embankment and sub grade. International Journal of Solid Waste Management, vol. 38(3), pp 169-181.

Sinha A.K., Havanagi V.G. and Shahu J.T (2018)*. Characterization of jarofix for usage in geotechnical projects. Proceedings of the Institution of Civil Engineers – Geotechnical Engineering. 171(5). pp 439-450.

Sinha A.K, Havanagi V.G. and Shahu J.T. (2019). Stabilized jarofix waste material for road construction, International Journal of Pavement Engineering, 10.1080/10298436.2019.1652299

GUIDELINES FOR STABILIZATION OF SILTY RAVINES

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INTRODUCTION

The current guidelines abide to stabilization of ravines essentially consisting of loose silty deposits. During rains, the rain droplets fall with high velocity impact on the soil particles directly, thereby loosening them. Rainwater flows from the banks towards the river in the form of small rivulets. Silty soil being poor in cohesion is easily carried by the rivulets. This movement of rivulets exerts driving forces causing dislodging of chunks of soil mass. As a result, the embankment gets eroded which results in widening of the cross-section of river and shifting of river banks towards structures constructed on banks. This issue basically consists of two parts, i.e., erosion control of top soil cover and global stability of slopes which have been discussed.

For erosion control of silty soils, different solutions like live plantations, bioengineering, hard armoring have been suggested in literature (Gupta et.al.2016). Grouted riprap and RECB blankets are generally adopted solutions for controlling erosion in practice and hence have been discussed in this document.

GROUTED RIPRAP

Grouted riprap is adopted to control the erosion of flatter slopes. This is adopted as a feasible solution where there is easy availability of stones. The components of grouted riprap are as shown in Figure 1. According to the US Army Corps of Engineers, firstly a non-woven geotextile material is to be laid as a filter, followed by a granular subbase and then by stone-grout layer. The non-woven geotextile is provided to avoid the washing out of retained subsoil material and also to allow water to flow out of the system owing to its property of in-plane transmissivity. The granular sub base is provided to avoid the overlaid grout from clogging the under laid geotextile. Also, drains are suggested to be provided at about 8 m center to center at regular intervals in longitudinal direction to direct the rain water flow and hence prevent the surface erosion.

ROLLED EROSION CONTROL BLANKETS (RECB)

RECB is usually adopted at sites wherein the retained subsoil is susceptible to differential settlements due to erosion. RECB blankets are laid over ready steep slopes and are anchored at the top and bottom in the head drain and the toe drain respectively as shown in Figure 2. Also a toe wall is provided to increase the overall stability. A non-woven geotextile is provided behind toe wall in contact with sub soil to prevent washing out of fines. The RECB prevents surficial erosion effectively behaving as a separator and the rain droplets falling with high velocity do not come in direct contact of soil mass. The RECB prevents piping and provides permeability in view of its apparent opening size and permittivity values. The coir would get biodegraded over a period of time and gets converted into useful manure to promote vegetation growth thus proving RECB to be eco-friendly solution. Due to the subsequent growth of vegetation, the roots penetrating into the soil acts like micro-reinforcement for the top cover of soil (Hall 1992) and thereby increasing the shear strength even under fully saturated condition.

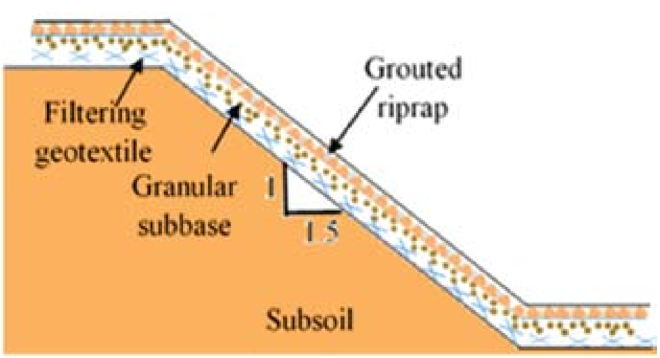

Fig. 1: Schematic diagram of a slope with grouted riprap protection

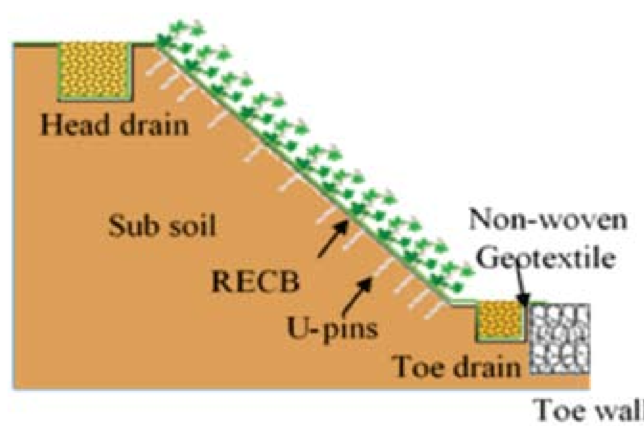

Fig. 2: Schematic diagram of a slope with RECB protection

The detailed procedure of laying RECB blankets is as follows:

Step 1: A soil mixture of river soil (silty sand), a good water retaining soil (having considerable cohesion) and cow dung in equal proportion can be sprinkled along the slope along with a suitable soil conditioner as shown in Figure 3.

Fig. 3: Image depicting site preparation

Then, the soil mixture should be sieved and sprinkled uniformly on site as shown in Figure 4. Big chunks of soil if any should be broken down manually.

Fig. 4: A typical view showing the sieving and the spreading of soil

Step 2: Laying of RECB

Coir geotextile arrive on site in the form of rolls of 30 m - 50 m length and 1.0 m - 1.9 m width. These should be laid along the slopes along full slope length with top edge and bottom edge being inserted into top and bottom trench drains and covered with soil as shown in Figures 5 and 6.

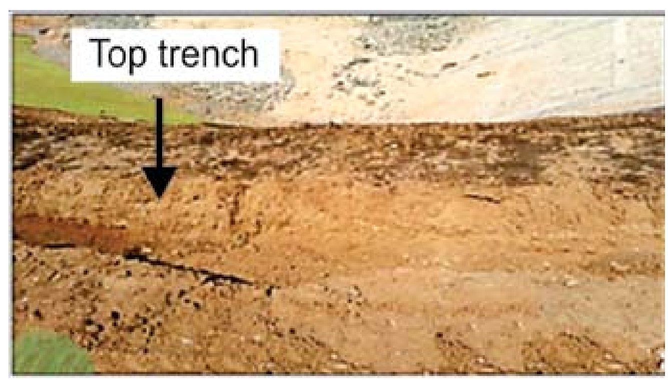

Fig. 5: View showing the top trench for inserting geotextile's top edge

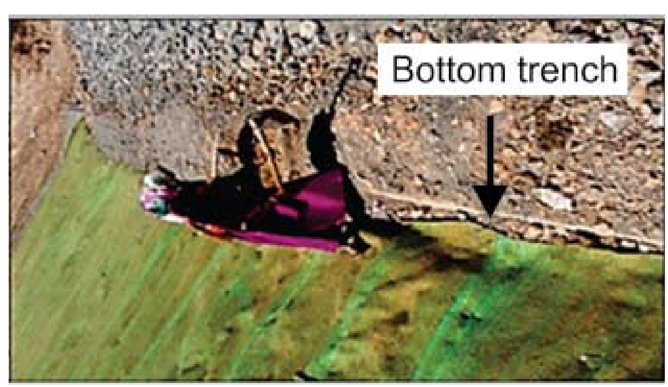

Fig. 6: View showing the bottom trench for inserting geotextile's bottom edge near toe wall

The adjacent blankets should be placed by providing aoverlap of approximately 10 cm and should be pegged down by inserting U-pins with high carbon steel of 3.8 mm diameter provided with GI coating to control erosion at joints as shown in Figure 7. GI pins should be inserted at every 1 m along the slope height and this spacing should be considerably reduced for the RECB provided at the ends.

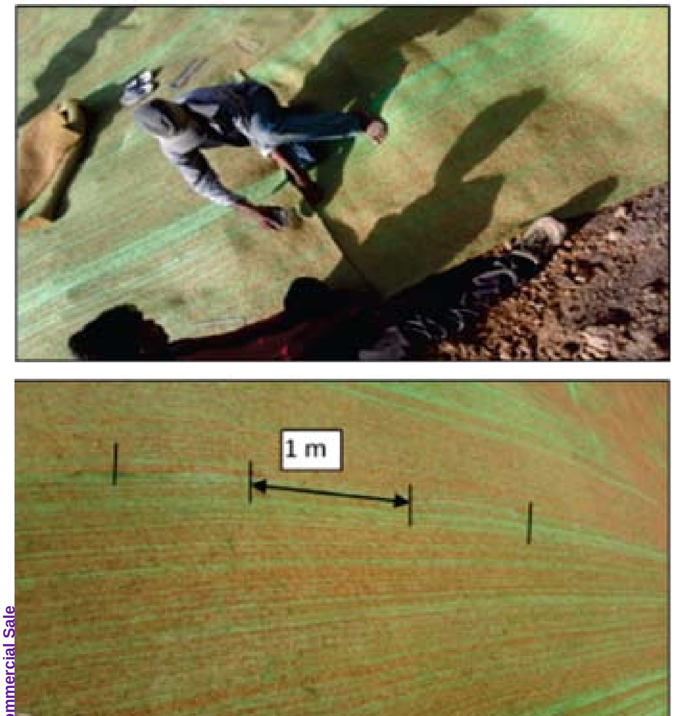

Fig. 7: Insertion of U-pins at 1 m intervals for joining adjacent RECB layers

Step 3: Construction of Toe Wall

RECB will help in resisting surficial soil erosion but will not ensure overall stability. In order to provide a berm, a toe wall should be provided supporting the bottom trench. This toe wall usually can be of different types. Gabions being pervious and flexible structure would discourage the development of pore water pressure and being flexible would itself undergo deformations without causing distress to the retained soil. These can be fabricated from the locally available stones and prove to be an eco-friendly and cost effective solution. Gabion gravity wall or RE walls with gabions as fascia (detailed discussion provided in later sections) can be provided as the toe wall as shown in Figure 8.

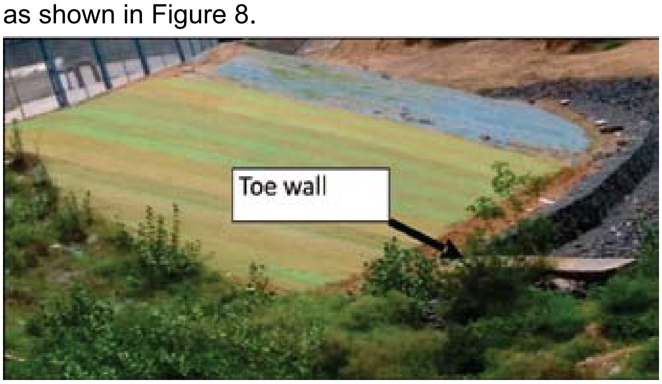

Fig. 8: Image depicting gabion toe wall placed at the base of slope

Step 4: Plantations Along Slope

The coir geotextile will allow rain water to seep partially into the slope. Coir being biodegradable material, over a period of time after full vegetation growth, will biodegrade and will act as a good manure for the vegetation. Normally, barren slopes are devoid of Nitrogen which is essential for the growth of vegetation. It takes years for the soil to absorb Nitrogen from the air. By using coir, the process of vegetation growth gets accelerated. Also, it will enhance the aesthetic beauty of the site. Before initiating the plantation, the entire slope covered with RECB should be sprinkled with water twice a day to create moist condition for plantation. The plantation can be carried out by cutting a small hole through RECB and planting a sapling at regular intervals as shown in Figure 9.

Fig. 9: Hammering of chisel for planting sapling

Issues Associated with Grouted Rip Rap and RECB through Case Study

The IIT Gandhinagar campus is located along the bank of Sabarmati river. The ravines along the bank of Sabarmati river have been getting eroded during rainy season which posed risk for the structures constructed on the banks of river as shown in Figures 10(a) and 10(b). The ravines essentially consist of loose silty deposits.

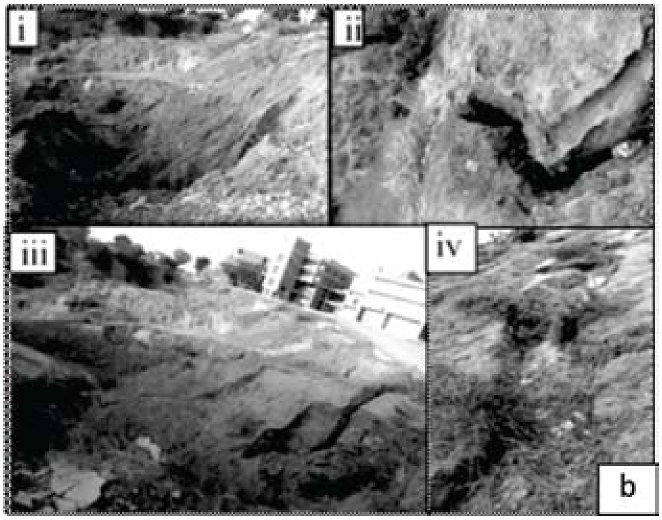

Fig. 10(a): A typical aerial view of the ravines at IIT Gandhinagar. (b) Views of uneven steep slopes at different locations in the ravines

A pilot study has been carried at the campus of IIT Gandhinagar wherein grouted riprap and RECB were laid along ravines for a stretch of 20 m adjacent to each other. The grouted riprap was provided along section 1 and RECB along Section 2 as shown in Figure 11. The sewage treatment plant was resting just on the edge of the ravine at Section 2 as shown in the Figure 11 and hence was at risk. The Section 1 has been levelled to achieve slope of 1H: 2V and Section 2 has been levelled to 1H: 1.5V. The specifications for the grouted rip rap and RECB used on site has been given in the Table 1. RECB has been provided on the steeper slopes (1H: 2V) in view of its capacity to enhance the factor of safety. The drainages provided for grouted riprap may not be sufficient to drain out the rainwater away from the slopes. Also, grouted riprap being rigid in nature, these are observed to be inefficient when the retained fill undergoes settlement.

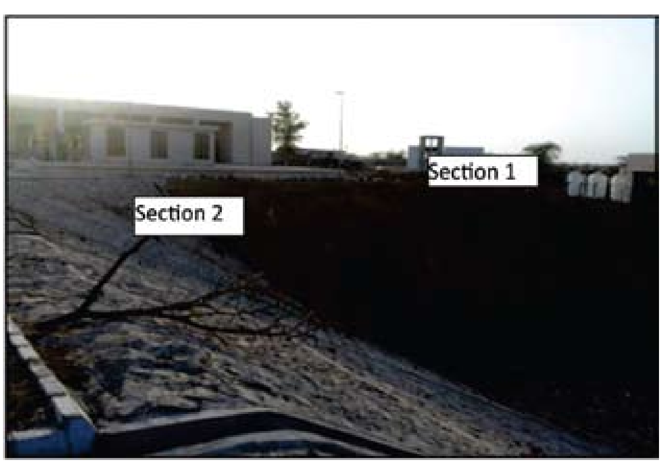

Fig. 11: Views at the pilot study site

The RECB used in this study consists of non-woven coir fibers sandwiched between woven polymer nettings on both the sides. The polymer nettings has aperture size lesser than 2 mm. The tensile strength of this blanket for embankment height greater than 6 m and slope steeper than 1:1 has been 7 kN/m in machine direction and 1.5 kN/m in cross direction. The nominal weight of the blanket provided has been 600 g/m^2.

Table 1: Specifications of the materials

Items	**Specifications**	
Grouted Riprap	Supply and filling soling stone for prevention of erosion at various ravines as per directions of engineer in charge with lead up to 50 m and lift up to 1.5 m	
	Cement mortar 1:6 of thickness 0.23 m	
	Non-woven geotextile filter material	
RECB	Tensile strength in machine direction	7 kN/m
	Tensile strength in cross machine direction	1.5 kN/m
	Nominal weight of blanket	600 g/sq.m

The site has undergone two rainy seasons. It has been observed that the grouted rip rap has developed cracks and hence has undergone failure as shown in the Figure 12. But the RECB laid at site has effectively sustained two rainy seasons and the vegetation growth has increased the aesthetic look of the site.

Fig. 12: Views showing pilot study Section 1 and Section 2 after 14 months

The RECB been pervious, flexible, cost effective, and eco-friendly solution has been provided at different locations in campus which required immediate attention as shown in Figures 13 and 14. For the critical sites, drains have been provided at regular intervals.

Fig. 13: Different locations in ravines with RECB solution

Fig. 14: Improved site for the same distressed site as shown in Figure 10b(iii)

Fig. 15: Distressed grouted riprap site due to improper drainage

At a few locations, where grouted riprap was provided, it has undergone tremendous distress as shown in Figure 15. This is because, grouted rip rap being rigid and impervious in nature and as also non-woven filter material was not provided between grouted rip rap and subsoil, during rains there was built up of pore water pressure. As the provided drains were catering to only the surface flow, has been observed that, the percolating water made its path through the developed weaker cracks leading to the movement of silt below the grout layer resulting in settlements due to wash out of fines.

At places, RECB too has undergone distress as shown in Figure 16. This was due to the inadequate provision of pins. As the U-pins have not been provided at closer intervals near the edges, water made its path through these joints and washed away the underlying soil as shown in Figure 16. This in times of cloud burst has also caused distress to the access road constructed on the backfill as shown in Figure 16(d). The road has undergone failure due to subsidence of the underlying soil mass. At connections, if required precautions are not taken, it may lead to huge cracked failure as seen in Figure 16(g).

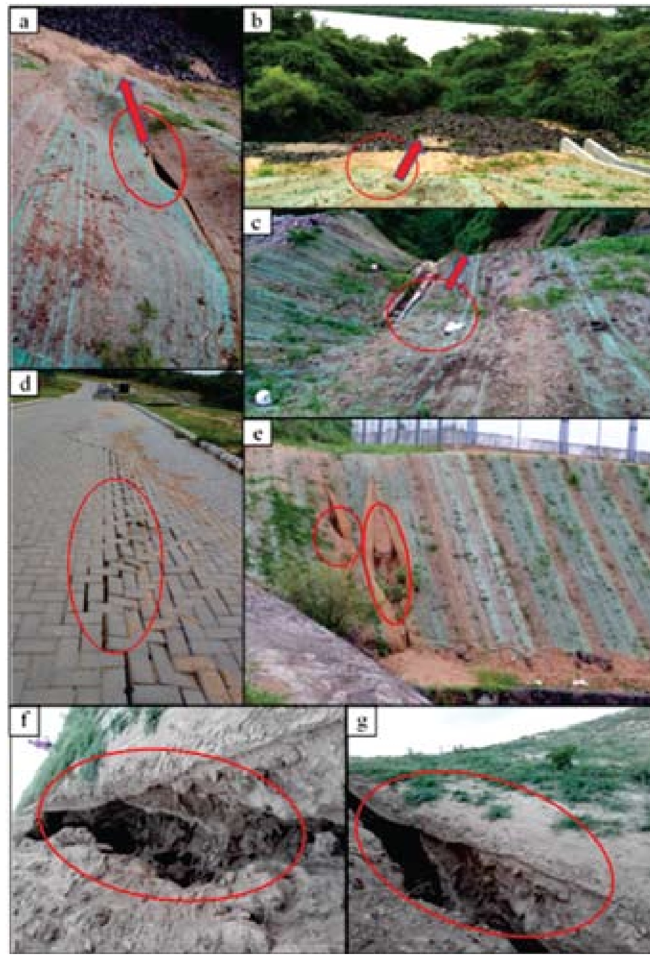

Fig. 16: Views depicting failure of RECB due to improper joint connections

It is recommended that the U-pins should be provided at regular intervals in longitudinal direction in staggered manner as shown in Figure 17. This is to ensure RECB to be in complete contact with soil in spite of the failure of pins at the overlapping joints.

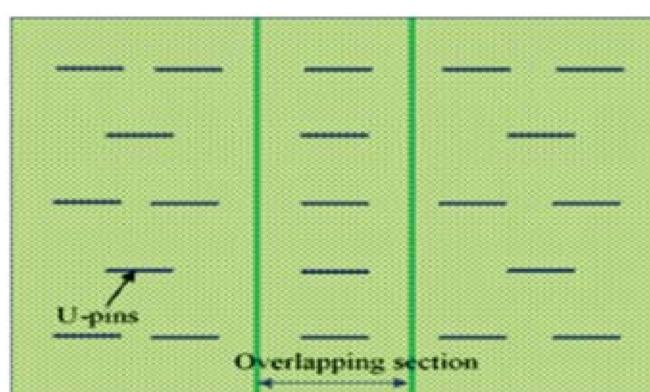

Fig. 17: U pins insertion pattern

As per the rates obtained from the engineers of Public Works Department, the cost of the RECB system and the grouted riprap system is Rs 345/m^2 and Rs 685/m^2 respectively for the specifications mentioned in Table 1. Hence, the flexible RECB system is found to be much more economical than the rigid slope protection measure as grouted riprap.

Concluding Remarks for Erosion Control

- Grouted riprap being rigid and impervious is not capable of performing the slope protection effectively.
- RECB along with gabion gravity toe wall being porous discourages the development of pore water pressure and are flexible enough to protect the siltys oil slope.
- RECB system as adopted prevents soil erosion; the provided gabion gravity wall imparts toe protection.
- U-pins used at joints should be provided at closer spacing to enable the RECB to cover and hug the soil surface in total without any possible opening during service life.

Providing coir mattresses can help in harnessing erosion of the top cover only. But, rain water pouring on the head of ravines or slopes would make its own path thus driving the mass of soil along with it. In order to provide an overall stability to the slopes, solutions proposed are discussed in the following section.

A typical solution for retaining cuts along river banks is the use of gabions in the construction of retaining structures. These are also widely used in civil engineering, road building, military applications and landscaping because of their varied advantages. These are easy to construct and are available in variety of sizes. These are flexible and permeable monolithic structures. Gabions in the form of mattresses, undergo deformations when subjected to erodible conditions but retain the integrity of structure. However, at the sites where predominantly cohesionless and permeable backfill material is available constructing a reinforced soil wall may also be a suitable solution. For such walls, the reinforcement is usually extruded or flexible geogrid and facia is discrete panel facia. But, concrete panels are not very tolerant to differential settlement. Gabions as fascia can be used which can tolerate larger deformations without much distress.

Gabion Retaining Walls

Retaining walls using gabion blocks are provided along hill side to protect the slope along roads and railway track and also along river banks. Depending upon the size of stones available in the vicinity, the size of the mesh required is decided. Gabions arrive on site in the form of folded sheets. These sheets are opened and the edges are laced together using wires of higher diameter. Thereafter, these boxes are filled with stones of suitable dimensions depending upon the surrounding mesh size. The availability of smaller size stones will necessitate smaller size of mesh to hold them together resulting in higher cost of the structure. But, smaller size of mesh will provide more tensile strength which may not be utilized to the fullest. This has evoked a need for reinforced soil walls with gabion facia as these have high tolerance with respect to deformations as compared to rigid discrete facia panels.

Reinforced Soil Walls

The locally available silty soil can be suitably reinforced to develop a retaining structure known as reinforced soil walls. The clauses mentioned in different codes have recommended the back fill soil to be permeable and cohesionless. They do not allow fines content in soil to exceed 21%. The reinforcement in such walls are usually extruded or flexible geogrid. The soil is compacted to a minimum of 95% of MDD and OMC maintaining minimum height of lift. The soil and the reinforcement behaves as a single unit and provides resistance against the lateral earth pressure due to the friction between soil and reinforcement from the retained fill and different magnitudes of upcoming surcharge. The facing is usually provided on the front side for the aesthetic purpose and to give support under unavoidable circumstances, if the soil and reinforcement unit undergoes any distress. Rigid facia units have led to the distress of structures at some places. This is because rigid facia does not settle with the back fill thus resulting into failure at the connections. Flexible facia units have proved to be effective in such situations. Gabion facia are flexible and most suitable for hilly areas and river banks which have enough availability of stones. The mesh size is decided on the basis of the size of available stones. Usually extensible geogrids are provided as reinforcements. Mesh wires with welded joints being inextensible can also be provided as reinforcements as they will help in reducing the costs. So, in the areas where there is minimal surcharge, steel mesh can be provided as reinforcement to utilize its complete strength. Gabion mesh is comparatively extensible because of its double-twisted joints as compared to welded mesh joints. Thus, gabion mesh can utilize its tensile strength by allowing the structure to take strain up to 10 %. Gabion mesh galvanized with zinc and coated with PVC can be used suitably on the basis of their tensile strengths. However, at the sites of heavy surcharge, still the strength can be achieved by placing one reinforcement sheet over another.

Comparative Study of Gabion Gravity Walls and Reinforced Soil Walls with Gabions as Fascia

Jadhav and Venkatappa Rao (2016) performed comparative analysis to suggest the suitable and cost-effective gabion structure for silty ravines. Accordingly, a Matlab code was developed to design two types of soil retaining structures namely, gabion gravity walls and soil reinforced with gabion mesh having gabion facia.

Gabion gravity walls have been designed assuming zero wall to soil interface friction angle. Thus, the lateral earth pressure has been computed using Rankine's theory for smooth walls. The base has been assumed to be 0.75 times the height of wall and an offset of 0.5 m has been provided at every 1 m. The unit weight of stones has been considered as 15.5 kN/m^3. All the walls have been designed for vertical back face and horizontal angle of slope for dry back fill. It has been assumed that the wall and retained fill are under drained conditions. The retained fill has been assumed to have angle of shearing resistance as 30°. A gabion gravity wall with the considered assumptions regarding dimensions is represented through Figure 18(a). Weight of the steel wire mesh has not been included to contribute in the vertical load of wall. The stability of wall has been checked with respect to sliding, overturning and bearing. The critical factor of safety values against sliding and overturning have been considered as 1.5 and 2 respectively. The structure has been rendered safe against bearing if the eccentricity of the resultant force acts within one-sixth of base width. Also it has been ensured that the upcoming bearing pressure is less than the bearing capacity of soil. The Matlab code has been developed to compute the volume of stones and surface area of steel wire mesh for the achieved safe design for a particular height of wall.

Reinforced soil walls have been designed following the conventional design guidelines excluding the contribution from the weight of facia in the stability of structure. Back fill and retained soil has been assumed same throughout the height of structure. Thus the interface angle° has been assumed to be equal to the angle of shearing resistance of the back fill soil. Height of lift is maintained 0.5 m throughout the height of structure for the depth of embedment of 1m. The external stability of structure has been ensured assuming a minimum width of 0.6 H or 2.5 m where 'H' is the height of structure including the depth of embedment when the effect of facia has been considered in the analysis. The codes specify minimum width to be provided as 0.7 H. But, as in the modified design advantage due to facia has been considered which has implicitly allowed to provide width as 0.6 H for external stability analysis. The gabion mesh reinforced soil retaining wall is as shown in Fig. 18(b). Ensuring external stability of structure has been given primary importance. The structure has been considered safe if the values of the factor of safety values obtained are greater than 1.5 and 2 against sliding and overturning respectively. The resultant of the forces should lie within one-sixth of the width of base to ensure safety against bearing failure. The internal stability has been ensured by providing reinforcement of suitable tensile strength against rupture and optimum embedment length. The design tensile strength for gabion mesh has been calculated by considering reduction in strength due to creep, installation damage and durability. As the steel mesh is comparatively inextensible, the creep reduction factor has been considered as 0.9. Steel being tough material, the reduction factor against installation damage and durability has been considered as 0.95 and 0.95 respectively. For greater heights, if the lower reinforcements fail against rupture, then reinforcements with available higher strength have been provided followed by rechecking the design. The embedment lengths have been computed by assuming the interface friction angle between reinforcement and soil as 0.9φ. This is because the gabion mesh has comparatively rough surface than geogrid. The minimum effective length of 1m beyond Rankine's zone has been provided. The surface of reinforcement required and volume of soil for reinforced soil unit and volume of stones for facia has been computed for different heights of wall. Following key analyses have been performed and the observations and results are discussed in detail in next section.

1. Comparison of conventional gravity gabion wall and soil reinforced wall with gabion facia with respect to the surface area of mesh required and total cost of project
2. Computing the cost of reinforced soil walls with gabion mesh including the effect of width of facia
3. Comparison of reinforced soil walls and gabion wall for different values of friction angles of reinforced backfill

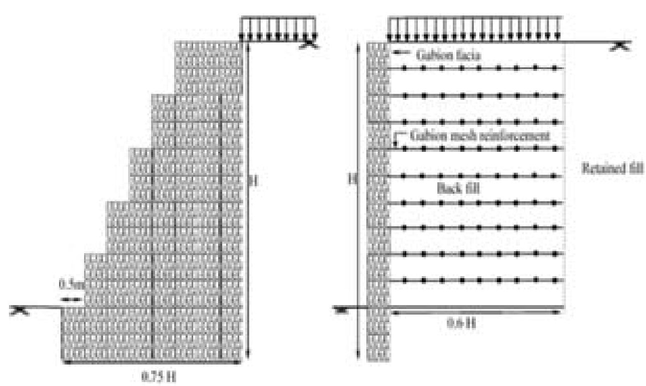

Fig. 18 a: Gabion gravity wall **b.** Soil reinforced gabion mesh with gabion fascia

(1) Using the code developed, the total surface area of gabion mesh required for the gabion wall and soil reinforced wall of desired height has been computed.

The computation is done in such a way that there is only one layer of mesh between two adjacent faces of gabion units. The total surface area of steel mesh required has been computed for walls of different heights and shown in Fig. 19. Also, the total surface area of gabion mesh required for soil reinforced wall as reinforcement and for facia units is also shown in Fig. 19. From this variation, it can be observed that the quantity of mesh required for wall is nearly 55 % more than that of mesh required for reinforced soil wall.

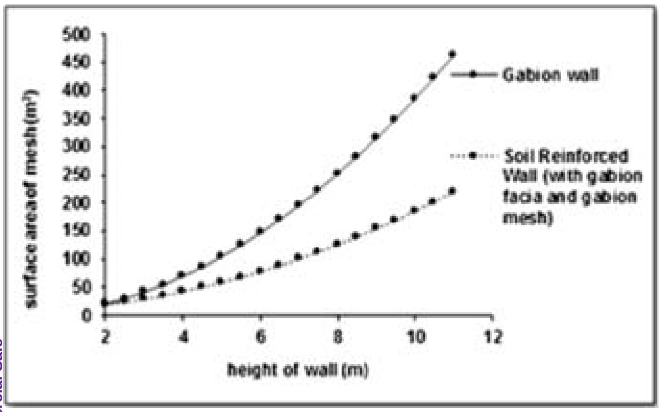

Fig. 19: Variation of surface area of gabion mesh with height of structures

(2) The idea was extended to perform cost estimation of these structures and has been analysed for different heights of wall as shown in Figure 20. The cost estimated for gabion gravity wall included the cost of gabion steel mesh with respect to its rate per square m and the cost of filling the stones including the labour, transportation and all other supplementary charges provided in accordance with the local contractor given in detail in Table 2. The cost computed for soil reinforced wall consisted basically the cost of gabion mesh, cost of stones required for facia and the cost of soil used as the backfill material. The backfill material considered herein has been assumed to be available locally. It can be observed from Fig. 20 that, reinforced soil wall with gabion mesh as reinforcement has resulted in almost 50 % savings. This is because, huge volume of stones have been replaced by comparatively cheaper soil available locally. So, at sites where significant volume of stones in desired sizes are not available, reinforced soil walls may prove to be a cheaper solution. The steel mesh used as a confining element along 6 sides in a gabion unit also has been reduced significantly. This saving has been estimated for only 1m length of wall. The project constructed for a stretch of at least 100 m can save a significant amount.

Table 2: Specifications of the materials used

Items	Specifications		Cost (Rs)
Gabion Mesh (Galvanized with zinc and PVC coated)	Size of mesh	Tensile strength (kN/m)	
	10x12	32	180/sq.m
	8X10	42.5	192/sq.m
Stones	Supplying and filling the gabion boxes with trap rubble stones of size 150mm to 250mm inclusive of all taxes, levys, labour, transportation, scaffolding for execution at all locations, leads and lifts		1170 / cu.m
earth work	Locally available soil		381 / cu.m

(3) The current design philosophies neglect the effect of facia as the facia is usually thin sections of discrete panels. But, flexible and comparatively massive gabions when provided as facia can contribute in the stability of the structure and are recommended to be used to avoid over-conservative design. With this intention, the effect of different size of facia on the cost of these structures has been studied. The results obtained by considering a surcharge of 20 kPa are shown in Figure 20. It can be observed that using gabion facia of 0.5 m thickness can save the cost of project by 12% as compared to wall design without considering the gravity effect of facia. However, if the width of facia is increased, then the cost incurred by the project exceeds the cost of wall designed without considering facia. When facia of 0.5 m width is provided for nominal sake, the width of the reinforcement unit provided is 0.8 H. This means that, the lateral earth pressure due to retained backfill and surcharge is resisted by the reinforced soil and reinforcement only. When 0.5 m facia provided is included in design, the width of reinforcement required is 0.7 H which in a way reduces the cost of earth work and reinforcement corresponding to 0.1 H. However, when the width of facia to be included in design is increased to 1 m, the width of reinforced zone increases to 0.85 H, increasing not only the length of reinforcement required but also the surface area of mesh for 1 m^3 unit. The width of the reinforced zone increases because the 1m width of gabion facia cannot provide enough resistance against sliding as compared to dense back fill soil. Based on the limited study, it can be concluded that, the optimum width of facia to be adopted in design is 0.5 m. Thus, the design philosophies can include facia of certain width in design in such a way that it optimizes the cost of project.

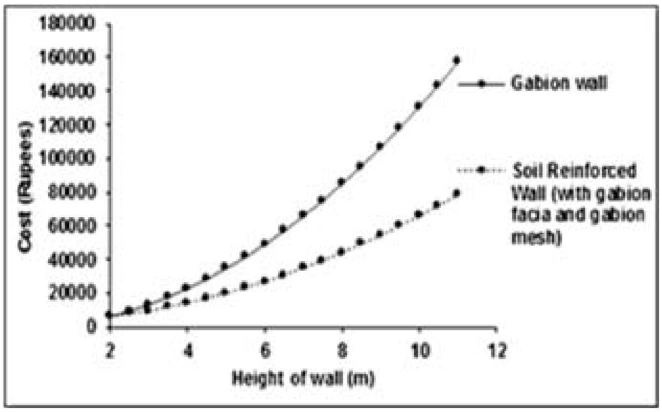

Fig. 19: Variation of cost of project with height of structures

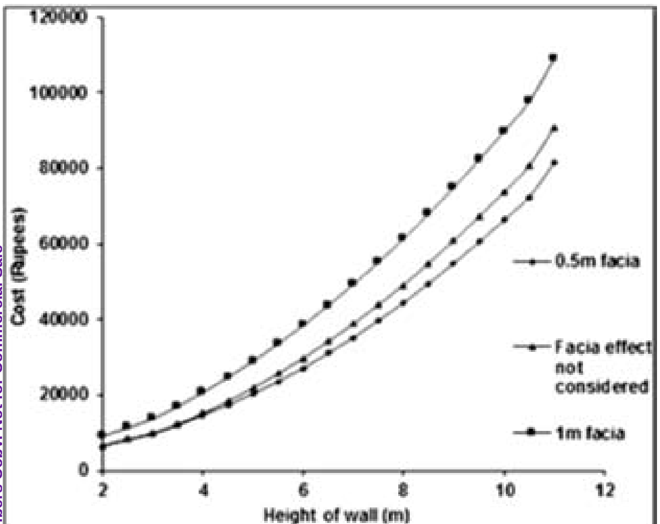

Fig. 20: Variation of cost of project with height of structures for different width of fascia

(4) The backfill soil available locally is not necessarily permeable and can have significant fine content in it. This can lower down the value for angle of shearing resistance from 30° to 25°. Codes generally do not allow to have backfill material to have fines more than 21%. Under the situations, when coarser permeable sand is not available in the near vicinity in required amount, it is always suggested to use the locally available finer material. The effect on cost of gravity wall and soil reinforced wall due to backfill with different φ has been studied in this section. It can be observed from Figure 21 that the cost of gabion gravity wall increases significantly with decrease in the value of φ as shown in Table 3. The decrease in φ increases the lateral earth pressure on wall which generates a need to provide walls with higher width. However, this increase is quite insignificant for soil reinforced wall. This can be attributed to the fact that the lateral earth pressure generated is being taken by the reinforcement of comparatively higher strength which does not cause significant increase in the cost of project.

Table 3: Percent reduction in cost of project at different friction angles

φ (°)	Base width for wall	cost of gabion wall	Length of wall	cost of Reinforced soil Wall (gabion mesh)	Reduction in cost (%)
30	L=0.75H	157163	L=0.6H	73423	53
28	L=0.85H	186764	L=0.6H	75625	60
26	L=0.95H	216365	L=0.65H	81602	62

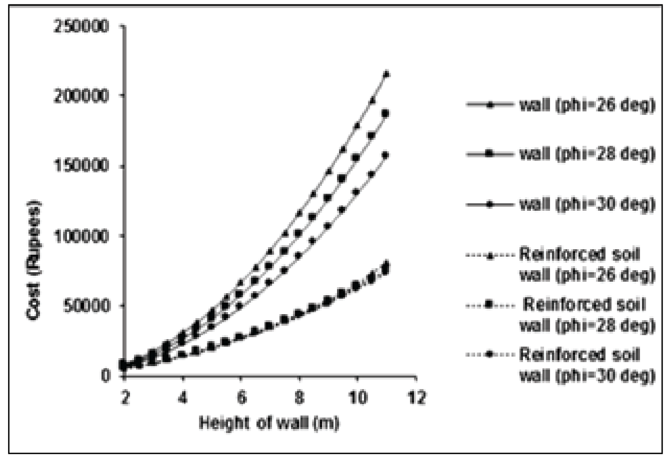

Fig. 21: Variation of cost of project with height of structures for different friction angles

CONCLUSIONS

1. Gabions can prove to be eco-friendly and flexible solution along river bank and hilly areas where adequate stones are available.
2. The quantity of steel mesh required and hence the cost of project can be reduced by 55% by providing reinforced soil wall instead of gabion gravity walls.
3. The design has been modified by including the effect of facia of 0.5 m width in design.
4 The inclusion of facia of 0.5 m width in design would contribute in optimizing the cost of project.
5. The percent reduction in cost increases with decrease in friction angle.
6. There is no significant increase in cost of reinforced soil walls due to decrease in friction angle.

On the whole, for the stabilization of ravines with loose silty deposits, RECB with gabion toe wall would prove to be cost-effective and an eco-friendly solution.

REFERENCES

- BS 8002:1994 Code of practice for Earth retaining structures
- BS 8006:1994 Code of practice for Strengthened Reinforced soils and other fills

- Gupta, M., R. Chitra., and Ratnam, M. 2016. Geosynthetics in river bank erosion control. Advances in Geosynthetics. Edited by Dr. G. Venkatappa Rao and Dr. G.V.S. Suryanarayana Raju. SAGES, Hyderabad, India, pp. 383-394.
- Hall, C. D., 1992. The contemporary approach to the role of vegetation in erosion control. Geosynthetics World, April/May 1992.
- IRC SP-102 2014 Guidelines Design and Construction of Reinforced Soil Walls
- IS 16014 (2012): Mechanically Woven, Double -Twisted, Hexagonal Wire Mesh Gabions, Revet Mattresses and Rock Fall Netting (Galvanized Steel Wire or Galvanized Steel Wore With Pvc Coating) - Specification [Med 10: Wire Ropes And Wire Products]
- Jadhav, P. (2018) Erosion control of silty ravines on the banks of Sabarmati river, 11th International Conference on Geosynthetics, Coex, Seoul, Korea, September 16-21, 2018.
- Jadhav, P. and Venkatappa Rao G. (2016) Reinforced soil retaining structures for ravines, 6 th Asian Regional Conference on Geosynthetics, New Delhi, India, November 8-11, 2016.
- U.S. Army Corps of Engineers, 1992. Design and Construction of Grouted Riprap

ADDRESSING SEEPAGE AND STABILITY CHALLENGES IN CANALS WITH SANDY SOILS : A CASE STUDY OF SARDAR SAROVAR PROJECT

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ABSTRACT

Sandy soils are predominant in many areas in India. Large infrastructural projects in recent time have consumed large quantity of good soils. When the canals are to be constructed from available sandy soils, stability of the canal embankment and seepage are the main issues. High embankment canals are unsafe because of lack of required cohesion. When the particle size is almost identical, compaction is difficult and hence the mechanical interlocking of particles is not there which cause embankment failure. Lack of cohesion and compaction, both, could cause the issue of stability. Seepage is due to voids which is again because of uniform size of particles and poor compaction. The paper discusses the application of geosynthetics in construction of two branch canals of the Sardar Sarovar Project which have been constructed from sandy soils and underlines how geosynthetics help avoid the issues with that kind of soil.

1. CONTEXTUAL BACKGROUND

Gujarat state of India is on the west coast and the soil types are varying throughout the state. Northern part is alluvial and also includes the dessert of Kachchh having only sandy soil as shown in Figure 1.

Sardar Sarovar Project consists of a dam and reservoir on the Narmada River having its command area of 1.8 million hectare. Its canal network is 76000 kilometer

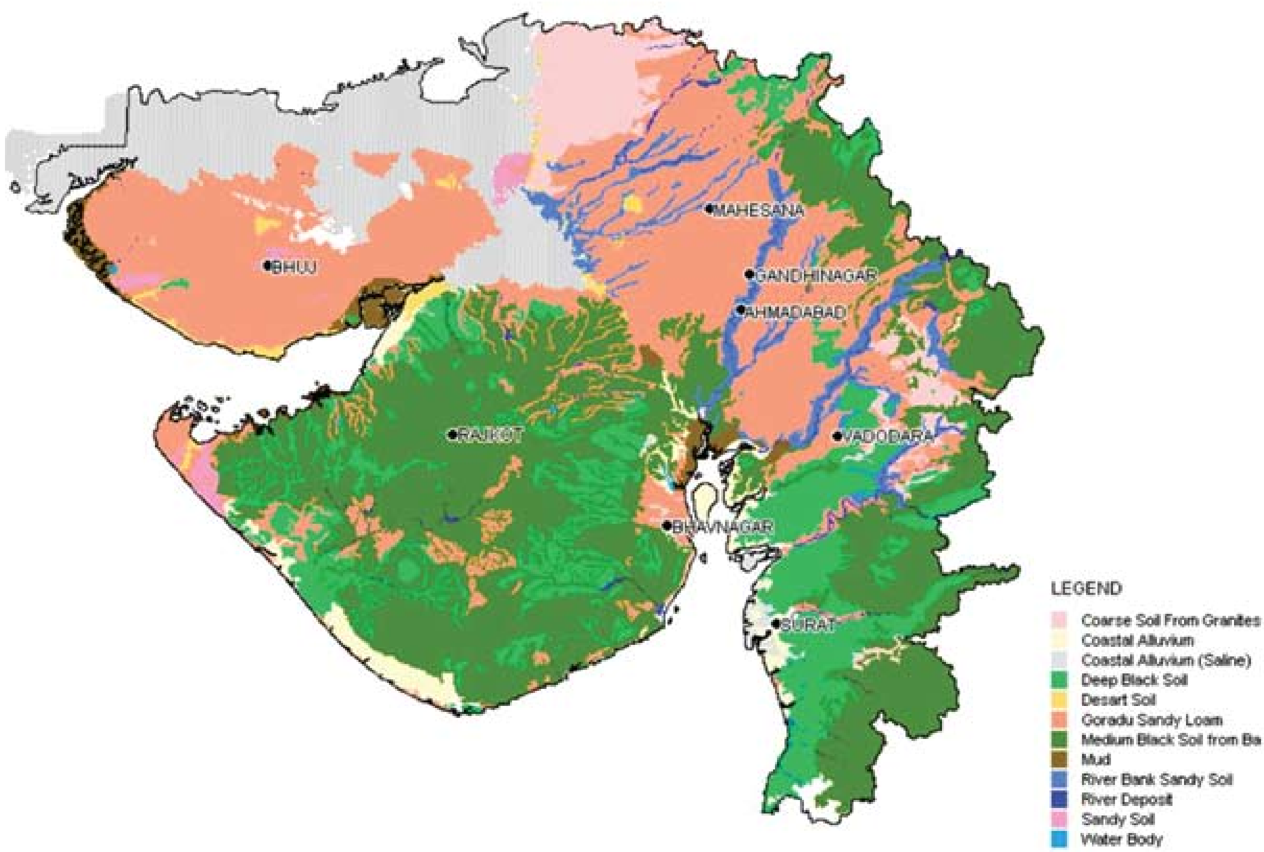

Fig. 1: Soil types of Gujarat

long. The command area is shown in Figure 2. A part of the command area is in the small dessert of Kachchh where locally available soil is only medium sand. Required quantity of good soil for construction of canals is available far off as whatever small quantity of good soil that was available sometime back was consumed in the infrastructure projects. Cost of long transportation of good soil in the given situation questions economic viability of canals in such areas. Therefore, conventional ways like using cohesive non-swelling soil can not be the solution. Instead of borrowing materials of desired qualities from far off, design with the available materials by applying some innovative ideas could be more viable.

Two branch canals of the Sardar Sarovar Project - Dhima Branch Canal and Gadsisar Branch Canal were designed to be constructed from the available sandy soil. Designed discharge of the said canals was 15 cubic meter per second and the length between 14 and 18 kilometer. Height of the embankment was approximately 4 meter.

2. CHALLENGES IN CONSTRUCTION OF CANAL EMBANKMENTS WITH SANDY SOILS

When the canals are to be constructed from sandy soils, two major challenges are there - stability of the canal embankment and seepage. High embankment canals are unsafe because the sandy soil does not have the required cohesion and hence mechanical failure is a common phenomenon. When the particle size is almost identical, compaction is difficult and hence the mechanical interlocking of particles is not there which causes embankment failure. Lack of cohesion and compaction could cause the issue of stability.

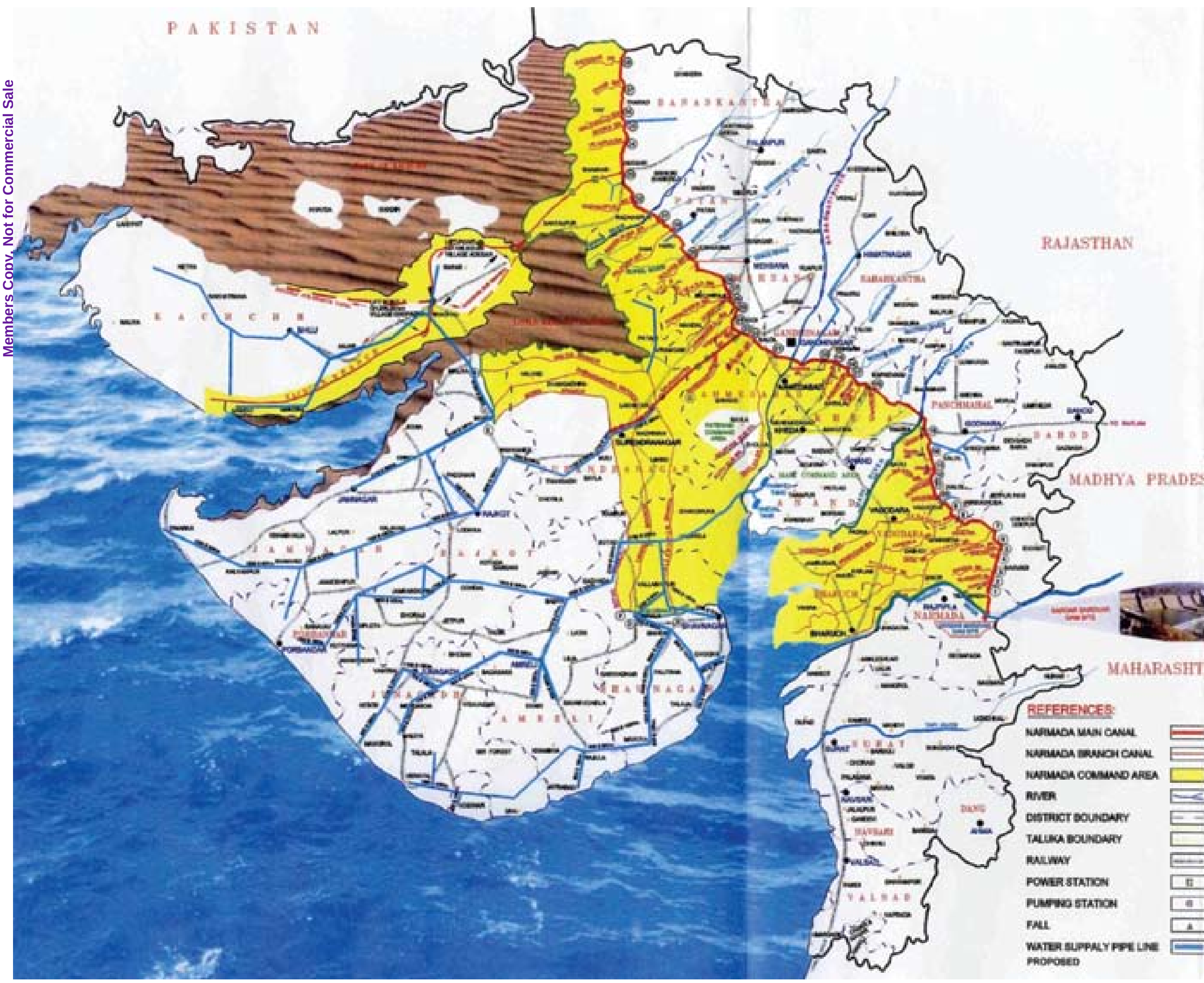

Fig. 2: Command area of Sardar Sarovar Project

Seepage is due to voids which again is because of larger particles and poor compaction. No lining can be completely impervious during continuous operation of the canal and hence seepage becomes a challenge due to voids which lead to high permeability. This results in to hydrostatic line much flatter than in the conventional embankment which may cut across the section above the toe as shown in Figure 3 and therefore wider section is required which again increases the cost. Seepage could also reduce the mechanical bond amongst the soil particles and hence could help the process of destabilization of the section. Dispersion of particles is a serious problem rather than a slip circle failure.

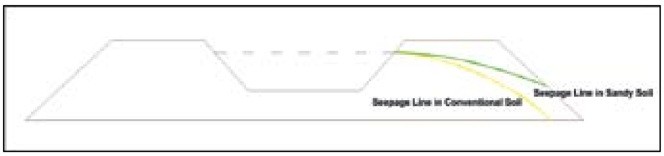

Fig. 3: Seepage lines in canal section

3. OPERATIONAL DIFFICULTIES IN CANALS HAVING EMBANKMENTS WITH SANDY SOILS

Canal Embankments with sandy soils may be designed with larger width or rock toe to ensure that the seepage line does not cross the section above toe but difficulties in operation are still there. Water levels in the canal fluctuate as per the demand variations in the command area. The water fluctuations result in to sudden variations of pore pressure in the embankment with sandy soil due to presence of large number of voids. Pore pressure variations may cause spreading or dispersion failure of the embankment as shown in Figure 4 because the sandy soil has large number of voids, poor mechanical interlocking due to uniform particle size and poor cohesion. Therefore, the operation of the canal requires too much of care and skill.

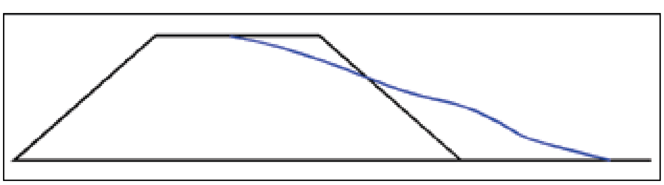

Fig. 4: Spreading failure of canal embankment

4. SOLUTION THROUGH GEOSYNTHETICS AND ASSOCIATED ISSUES AND THEIR STRATEGIC SOLUTIONS

Because of the sand as predominant type of soil, stability calculations were not showing safe embankment in the Dhima and Gadsisar Branch Canals or they require large embankment due to almost flat phreatic line. In order to ensure stability, it was thought wise to make right use of the high value of co-efficient of roughness of sand and design the canal section as reinforced earthen embankment. It worked very well and with the conventional size of the canal sections, the stability was obtained theoretically. From construction point of view, Geogrids were found more suitable and hence were provided for the slope stability. The design was carried out as per Guide to Reinforced Fill Strcuture and Slope Design, The Government of Hong Kong, 2002.

As per design, the properties of the geogrid required were as per Table 1 and three levels of geogrid as per height of the canal embankments were provided as following.

1st Layer at [Canal Bed Level – 0.30] m level

2nd Layer at [Canal Bed Level + 0.40] m level

3rd Layer at [Full Supply Level - 0.40] m level

Generally the concrete lining is saturated due to continuous flow of water. After the lining is saturated, the seepage line develops in sandy embankment in a short period. Sometime, cracking of lining in course of time is also not possible to be ruled out which could lead to high seepage. Due to pervious embankment, seepage line is much flatter as compared to cohesive embankment. In spite of georein for cements leading to stable embankment, the seepage line may require wider embankment to ensure it to be within the embankment width. Therefore, it was imperative in the design to check the entry of water in to the embankment. The best option was to provide geomebrane.

Table 1: Properties of geogrid

Property		Test Method	Unit	TG U-60
Ultimate Tensile Strength	MD CD	ASTM D-6637	kN/m	60 20
Reduction Factor (RF) and Machine Direction Long Term Design Strength (LTDS)				
Creep				1.55
Installation Damage	Sand/ Silt/ Clay			1.05
	<7.5 mm Gravel			1.15
Durability	pH – 4 to 9			1.15
LTDS – 120 Years, 40° C : Sand/ Silt/ Clay; pH – 4 to 9			kN/m	32
LTDS – 120 Years, 40° C : Gravel < 7.5 mm ; pH – 4 to 9			kN/m	29.3
Aperture (± 2 mm)			mm	30 X 25

In providing the geogrid, no significant problem was found in the construction methodology. The real issues were in subgrade preparation and concrete lining on geomembrane as it was to be done with the paver machine. In sand, subgrade used to be disturbed in a very short period and hence soon after preparation of the subgrade the lining was required to be taken up. Therefore, the construction schedule became very clumsy. The complexity was much greater due to paver machine which obviously cut the geomembrane. The solution of this issue was worked out by devising a special construction sequence. In the first stage, the lining was taken up for one entire side of the canal section plus little more than the half bottom width of the canal section. For this purpose, the geomembrane was got manufactured for a width equal to inclined side length of the canal plus little more than the half bottom width of the canal plus some overlap. The rail of the paver machine was placed off-centered in the bottom of the canal. In the second stage, the lining was done on the other slope of the canal and remaining bottom width. The width of the geomembrane was equal to inclined side length plus remaining bottom width plus some overlap. Thus, the geomembrane was manufactured in different widths. In the second stage, the rail of the paver machine was placed on the concrete lining. Because the construction joint was required to be in a staggered fashion, lest it might develop a longitudinal failure in course of time, the first and second stages were operated in alternate sequence in different stretches of the canal length. The completed cross section of the canal was as shown in Figure 5. First and second stages of lining are shown in Figures 6 and 7.

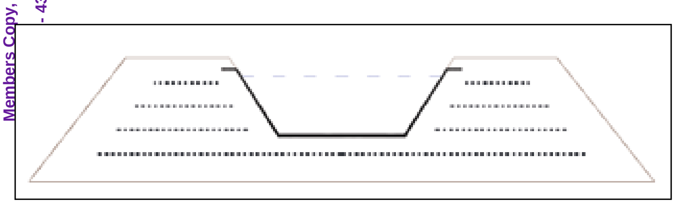

Fig. 5: Canal section with geo reinforcements and geomembrane

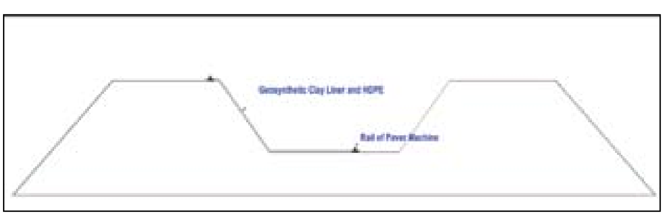

Fig. 6: First stage of concrete lining with paver machine

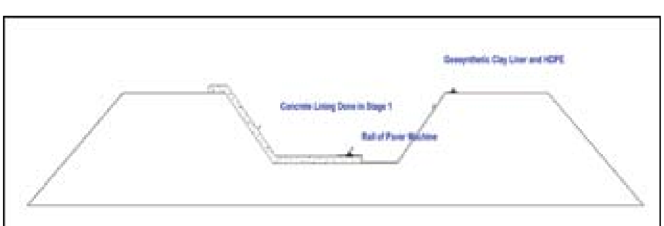

Fig. 7: Second stage of concrete lining with paver machine

Another problem with lining with geomembrane and paver machine was that very thick HDPE sheet was required to take care of the load. Moreover, due to continuous operation of canal, some settlement in the embankment was also likely which would result in to heavy tension in the HDPE which also required thick material. Handling thick sheet and placing it in place was very difficult. The solution of this problem was derived from the old technique of using cohesive non-swelling soil pad beneath the lining for the canals having sandy embankments as shown in Figure 8. Here, borrowing large quantity of clay was not possible and hence Geosynthetic Clay Liner was used as an alternative. Its function was to become impervious when come in contact with water. Because its upper surface was delicate, HDPE sheet was used as a protective layer which was only 0.5 mm thick. The two-layered geomembrane - Geosynthetic Clay Liner and High Density Poly Ethylene provided for checking the seepage was designed to take the load of the handling and paver machine. Because of smooth surface of the HDPE on which the concrete lining was to be placed by paver machine, the issue of slipping down of the fresh concrete was solved by modifying the concrete mix design. Though the main function of geomembrane was to check the entry of water, its role was not limited to do so. Because some shrubs tend to crop up from the soil, geomembrane was supposed to take the stress induced as it would be first attacked by such cropping of shrubs and the lining would be protected from such forces. This clearly drives home one point that the canal treated with geomembrane is supposed to have no weed growth in the canal prism leading to much better performance as compared to the canal with no geomembrane which almost always has weed growth inside the section. This aspect is a value addition kind of contribution by the geomembrane.

Fig. 8: Canal section in sandy embankment with cohesive non-swelling pad

The said proposition was properly designed and implemented in 2013-14 and Photograph 1 shows how it is flowing. Both the canals - Dhima and Gadsisar Branch Canals are flowing very well. They have also witnessed a heavy rainfall - over 100 cm. in 3 days in the year 2015 which is unprecedented in a dessert. The neighbouring canal got heavily damaged but the said canals sustained even heavy rainfall and also the inundation for some days for which the canals are never designed.

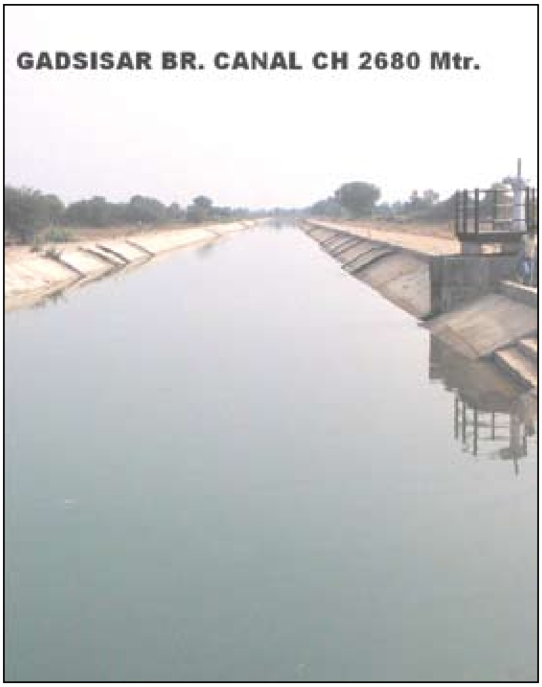

Photo 1: Completed Gadsisar Branch Canal

7. CONCLUSION

While constructing infrastructural facilities, in some cases, the availability of soil of a particular type is a problem leading to unviability of construction due to unsurmountable technical challenges. If the issue is tried to be sorted out by borrowing right quality soil from far off, the exorbitant cost hike makes the project unaffordable. In such a situation, solution with application of geosynthetics can really make the project viable technically and economically as well. However, it is clear that geosynthetics in itself can not become a solution in itself but can be used an essential part of the solution and hence the solution is required to be worked out as a wholistic system. Therefore, each case of application of geosynthetics is a customized solution and contains some uniqueness.

REFERENCE

Guide to Reinforced Fill Structure and Slope Design, Civil Engineering Department, The Government of the Hong Kong 2002

ISSUES IN USING GEOSYNTHETICS IN HYDRAULIC STRUCTURES UNDER RESTORATION

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ABSTRACT

Granular filters were traditionally used as filter media in embankment dams. Now-a-days, Geosynthetics are being increasingly used as filter media in embankment dams. There are some critical issues that need to be addressed when geosynthetics are recommended in place of granular filters. Durability and service life of geosynthetics is an important concern, particularly when they are subjected to humid environment; and, inaccessibility for frequent inspection, and repairs if needed, are also a matter of concern. These issues are discussed in review paper. The focus is on use of geosynthetics in embankment dam restoration works in existing dams.

Keywords: *Geosynthetics, Embankment dams, Filter Media, Dam Restoration.*

1. INTRODUCTION

Granular filters were traditionally used as filter media in embankment dams. Now-a-days, Geosynthetics are being increasingly used as filter media in embankment dams. This paper deliberates the durability and service life of geosynthetics used as functional elements in embankment dams. The importance of seepage control and periodic remediation of embankment dams is dealt in Section 3. Section 4 & 5 presents the designer's dilemma in deciding to go in for geosynthetics in place of more conventional natural granular filter media. Section 6 details the ASTM Standards for determining geosynthetics (physical and other) properties. This is followed by the Discussion and Conclusion Sections.

2. LITERATURE REVIEW

Some of the important references citied in this paper are given below.

1. *Geotextiles in Embankment Dams*: Status Report on the Use of Geotextiles in Embankment Dam Construction and Rehabilitation[1], Federal Emergency Management Agency, April, 2008. It is an authoritative and comprehensive document on the state of the art on the topic? The information available in this document is extensively made use of in the preparation of this paper, and also, in drawing many of the conclusions therein. The definition sketch as Figure 1 and the information on ASTM Standards for various tests are exclusively drawn from this document.
2. Durability of geosynthetics for highway applications, Federal Highway Administration Reports[2]. The FHWA Reports. FHWA-RD-97-142, 97-143, 97-144, and 00-157 give valuable information on the durability and aging process of the geosynthetics and the relevant tests thereof.
3. *Technical Note 2*: Durability of high-strength geosynthetics[3] by Tencate® Mirafi®, explains the manufacturer's version of durability criteria for selection of geosynthetic products for specific situations and work environments. Though the focus of this technical note is the strength reduction associated with aging and the 'durability reduction factor (RFD) 'associated with durability testing, valuable information on the different types of geosynthetic products available in the industry and their uses is presented in this report.
4. Other references from which information were extracted in the preparation of this paper, are included in the References Section.

Literature on the degradation and aging process of geosynthetics, indicate four factors that influence the design life of geotextile. These are: the physical environment, the chemical environment, the installation method and the storage conditions. These are explained later in the section on "Geosynthetic properties and tests".

3. PERIODIC REMEDIATION OF EMBANKMENT DAMS AND SEEPAGE CONTROL

3.1 What are Hydraulic Structures?

Hydraulic structures are water retaining structures such as: dams, weirs, barrages, check dams etc. Dams are classified based on: their uses (irrigation, hydropower, flood control, navigation and/or multipurpose); their function (storage, detention, diversion, coffer dam, debris dam or check dam, etc.,); their size (large, medium, small, and micro dams); their materials of construction (concrete, masonry, steel and timber (rigid dams); and earth dam, rockfill dam and tailings dam (non-rigid dams)); and also, based on their design aspect (gravity dam, buttress dam and arch dam).

If planned and constructed properly and as per specifications; generally, dams have long service life (25, 50, 100 years), and from a structural point of view, normally they outlive their service life. This is because, the functional aspects of the other components of the project such as powerhouse, reservoir, canals etc., become defunct or undergo remodelling much earlier. However, like any other civil engineering infrastructure project, dams also encounter problems of structural fatigue as they age, and also, develop maintenance issues during their service period.

3.2 Why Internal Drainage is Provided in Dams?

Since dams have one side for potential water retaining, and invariably there is seepage water in the body of the structure, the constant exposure to moisture in the dam body is a sure fact of their existence.

In rigid dams such as concrete and masonry dams, an uplift pressure caused by blocked drainage in the dam body is a major cause of worry. When the dam is constructed, vertical drainage holes are drilled in the dam body at intervals throughout the longitudinal profile for relieving any seepage water present in the dam body. These are mostly connected to internal drainage and inspection galleries that run though the dam body, and the drainage water is exited out of the dam body on the downstream side either into the open drainage or pumped out if the elevation of the exiting drainage is below the natural river bed.

Earthen dams (non-rigid dams) are susceptible to degradation resulting from cracks on the upstream face, erosion of the facing materials, piping of fines from dam core, all of which may result in unacceptable amounts of leakage through the dam. Internal erosion of the dam body due to seepage forces is of significant concern due to potential piping and structural instability that could lead to catastrophic failure. External erosion by the reservoir water generates cracking in the facing of the dam, which leads to an increase in the flow of water into the dam body, and ultimately, to internal erosion and structural instability. Unless properly treated, seepage of reservoir water through dam foundation and abutments can also present serious problems. The main purpose of controlling the seepage of water through a dam foundation is to prevent the foundation material from piping and washing away, which could result in structural failure due to loss of support. Another purpose is to reduce the foundation uplift pressures.

A typical earth dam design has a near impervious central core and two flanges on their side, one on the upstream side and the other on the downstream side, consisting of more pervious locally available earth or rockfill material. The dam's side slopes are so designed that the phreatic line stays within the dam body under all conditions of reservoir storage and depletion. To ensure this, mostly a downstream toe-drain is provided, which carry the (water-so-ever) seepage water that exit the dam-body, to natural drains or back to the river; and also, the internal downstream filter drains are extended into the dam body, adequately. The upstream internal filter drains are provided to quickly drain the seepage water in the upstream flange, when quick drawdown of the reservoir water level occurs. As mentioned earlier, traditionally, the above filter media is made up of well graded natural granular material. Now-a-days, factory engineered geotextiles offer excellent alternatives to the natural granular filters.

3.3 Why We Need to Carryout Restoration Works in Dams Periodically?

Dam safety is a public safety issue. Even when functionally a dam has completed its service life and economic viability, the project cannot be scrapped and the dam left to die. It should be ensured that the structure so left behind does not become a public safety risk. If needed, it is to be decommissioned. Generally speaking, the structural life of a dam exceeds several times its useful life because of many other factors such as the excessive sedimentation of the reservoir, the project becoming economically unviable, the project being technologically outdated, etc. Need for remediation arises from: (1) minor defects in original construction manifesting, with passage of time, as major seepage sources in dam, (2) damages caused to the embankments due to external factors and events such as floods, new developmental activities in the vicinity, etc. Purpose of the restoration is to bring the structure back to near original performance.

4. USE OF GEOSYNTHETICS - IN EMBANKMENT DAMS RESTORATION WORKS IN EXISTING DAMS

Geotextiles are used in a variety of applications in embankment dam construction and rehabilitation. Geosynthetic applications in dams specifically are: barrier to fluid, drainage, protection of geomembranes, filtration, reinforcement, and surficial erosion control. Various geosynthetics products are: Geotextiles, Geogrids, Geomembranes, Geocells, Geosynthetic Clay Liners (GCL), Geonet, Geopipe, Geofoam, Drainage cells and Geocomposites. Out of these, those mostly used for the drainage and filtration functions (which are the focus of this paper) are: Geotextiles, Geogrids, Geocells, Geonet, Geopipe, Drainage cells and Geocomposites. [Verma 1994]

Some examples where geosynthetics are used as filter media in embankment dams are given below: In Valcros Dam, France (1970) and in Many Farms Dam, Arizona (USA) the geotextile filter was used in the downstream side of dam-wrapped around gravel drain and toe drain pipe, respectively. In Sidi M' Hamed Ben Taiba Dam, Algeria (2003), the geotextile was wrapped around horizontal drain trenches installed in the upstream side of the dam. In Dzoumogne Dam, France (2000), a non-woven geotextile was used as filter around a gravel chimney and blanket drain and in Samira dam, Niger (2000), a non-woven geotextile was used wrapped around vertical sand drain and horizontal blanket as filter. (Artieres et al. 2009). In more recent literature, in Kandaleru Earthen Bund Reservoir Slope-Stabilization Work (500 m), Chennai, India (2010), the drainage layer was laid using polypropylene instead of conventional filter-material. (Raju 2010).

A geotextile will perform a filtration function if it allows liquid (water) to pass while controlling the soil or particulate migration through the geotextile. The geotextile, therefore must be located between the soil being retained and the open drain material, perforated pipe or drainage geosynthetic (i.e. geonet). Typically, nonwoven needle-punched geotextiles are used. However, woven monofilament geotextiles have performed well and knit geotextiles have been used around perforated pipes as a two stage filter in combination with a primary sand filter layer.[1]

A geosynthetic that is thick and permeable in its plane will provide a planar conduit for fluid (or gas) flow. This function of planar flow is usually performed by a geosynthetic designed for planar flow such as a geonet, geonet composite, structured (drain) geomembrane, thick coarse fibre geotextile, geocomposite (geomat) or wick drain. It is important to note that a geotextile is often used as a filter prior to fluid entering a transmissivity geosynthetic (i.e. wick drains and geonet composites[1]

It is important from a designer's point of view to know what the functional application is and where it can be used relative to embankment dams in order to consider a geotextile for use in a particular function. Figure 1 shown is a 'Definition Sketch' reproduced *as is* from the FEMA Report (2008, pp 51, Figure 2.28)[1] with the caption at the source literature as "Examples of some of the functions of geotextiles in dams". It is reproduced here because of clarity with which the figure provides information on the various functional elements the geotextile can perform in an embankment dam. The focus of this paper is to cover use of geosynthetics / geotextiles for only the 'filtration' and 'drainage' functions. As such information available therein on other functions, namely, separation, planar drainage, reinforcement and protection are not extracted here. A geotextile (nonwoven or woven) performs a filter function if it allows water to pass while controlling the soils migration through the geotextile. Sometimes the filtration application is a dual function (i.e. filtration and drainage). The locations or the functional elements in the dam are: (a) filter between zones of protective stone and embankment soils or transition soils, (b) internal chimney drain upstream filter, (c) internal toe-drain filter, (d) internal downstream filter, and (e) external toe-drain filter.[1]

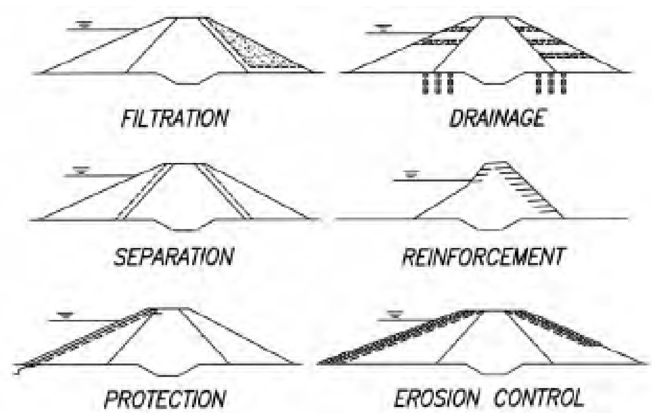

Fig. 1: Examples of some of the functions of geotextiles in dams

Source: FEMA Report (April 2008), Chapter 2—Overview of Geosynthetics for Use in Embankment Dams, Figure 2.28.with caption "Examples of some of the functions of geotextiles in dams".

5. USE OF GEOSYNTHETICS – THE DESIGNER'S DILEMMA

The designer's dilemma in recommending geosynthetics in a project which has so far not used geosynthetic products as a functional element in an embankment dam reflects the following concerns:

(a) dam safety risk in case of failure of the new product as per expectations,
(b) degradation properties and service life,
(c) inaccessibility after construction phase, and,
(d) Cost of replacement.

Let us discuss these one by one.

5.1 Dam Safety Risk

FEMA Report reiterate the current state of practice in USA vis-a-vis those in Europe and elsewhere, as under:

'Most practitioners in the United States limit the use of geotextiles to locations where there is easy access for repair and replacement (shallow burial), or where the geotextile function is not critical to the safety of the dam, should the geotextile fail to perform.[1]. In a limited number of cases, geotextiles have been used as deeply buried filters in dams in France, Germany, South Africa, and a few other nations. Most notable, is the geotextile installed as a filter for Valcross Dam which has been successfully performing for over 35 years. These applications remain controversial and are not considered to be consistent with accepted engineering practice within the United States. Because geotextiles are prone to installation damage and have a potential for clogging, their reliability remains uncertain. Many organizations for bid their use in embankment dams in critical applications where poor performance could lead to failure of the dam or require costly repairs'.[1]

5.2 Degradation Properties and Service Life Expectancy

In contrast to other infrastructure projects such as in highway projects (where geosynthetics are extensively being used), in the case of hydraulic projects, the geotextiles are constantly exposed to presence of moisture in the environment. Whether the physical properties of geotextiles will deteriorate in a more humid environment than in a dry environment?

The tests for determination of hydrolysis degradation indicate that the chemical composition of the geotextile as well as the pH value of the liquid in which it is immersed influence the degradation rate. Geosynthetics made out of polyester resins (PET) are most affected by hydrolysis when immersed in liquids with high alkalinity. Since both the conditions are absent in the 'flood control' and 'irrigation' embankment dams, these concerns can be adequately addressed by choosing the right kind of geosynthetics product for the project.

The degradation properties are: (a) Ultraviolet (UV) Degradation, (b) High temperature Degradation, (c) Oxidation Degradation, (d) Hydrolysis Degradation, (e) Chemical Degradation, (f) Radioactive Degradation, (g) Biological Degradation, (h) Other degradation such as ozone, rodent or termite attack, and, (i) Aging.

In rigid dams (such as concrete and masonry dams), when geotextiles are used in the exposed situations, the quality deterioration of the physical properties of the geosynthetics with elapse of time under the combined action of heat and ultraviolet radiation is a matter of concern.

For waste containment and for shallow burial in extreme climatic conditions, temperature degradation should be considered. In embankment dams for irrigation projects, with geosynthetics under buried conditions, temperature degradation is not a major concern.

5.3 Inaccessibility After Construction Phase

If the location where the engineered product is used is accessible for inspections and replacement is easy, it offers no serious problems for the maintenance engineers at the project. In earthen dams, when geotextiles are used for either upstream protection or downstream protection, once the geotextile is placed in position and the over burden soil layers are placed; it is not possible to go back to the geotextiles for repairs or modifications. In earthen dams after impounding, since hydraulic pressure exerted by seepage water is always present in the core section of the dam, new insertion or replacement of already placed geotextile is a difficult task. Puncture of the geosynthetics by big animals walking over the area or by burrow animals digging holes in the geosynthetics is one of the many concerns while using geosynthetics in embankments and levees.

Service life of the manufactured / engineered product vis-a-vis that of the natural materials used in the construction of dam in locations which are inaccessible after the construction phase of the project is over and the maintenance phase of the project is commenced, is a major area of concern. The engineered product is generally, given a life-span estimate in the form of guarantee certificates (in number of years of trouble free service under normal prescribed maintenance protocols) given by the manufacturer of the product. What happens after this service period is over is the big question. Should the product be inspected in place, and if needed, replaced if found that the performance qualities have deteriorated beyond local repairs?

5.4 Cost of Replacement

But if the location is inaccessible after construction for normal maintenance, and replacement of the product will entails removal and replacement of additional overlaying layers of soil and rock, then the procedure and costs of such operations are to be well thought about much in advance and planned so.

Cost is major concern while considering whether or not to use or in the choice of specific geosynthetics product. Large demand within the country and resultant increase in production may bring down unit prices of these products eventually.

5.5 Manufacturer's Views

According to Ten Cate® [3], "The purpose of this technical note is to discuss the relation between the potential affect of chemical and biological degradation and the partial reduction factor for durability, RFD, used in the calculation of the long-term design strength of their *branded* geogrids and high-strength woven geotextiles."

Allaying most of the fears and concerns presented in earlier subsections, according to Ten Cate® 'Technical

Note 2 titled Durability', in road construction and other applications', geosynthetics fulfil one or more functions: separation, filtration, reinforcement, drainage, protection and erosion control; and for drainage systems, they have developed a products range from which, the geosynthetic with optimal filtration properties for a specific application can be selected. Similarly, they have developed geosynthetic composites that function as drainage systems. According to them, "Durability is a major issue for all polymeric materials, including geosynthetic-reinforcing materials geogrids and high-strength woven geotextiles, when long design lifetimes are required. Degradation of a polymeric material typically results in a loss of strength. All generally accepted soil reinforcement design procedures require that this potential strength loss be accounted for in the determination of the reinforcing material's long-term design strength".

6. GEOSYNTHETICS PROPERTIES AND ASTM TEST PROTOCOLS

The FEMA Report (2008)[1] in article 2.12 describes the various geotextile properties and their testing methods. It is mentioned that the geotechnical manufacturing community and the geosynthetics industry in general are mature in that a completely unified set of standards exists through ASTM, ISO, CEN, etc. The ASTM Standards relating to testing different properties of the geosynthetics are given grouped into five categories: Physical properties, Mechanical properties, Hydraulic properties, Endurance properties, and Degradation properties. These are:

1 *Physical Properties*

Specific Gravity – (ASTM D792 or D1505)
Mass per Unit Area – (ASTM D5261)
Thickness – (ASTM D5199)
Flexural Stiffness – (ASTM D1388)

2. *Mechanical Properties*

Compressibility – (ASTM D6364)
Tensile Strength – (ASTM D751, D4632, and D4595)
Seam Strength – (ASTM D4884)
Burst Strength – (ASTM D3786) (test now being phased out)
Tear Strength – (ASTM D4533, D751)
Puncture – (ASTM D4833, D6241)
Puncture – Hydrostatic Vessel (ASTM D5514)
Large Scale Performance Direct Shear (ASTM D5321)

3. *Hydraulic Properties*

Porosity – (ASTM D6767)
Percent Open Area (POA), Apparent Opening Size (AOS), Equivalent Opening Size (EOS) – (ASTM D4751)
Permittivity – (ASTM D4491, D5493)
Transmissivity – (ASTM D4716)

4. *Endurance Properties*

Creep – (ASTM D5262)
Abrasion – (ASTM D4886) ("Tabor Abrasion")
Clogging – Gradient Ratio (ASTM D5101),
Clogging – Hydraulic Conductivity Ratio (ASTM D5567)
Clogging – Biological clogging (ASTM D1987)

5. *Degradation Properties*

Ultraviolet (UV) Degradation – The most widely used test devices for UV resistance are Xenon-Arc (ASTM D4355) and Fluorescent UV (ASTM D5208).
Temperature Degradation –(ASTM D746)
Oxidation Degradation – (ASTM D794)
Hydrolysis Degradation – Polyester resins are the most susceptible to hydrolysis, especially when immersed in liquids with high alkalinity.
Chemical Degradation – (ASTM D543, ASTM D5322, ASTM D5496)
Radioactive Degradation – only an issue for waste containment
Biological Degradation – Micro-organisms do not readily degrade geotextiles
Other Degradation – Degradation processes such as: ozone, rodent or termite attack—this has not been found to be a problem in most geotextile applications
Aging – Specific test standards have not been developed to measure aging of geotextiles.

It may be noticed that the endurance and degradation properties of the geosynthetic determine the durability and service life of the product. Depending on a realistic assessment of the physical situation at the project site and anticipated working environment, the geosynthetic product should be selected/designed, if needed, in consultation with the manufacturers. The general conclusion is that If these assessments are realistic, the product's service life will be able to meet the expectations and that of the other functional elements in the structure.

7. DISCUSSION

In Section 4, we brought out how the filtration and drainage functional units in the embankment dam perform. The product range is impressive. The need for evaluating the option of using or not using geosynthetics for the given problem solution should become as standard practice for the design and planning teams for the project. This will ensure economic designs for restoration works.

In Section 5, we described the designer's dilemma of four issues: Dam safety risk, product durability and inaccessibility after geosynthetic placement in position and manufacturers' claims. It turn out that numerous products from many manufacturers are available in the market. The choice of the right manufacturers and their appropriate product for the project is the designer's skill in effecting cost effective solutions.

In Section 6, the geosynthetics properties and the corresponding ASTM test protocols for determining the specific geosynthetics property are listed. The physical properties mostly are descriptive property than a design property. The mechanical properties are important when the geosynthetics are used as reinforcement or soil stabilization. The hydraulic properties are of prime importance when the filtration and drainage functions are invoked. The endurance properties and the degradation properties are important while deciding on the durability and the service life of the geosynthetic.

In general, Granular filters in existing dams can be advantageously replaced by geosynthetics filters in certain situations. However this is to be a highly site specific decision. The service life of the geosynthetics should match with service life of the structure itself. There are a number of test procedures to determine the durability of the geosynthetics used. The durability of geosynthetics is dependent on: (a) The physical environment, (b) The chemical environment, (c) The installation method, and (d) The storage conditions.The factors to be tested for durability are: (a) Temperature stability, (b) Exposures to light and heat, (c) Exposure to natural elements, (d) Chemical resistance, and (e) Aberration resistance. Standard product manufacturers can design products with such properties as to be functional in a given environment. Geosynthetics and geotextiles in particular, must be treated as any other construction material used in civil engineering in that their strengths and weaknesses must be recognized and properly addressed in design and construction.

8 CONCLUSIONS

The paper is focussed on the use of geosynthetics as filter media (in place of granular filters usually provided in embankment dams) when remediation of existing embankment dams are taken up. The FEMA Report (April 2008)[1]is extensively cited. It is concluded that if the anticipated physical, chemical and biological site conditions are well articulated at the (restoration) planning stage itself, site specific geosynthetic product ranges can be designed and manufactured that would give optimal performance, and at the same time, have adequate durability and service life as that of the rest of the elements of embankment dam.

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REFERENCES

Abhilasha, P.S. and Thadicaran, Antony Balan, G., PhD, PE, (2014). Numerical Analysis of Seepage in Embankment Dams, *IOSR Journal of Mechanical and Civil Engineering, pp*13-23.

Abhilasha P. S., M.Tech., and Antony Balan G. Thadicaran, Ph.D., P.E., (2015) Use of Geosynthetics as Filter in Embankment Dams-–Need for a Dedicated Decision Support System, *Geosynthetics Conference-2015.* IFAI., Portland Oregon, USA. February 15-18, 2015

Antony Balan G. Thadicaran, Ph.D., P.E., and Abhilasha P. S., M.Tech.,(2013). Use of Geosynthetics as Filter Material in Blanket/ Core of Earthen Dams, CBIP, Bhopal, Madhya Predesh, India, May 2013.

Chapman and Hall. (1988). *Durability of Geotextiles,* CRC Press, ISBN: 0412306603, 9780412306600.

FEMA Report. Federal Emergency Management Agency, (April 2008). Geotextiles in Embankment Dams, "*Status Report on the Use of Geotextiles in Embankment Dam Construction and Rehabilitation"*

FHWA Report[2]. FHWA-RD-97-142 : Stress Cracking Potential of HDPE Geogrids`

FHWA Report[2]. FHWA-RD-97-143 : Development of Protocols for Confined Extension/Creep Testing of Geosynthetics for Highway Applications

FHWA Report[2]. FHWA-RD-97-144 : Testing Protocols for Oxidation and Hydrolysis of Geosynthetics

FHWA Report[2]. FHWA-RD-00-157 : Long-Term Durability of Geosynthetics Based on Exhumed Samples From Construction Projects.

Jafari, Navid H., S.M.ASCE; Timothy D. Stark, Ph.D., P.E., F.ASCE; and R. Kerry Rowe Ph.D., P.Engg., F.ASCE. (2013). Service Life of HDPE Geomembranes Subjected to Elevated Temperatures, *Jounal of hazardous, Toxic, and Radioactive Waste,* 10.1061/(ASCE)HZ.2153-5515.0000188, 16-26.

Murray, R.T. (1988). *The Users' Point of View. Durability of Geotextiles.* Source Agency: Transport Research Laboratory, Crowthorne House, Nine Mile Ride, Wokingham, Berkshire RG40 3GA, London England, United Kingdom, Accession Number: 00493035, p. 11-14, ISBN: 0-412-30660-3.

Tencate® Mirafi®. Technical Note 2: Durability of high-strength geosynthetics[3]. Website.

BEHAVIOUR OF GEOSYNTHETIC ENCASED GRANULAR COLUMNS UNDER VERTICAL AND LATERAL LOADING

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ABSTRACT

Granular columns installed below the embankments are subjected to combined action of vertical and lateral loading depending on their location. The columns installed near the centerline of the embankments are primarily subjected to vertical loading while those near the toe are subjected to predominant lateral loading. In this paper analyses are carried out separately to analyze the behaviour of geosynthetic encased granular columns (EGC) subjected to vertical and lateral loading. Along with small scale laboratory studies, 3-dimensional slope stability analyses are carried out using FLAC^{3D} software to analyze the behaviour of granular columns in model and prototype scales. From the analyses it is observed that geosynthetic encasement helps in improving the vertical and lateral load carrying capacity of the columns. EGC also helps to mobilize higher factor of safety compared to ordinary granular column. Encasement of column helps to reduce the area replacement ratio, which leads to consumption of lesser quantity of natural resources and better design procedure.

Keywords: *geosynthetic, vertical loading, lateral loading, soft clay, 3-dimensional*

1. INTRODUCTION

The soft foundation soil at several infrastructure sites, such as port and harbor structures, bridges, highway and railway embankments etc. may have low bearing capacities and high compressibility to support the structures within allowable settlement limits. The conventional foundations in poor foundation soils will prove expensive and thus, in-situ ground improvement techniques are adopted for ground treatment. Depending on the type of soil and the type of structure, various ground improvement techniques are used to improve the behaviour of foundation soil. The particular ground improvement technique employed for a given project depends largely on economic feasibility and lead time available. This challenges the geotechnical engineer to find innovative solutions in improving the engineering properties of soil tailored for the design requirements. The proposed design should take into account the economy of the project, able to minimize the construction period and also need to be environmentally sustainable.

The granular columns have been widely used for improvement of soft clay soils. Granular columns (also known as stone column, sand column or granular piles) are formed by replacing some part of the native ground with granular materials (sand or aggregates). Generally granular materials are compacted using vibro-flotation technique to form a cylindrical structure in the ground which is known as granular column (IS 15284 Part-1 2003). These columns have high stiffness and better shear strength compared to the virgin soil, which helps to improve the load carrying capacity of the foundation and helps in reducing the settlements. They can accelerate the rate of consolidation of soft ground, minimize the post construction settlement, improve the bearing capacity, reduce total and differential settlements, improve stability of slopes and reduce the chances of liquefaction (Barksdale and Bachus 1983; Alamgir et al. 1996; Adalier and Elgamal 2004). Granular columns can be used in soils which are not conducive to chemical reactions required for lime or deep mixed columns. Ordinary granular columns (OGC) are widely used to support flexible and rigid structures such as embankment, oil storage tank, buildings etc. (Han and Gabr 2002, Murugesan and Rajagopal 2006; Gniel and Bouazza 2009, 2010; Deb and Mohapatra 2013; Indraratna et al. 2013; Zhang et al. 2014; Ali et al. 2014; Shahu and Reddy 2014) constructed over soft clay.

It is well established that granular columns derive their load carrying capacity due to lateral confinement from the surrounding soils (Hughes et al. 1975). In case of very soft clay (s_u <15 kPa) owing to low lateral confinement, granular columns may not derive significant load carrying capacity (Raithel et al. 2002). According to Chummar (2000) and Thorburn (1975), bearing capacity of the improved ground cannot be increased beyond 25 times the initial undrained shear strength of clay. Apart from insufficient lateral confinement, contamination of granular materials with fine clay particles can reduce their frictional properties and affect its drainage efficiency (Murugesan and Rajagopal 2008).The above limitations of OGC can be overcome using geosynthetic encasement. Geosynthetic encasement provides additional confinement to the granular columns which helps to improve their load carrying capacity (Murugesan and Rajagopal 2006, 2007, 2008, 2010). The filtration property of the geosynthetic encasement also helps to prevent the clogging of granular columns, which helps to maintain their discharge capacity and shear strength properties.

Various studies are reported in the literature to understand the behaviour of encased granular column (EGC) subjected to vertical loading (Yoo and Kim 2009; Gniel and Bouazza 2010; Khabbazian et al. 2010; Lo et al. 2010; Murugesan and Rajagopal 2010; Pulko et al. 2011; Keykhosropur et al. 2012; Dash and Bora 2013; Elsawy 2013; Ghazavi and Afshar 2013; Chen et al. 2014; Hosseinpour 2014, 2015). In practice granular columns installed below the embankment are subjected to combined action of vertical and lateral loading depending on their location (Fig 1). Columns near the centerline of the embankment are primarily subjected to vertical loading. Towards the toe of the embankment the loading direction gradually changes from vertical to lateral. Near the toe columns are primarily subjected to lateral loading as shown in Figure 1.

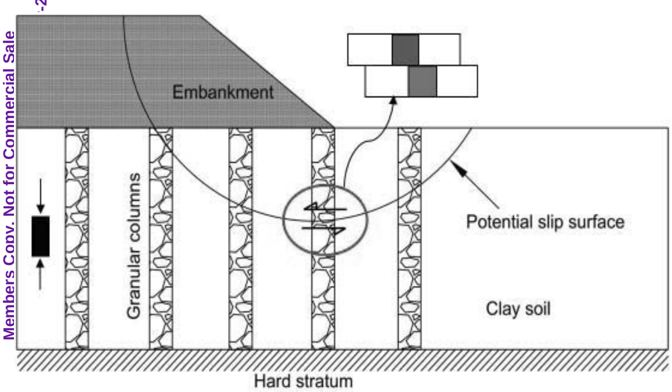


Fig. 1: Deformation pattern of granular columns below an embankment

In this paper analyses are carried out separately to analyze the behaviour of EGC subjected to vertical and lateral loading. Along with small scale laboratory studies, 3-dimensional (3-d) slope stability analyses are also carried out using FLAC^{3D} (Fast Lagrangian Analysis of Continua in 3-dimensions) software to analyze their behaviour in both model and prototype scales. From the analyses it is observed that geosynthetic encasement of granular columns helps in improving the vertical and lateral load carrying capacity. From the slope stability analysis it is observed that encased granular column helps to mobilize higher factor of safety (FS) compared to equivalent ordinary granular columns. Higher secant modulus of geosynthetic encasement found to change the failure mechanism from deep seated to toe failure due to increase in strength of the foundation soil. EGC helps to reduce the area replacement ratio (a_s), which leads to consumption of lesser quantity of natural resources and better design procedure.

2. LABORATORY STUDIES

2.1 Lateral Loading on Granular Column

The behavior of granular columns subjected to lateral loading was studied using large direct shear (LDS) box apparatus. Model granular columns were installed in the shear box and they were subjected to lateral loading by moving the lower box relative to the upper box. The details of LDS equipment, materials and the tests procedure are explained in detail in the subsequent sections.

The 3-d view of automated LDS box with different components is shown in Figure 2. It consist of two boxes (upper and lower) filled with compacted soil. The plan size of the upper box is 305 mm < 305 mm. Width of the lower box is same as that of the upper box but it is of longer longitudinal dimension to maintain the uniform contact surface during shearing of the soil and to prevent the loss of soil from the upper box. The upper box is restrained at its sides (not shown in figure) whereas the lower box is free to move in the horizontal direction on smooth rollers. The arrow direction in Figure 2 indicates the direction of movement of the lower box. The shear force generated during the movement of the lower box is measured using an S-type load cell of 44 kN capacity with a resolution of 0.025 kN and the shear displacement is measured using an LVDT as shown in the figure. All the data are recorded at regular time intervals and stored in a computer with the help of a data logger. Normal pressure is applied on the soil sample using a rubber bladder which is attached to the bottom of the top plate. An air pressure regulator is used to control the applied pressure and the actual pressure on soil sample is measured using pressure gauge attached on the top plate. The gap between the upper and lower box was maintained uniform at 2-3 mm for all the tests.

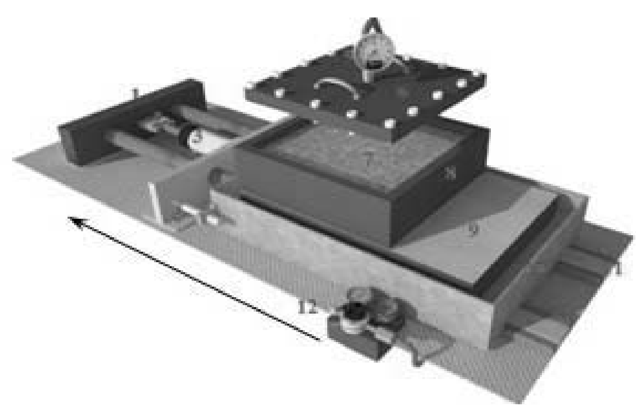


Fig. 2: Three dimensional view of large direct shear box

Sl. No.	Component Name
1	Reaction frame
2	Load cell
3	Actuator
4	LVDT

5	**Pressure gauge**
6	**Top plate**
7	**infill soil**
8	**Top box**
9	**Bottom box**
10	**Outer box**
11	**Rail**
12	**Air Pressure Regulator**

2.1.1 *Test Procedure*

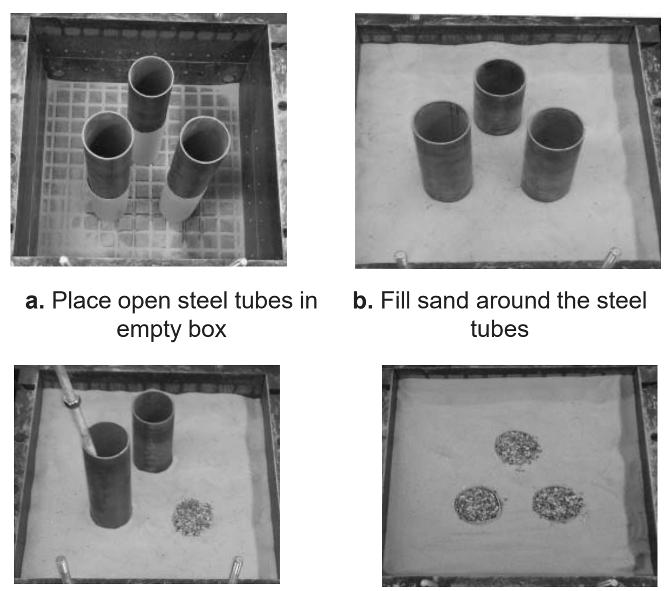
Dry fine sand having coefficient of uniformity (c_u) and coefficient of curvature (c_c) of 1.58 and 0.98 respectively was used in the tests. Instead of normally consolidated clay soil, sand was used due to easem of placement, ease of achieving consistency between the tests and similar strength behavior (Prisco et al. 2006, Mohapatra et al. 2014). Two types of aggregates were used for preparation of granular columns. The 50 mm diameter granular columns were constructed using aggregate passing through 4.75 mm and retained on 2 mm sieve. The c_u and c_c values of the aggregates are 1.55 and 0.97 respectively. 100 mm diameter granular columns were constructed using aggregates passing through 9.5 mm and retained on 2 mm sieve. The c_u and c_c values of this aggregate are 2.0 and 0.91 respectively. Ratio of diameter of granular column to maximum size of aggregate was kept 10 to avoid the size effects (Fox 2011, Stoeber 2012). The sand and the aggregates can be classified as SP and GP respectively as per USCS classification. Various properties of sand and aggregates are given in Table-1. Detailed explanations for the use of sand and different size of aggregates are given by Mohapatra et al. (2014, 2016). A woven geotextile having ultimate tensile strength (ASTM D4595) of 34 kN/m at 37% strain, attached using quick setting epoxy adhesive was used to fabricate the encasement tube. Constant seam width of 15 to 20 mm was maintained for all the specimens.

Table 1: Properties of sand and aggregates

Materials	D_{10}		D_{30}		D_{60}	
	50 mm φ	**100 mm φ**	**50 mm φ**	**100 mm φ**	**50 mm φ**	**100 mm φ**
	2.2	**2.6**	**2.7**	**3.5**	**3.4**	**5.2**
Sand	**0.24**		**0.3**		**0.38**	

The sample preparation procedure for EGC was divided into four steps as explained in Figure 3. In step-1, (Figure 3a) a hollow cylindrical steel tube with geosynthetic encasement was placed inside the large direct shear box at the required position. The inside diameter of the tube was equal to the diameter of granular column. In step-2, (Figure 3b) sand was poured around the steel tubes in three equal layers and compacted using needle vibrator to achieve the required relative density (72%), which corresponds to a dry density of 1.66 g/cm^3. In step-3, (Figure 3c) aggregates were placed inside the tube in three equal layers and compacted using a steel rod to achieve the required dry density. The dry density achieved after compaction for 50 mm and 100 mm diameter granular columns were 1.65 g/cm^3 and 1.75 g/cm^3 respectively. In the last step, (Figure 3d) the steel tubes were removed by pulling them vertically upward leaving the EGC inside the sand. The smooth internal and external surface of steel tubes minimized the disturbance to the sample. Proper care was taken to maintain the verticality of steel tubes while removing them from the shear box. Height of the sample in the shear box was always maintained at 140 mm. Similar procedure was followed for OGC without geosynthetic encasement.

Three different plan configurations were investigated for large direct shear testing which correspond to different a_s, as given in Table 2. The 50 mm and 100 mm diameter individual granular columns (C50 and C100) were installed at the center of the shear box which correspond to a_s of 2.11% and 8.44% respectively. To study the group effect, 50 mm diameter granular columns were installed in triangular (T50) and square (S50) arrangements at 100 mm center to center spacing. T50 and S50 correspond to a_s of 6.33% and 8.44% respectively. The tests were carried out at four different normal pressures of 15 kPa, 30 kPa, 45 kPa and 75 kPa. The details of large direct shear testing program are given in Table 2. All the tests were carried out at a strain rate of 1 mm/ min.



a. Place open steel tubes in empty box

b. Fill sand around the steel tubes

c. Place and compact aggregate in steel tubes

d. Remove steel tubes to form granular columns

Fig. 3: Installation process of granular columns in direct shear box

Table 2: Details of Large Direct Shear Testing Program

Test arrangement	Notation	a_s (%)
Dense sand	Sand	0
50 mm diameter granular column at center	C50	2.11
100 mm diameter granular columns at center	C100	8.44
50 mm diameter granular columns in triangular pattern	T50	6.33
50 mm diameter granular columns in square pattern	S50	8.44

2.1.2 *Results and Discussion*

Higher shear stresses are mobilized by treating the sand with OGC due to higher friction angle of aggregate (Figure 4). The columns and the sand around behave as a composite and helps in mobilizing higher shear resistance. With the increase in a_s higher shear stresses are mobilized. Triangular arrangement (T50) with lesser a_s mobilize higher shear resistance compared to single column at center (C100) with higher a_s due to group action of granular columns. Square arrangement (S50) was found to mobilize higher shear resistance compared to T50 due to higher a_s.

Figure 5 shows the results of woven geotextile encased granular column. From the figure it can be observed that considerable improvement in the shear stresses are observed due to encasement of granular column. Hoop forces mobilized by the geosynthetic encasement provides additional confinement to the granular column and also prevents its rupture failure (Figure 6). The arrow direction in Figure 6 indicate the direction of movement of bottom box. Compared to OGC in case of EGC strain hardening behaviour is observed at large shear displacement due to hoop forces mobilized in the encasement (Figure 5). Higher a_s helps to mobilized higher shear resistance in case of EGC. Due to group mechanism T50 having lesser a_s mobilize higher resistance compared to C100.

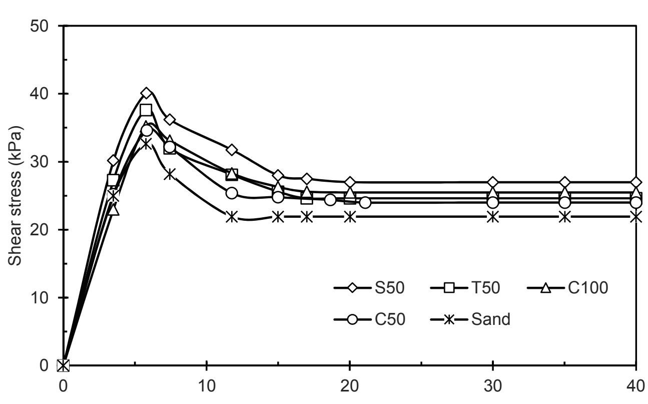


Fig. 4: Shear stress vs. horizontal displacement for OGC (σ =30 kPa)

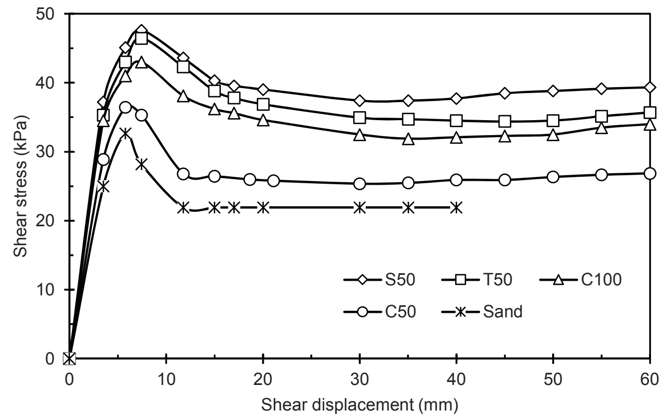


Fig. 5: Shear stress vs. horizontal displacement for EGC (σ =30 kPa)

2.2 Behaviour of Granular Columns under Vertical Loading

The results from the previous section have demonstrated that the encasement of granular columns increases their lateral load carrying capacity. In this section some results of granular columns subjected to pure vertical loading are presented.

In the present study the load tests were conducted on granular columns installed in a unit cell tank which would represent a typical granular column and the intervening soil in a grid of columns. The cylindrical unit cell tank used in this study was of 210 mm diameter and 500 mm height. The plan area of the tank is equivalent to a typical unit cell area of granular columns installed at a center to center spacing of 200 mm in square pattern and 186 mm spacing in triangular pattern. The clay bed in the unit cell tank for the laboratory tests was prepared by consolidating a clay slurry in a laboratory controlled conditions. The granular column of the required diameter was installed at the center of the tank with displacement method. More details of these tests are reported in Murugesan (2007).

Fig. 6: Mode of failure of EGC (C50)

The properties of clay bed formed in the unit cell tank are listed in Table 3. For every test, fresh clay bed was prepared with consistent properties to have proper comparison between different tests. The stone aggregates used to form the granular columns were granite chips of size 2 to 10 mm and having uniform gradation. The peak angle of internal friction of stone aggregates determined from the direct shear test data is 41.5° within a normal pressure range up to 300 kPa. The density of the stone aggregate in all the tests was maintained close to 1.6 g/cc. In the present study woven and nonwoven geotextiles were used as encasement for the granular column. As the geosynthetics were stitched to form the tube for encasing the granular column, the seam strength of the geosynthetic was also determined with geosynthetic specimens having a horizontal seam at mid length. The properties of the geosynthetic are listed in Table 4.

Table 3: Properties of Clay bed

Properties	Value
Liquid limit	49%
Plastic limit	17%
Specific Gravity	2.59
In-situ Moisture	47±1%
In-situ vane shear strength	2.5 kPa
Consistency Index	0.06
Dry unit weight	11.56 kN/m^3
IS Soil Classification Symbol	CH

Table 4: Properties of geosynthetics used for the encasement

Strength properties (kN/m)	Woven geotextile	Nonwoven geotextile
Ultimate tensile strength	20	6.8
Ultimate seam strength	4.7	5.1
Initial modulus (based on seam strength)	17.5	12

The granular columns thus formed were subjected to vertical loading at a constant strain rate of 1.2 mm per minute through a concentrically placed loading plate of diameter equal to that of the granular column. All the load tests were performed by loading only the granular column, in order to directly compare the improvement in the load capacity due to encasement. The loads corresponding to different displacements were measured using a proving ring (having accuracy of 0.01 kN). Figure 7 shows the schematic of the load test set up. Three series of tests were conducted by varying the diameter of the granular columns, viz. 50 mm, 75 mm and 100 mm. The first series of tests were performed on virgin clay bed without any granular columns. A second series of tests were performed on ordinary columns of different diameters without any encasement (referred to as OGC). The third series of tests were performed on geosynthetic encased granular columns (referred to as EGC) with woven and nonwoven geotextiles and different diameters.

2.2.1 *Numerical Analyses*

The results from the laboratory tests of the present work were back-predicted through numerical simulations with relevant material properties used in the experiments. All the analyses in this investigation were performed using the finite element program 'GEOFEM' which was originally developed at the Royal Military College of Canada (Rajagopal and Bathurst, 1993) and subsequently modified at IIT Madras. In finite element models, the cylindrical unit cell was idealized as axi-symmetric case with radial symmetry around a vertical axis passing through the center of the granular column. As the problem can be modeled as axi-symmetric case, one half of a typical vertical section passing through the central vertical axis is considered for the analysis. This area is discretized using 8-node quadrilateral elements for all the components in the system as shown in Figure 8. In the current investigations the granular columns and the soft soil are modeled using hyperbolic non-linear elastic equation given by Duncan and Chang (1970), Equation 1.

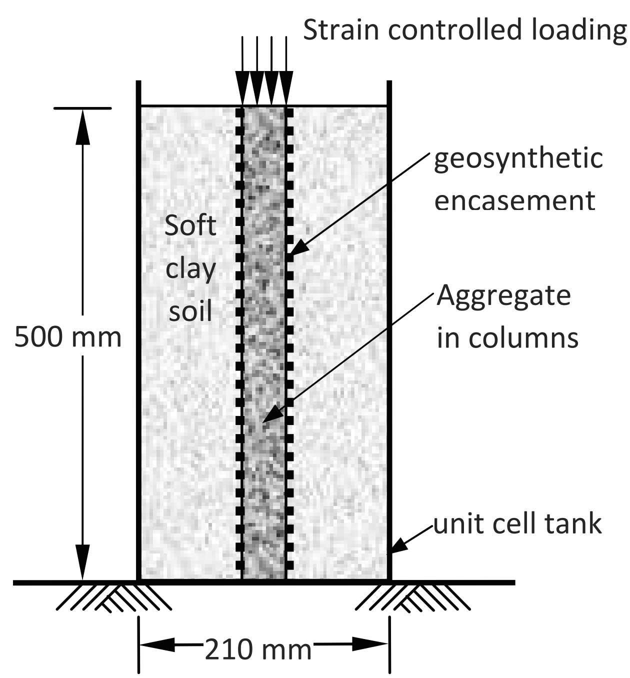


Fig. 7: Schematic of the Load tests on granular columns in a unit cell

$$E_t = \left[1 - \frac{R_f(1-\sin\phi)(\sigma_1-\sigma_3)}{2c\cos\phi + 2\sigma_3\sin\phi}\right]^2 K p_a \left(\frac{\sigma_3}{p_a}\right)^m \quad (1)$$

In which *K, m,* R_f*, c,* φ are the Young's modulus number, exponent, failure ratio, cohesion and angle of internal friction. The geosynthetic used for encasement of granular columns is assumed as linear elastic and modelled as a continuum element whose Young's modulus (*E*) was derived from the relation $J=E\times t$. Where J is the secant modulus of the geosynthetic and t is the thickness of the geosynthetic. The shear strength parameters were considered as that of clay soil and aggregates as reported in the details of laboratory experiments. The hyperbolic parameters considered are listed in Table 5. In order to reduce the number of parameters in the investigation, it is assumed that the contact between the different materials is perfect thus avoiding the need for interface elements. However, the elements immediately adjacent to the geosynthetic encasement are given lower shear strength values equal to 2/3rd of the strength of the parent material in order to allow the relative deformation between the encasement and adjacent materials. Further, the effects of granular column installation on the development and dissipation of the pore pressures are not considered in the analysis.

Table 5: Hyperbolic Material Properties used in the Numerical Simulation of the experiments

Materials	Hyperbolic model parameters						
	K	m	v	Rf	c' (kPa)	φ'	γ (kN/
Granular column	**250**	**0.7**	**0.3**	**0.7**	**0**	**41.5°**	**16**
Foundation	**15**	**0.5**	**0.45**	**0.7**	**2.5**	**0°**	**17**
Geosynthetic encasement	**Linear Elastic with Poisson's ratio, v = 0.3**						

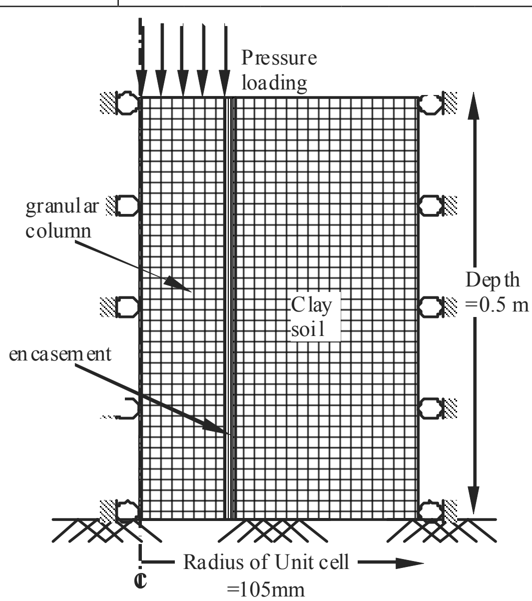


Fig. 8: Finite element mesh used for numerical simulations

2.2.2 *Results and Discussions*

Figure 9 shows the pressure settlement curve obtained from laboratory tests for the case of virgin clay bed, OGCs and EGCs encased with nonwoven geotextile, of three diameters, 50, 75 and 100 mm. The loading on clay bed and OGCs shows clear catastrophic failure.

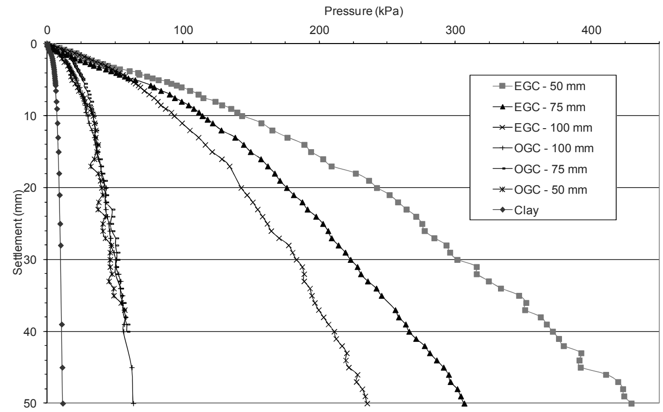


Fig. 9: Pressure-settlement response of OGC and EGC with nonwoven geotextile encasement

The EGCs have shown elastic behaviour up to very large pressures without any trace of limit pressure. The load carrying capacity of individual granular column at a settlement of 10 mm is increased by 3 to 5 folds because of encasement. The EGCs behaved like elastic semi-rigid flexible piles. In the case of EGCs the compression of the granular column was mainly due to the readjustment of the particle within the granular column and the elongation of the geosynthetic encasement. In the present study the failure was not observed even at a settlement of 50 mm (i.e. 10% of the column length). Figure 10 shows the load settlement response of the granular columns encased with woven geotextile for the three diameters (50 mm, 75 mm and 100 mm) of the column. EGCs with woven geotextile show stiffer response than that of EGCs with nonwoven geotextile. This is because of higher modulus of the geotextile. In both the cases, it is observed that the load capacities of OGCs are almost the same for all the diameters. Whereas for the EGCs it could be observed that as the diameter increases the load capacity of encased granular column decreases. The load capacity is found to depend very much on the diameter of the granular column.

The influence of the encasement modulus and the shear strength of the clay soil surrounding the granular column was investigated through finite element model studies. Typical results are illustrated in Figure 11. It could be observed that the pressure-settlement response of the ordinary granular columns is strongly dependent on the strength of the surrounding soil. On the other hand, the performance of EGCs was found to be relatively less affected by the strength of the surrounding soil. This is especially true for granular columns encased with a geosynthetic having modulus values more than 5000 kN/m.

From the different laboratory and numerical results presented here, it can be concluded that the elastic limit of granular columns can be increased by encasing the columns with stiffer geosynthetics. The load capacity and stiffness can be improved by suitably choosing a strong geosynthetic. While the ordinary granular columns may undergo sudden failure under the applied loads, the encased granular columns can undergo much larger deformations without exhibiting failure.

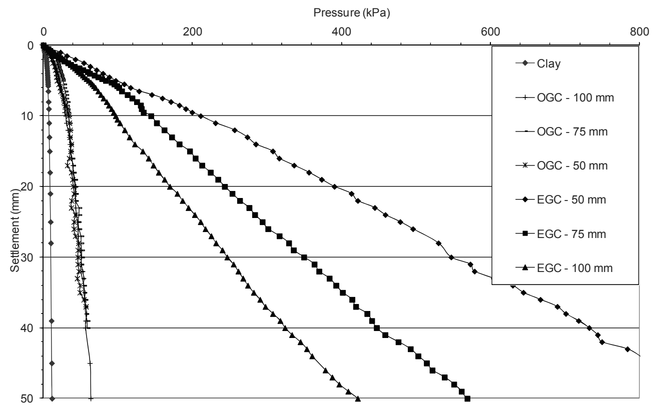


Fig. 10: Pressure-settlement response of OGC and EGC with woven geotextile encasement

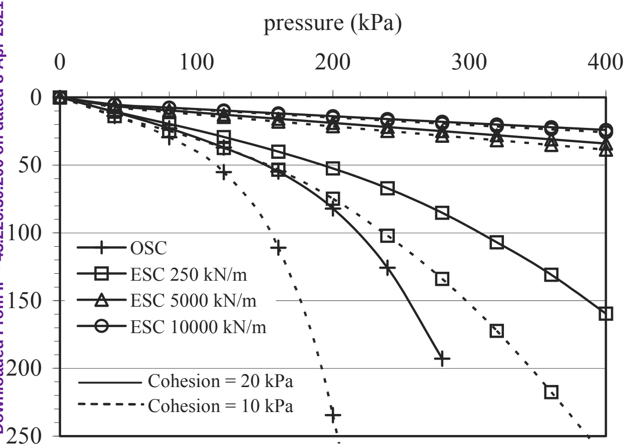


Fig. 11: Influence of shear strength of soil on the performance of OGC and EGCs

The reason for the improvement in the performance with the geosynthetic encasement could be understood by the increase in the confining pressures within the granular columns of EGCs, Figure 12. The confining pressure observed within the granular column with depth is shown in the figure for the Ordinary granular column (OSC) and for EGCs with different modulus values of the geosynthetic encasement. It could be observed that the confining pressure within the ordinary granular column increased within the top 2 diameters and later has fallen down to the level of K_o state. On the other hand, the confining pressures within the geosynthetic encased granular columns have shown significant increase compared to the OGC. The increase in the confining pressure with geosynthetic encasement was found to increase with its modulus value. The strength of the granular medium is directly proportional to the operating confining pressures in the medium. The improvement in the performance of the EGCs could be understood from the increase in the geosynthetic induced increased confining pressures.

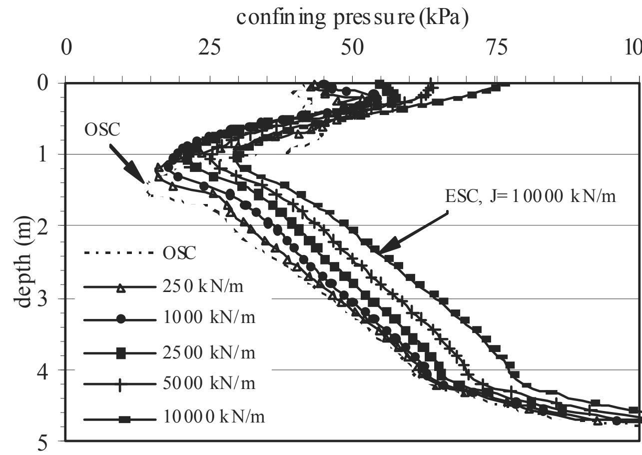


Fig. 12: The confining pressures within the granular column in OGC and EGC

Based on the results observed from the different numerical analyses and the laboratory tests, the performance of the load carrying capacity with different types of load bearing elements can be illustrated as follows. The load bearing capacity of the soft clay soils is the least and that of end bearing piles is the highest. The load bearing capacity of clay soil treated with ordinary granular columns is marginally higher than that of clay soils. The load capacity of the geosynthetic encased granular columns can be controlled by suitably choosing the geosynthetic material. As the geosynthetic modulus is increased, the load carrying capacity can approach that of end bearing reinforced concrete piles.

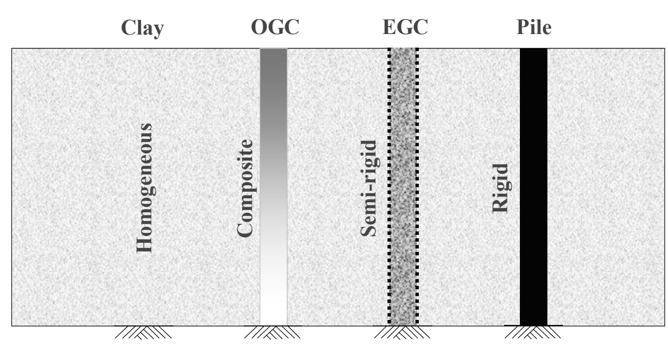


Fig. 13: Schematic illustration of different load bearing elements

3. 3-D SLOPE STABILITY ANALYSES

The above described laboratory and numerical results prove that the geosynthetic encasement helps in increasing the lateral and vertical load carrying capacity of granular columns. The actual behaviour of granular columns in the field is a combination of compression (including bulging), shear and bending. For bulging to occur realistically, a minimum amount of slenderness is necessary. The study of the performance of encased granular columns in real loading cases like under embankment loading will give more insight into their overall behaviour.

To simulate the behaviour of granular column in actual field condition (prototype scale), 3-dimensional (3-d) slope stability analyses are carried out using FLAC^{3D} software (Version 5.0). The FLAC^{3D} program is an explicit finite-difference program, which offers ideal analysis tool for solution of 3D problems in geotechnical engineering. FLAC program is better in analyzing the large strain response of soils compared to FEM program due to its Lagrangian formulation.

3.1 Problem Definition and Modeling Techniques

In this paper a three dimensional (3-d) finite difference method was used to calculate the Factor of Safety (FS) of embankment supported on granular column-improved soft clay. The model geometry with various dimensions is described in Figure 14. FLAC uses strength reduction technique to calculate FS by progressively reducing the shear strength of the materials (c and φ). To get more realistic results the granular columns were modeled as cylindrical elements instead of wall elements. Soft clay, embankment and granular column are modeled as elastic perfectly plastic materials using Mohr-Coulomb failure criteria. Liner elastic model was used to model the geosynthetic encasement. The properties of different materials used in the modeling are given in Table 6.

Figure 15 shows the maximum shear strain contours in case of EGC supported embankment. From the figure it can be seen that the slip surface in this case is neither continuous nor circular. Similar observation was also reported by Abusharar and Han (2011). Deformation mode entire structure (DF=3) is also shown in the figure. It can be observed that granular columns near the center line of the embankment undergo significant vertical compression. Near the toe of the embankment, no vertical compression is observed in the granular columns whereas columns are found to undergo significant lateral deformation. Significant amount of heaving is also observed near the toe of the embankment due to lateral flow of the soft foundation soil which exert lateral loading on granular column. Hence, the granular columns should be designed for both vertical and lateral loading, when installed bellow an embankment. Figure 16 shows the variation of FS with a_s. Different diameters of granular columns were considered for the analyses, starting from 0.8 m to 1.4 m which correspond to the a_s of 8.04% to 24.62% respectively. From the figure it can be observed that geosynthetic encasement help to mobilize higher FS compared to OGC, as it prevent the rupture failure of granular columns. Higher FS was mobilized for larger diameter granular columns as it could provide better resistance to the lateral soil movement.

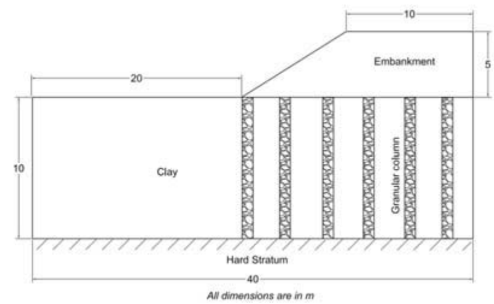


Fig. 14: Granular column supported embankment

Table 6: Properties of different materials used in the modeling

Parameters	Values
Granular column	
Bulk unit weight (kN/m^3)	19
Poisson's ratio	0.3
Friction angle (°)	38
Cohesion (kPa)	0
Soft clay	
Bulk unit weight (kN/m^3)	15
Poisson's ratio	0.45
Friction angle (°)	0
Cohesion (kPa)	10
Embankment fill	
Bulk unit weight (kN/m^3)	18
Poisson's ratio	0.3
Friction angle (°)	32
Cohesion (kPa)	0
Geosynthetics	
Thickness	1 mm
Poisson's ratio	0.33
Modulus (kN/m)	500

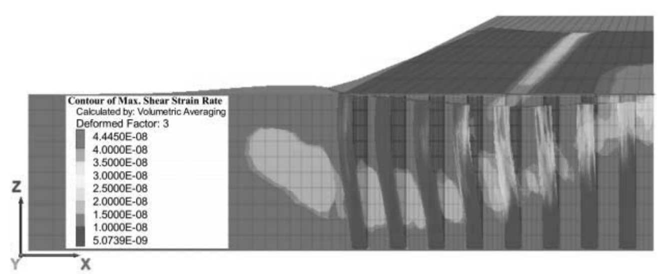


Fig. 15: Contour of maximum shear strain rate and deformation paten of EGC supported embankment

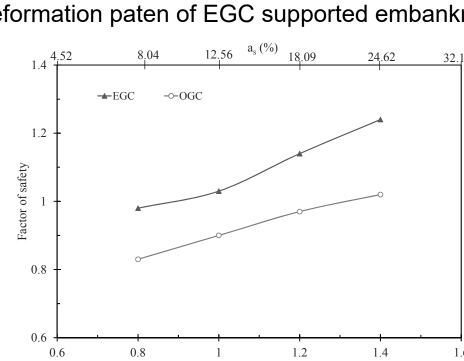


Fig. 16: Variation of FS with area replacement ratio (a_s)

It is observed that the a_s of granular columns can be reduced from about 25% with OGC to around 10% with suitable geosynthetic encasement while achieving similar performance, i.e. the factor of safety can be increased for the same a_s by using stiffer encasement materials, Table 7. The significant reduction of the natural aggregates for the construction of granular columns leads to more economical and faster constructions and leads to environmentally sustainable constructions.

Table 7: Variation of FS with a_s in case of OGC and EGC supported embankment

a_s (%)	FS		
	OGC	EGC	
		J = 500 kN/m	J = 2500 kN/m
8.04	0.83	0.98	1.29
24.62	1.02	1.24	1.30

4. CONCLUSIONS

Based on the above experimental and numerical studies following conclusions are drawn

(a) The geosynthetic encasement increases the vertical and lateral load carrying capacity and the stiffness of the granular columns.

(b) The stiffness of the granular columns can be increased by the increase in the modulus of the geosynthetic encasement. The settlements of the ground under the applied loads can be significantly reduced due to the increased stiffness of the granular columns.

(c) Rupture failure of OGC takes place along the predefined failure plane irrespective of the arrangement of granular column (single or group) and magnitude of normal pressure.

(d) While the OGC reinforced soil reaches limit state at some stage, the EGC reinforced soil exhibits strain hardening type behaviour due to continuous increase in confining stresses in both granular columns and the intervening soil.

(e) Group arrangement mobilizes higher shear resistance in the intervening soil between the columns due to the confinement effect. This effect is more prominent in case of EGC compared to OGC in triangular arrangement.

(f) Significant improvement in the factor of safety is observed by the geosynthetic encasement of the granular column, as both the strength and stiffness of the granular columns are increased by the encasement.

(g) Use of geosynthetic encasement helps in significant reduction of the natural aggregates for the construction of granular columns, which leads to more economical and faster constructions and leads to environmentally sustainable constructions.

5. ABBREVIATIONS AND NOTATIONS

DF	Deformation factor	c_c	coefficient of curvature
EGC	geosynthetic encased granular	c_u	coefficient of uniformity
FS	Factor of safety	s_u	undrained shear strength of clay soils
LDS	Large direct shear	s	center-to-center spacing of granular
OGC	ordinary granular column	J	secant modulus of geosynthetic encasement (kN/m)
3-d	three dimensional	φ_e	friction angle of embankment fill (°)
a_s	area replacement ratio	φ	friction angle of embankment fill (°)

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REFERENCES

ASTM D4595. Standard test method for tensile properties of geotextiles by wide–width strip method,

American Society for Testing and Materials. West Conshohocken, Pennsylvania, USA.

Alamgir, M., Miura, N., Poorooshasbh, H.B. and Madhya, M.R. (1996). Deformation analysis of soft ground reinforced by columnar inclusions. *Computers and Geotechnics*, 18(4): 261-290.

Adalier, K. and Elgamal, A. (2004). Mitigation of liquefaction and associated ground deformations by stone columns. *Engineering Geology*, 72, 275–291.

Abusharar, S. W. and Han, J. (2011). Two-dimensional deep-seated slope stability analysis of embankments over granular column-improved soft clay. *Engineering Geology*, 120, 103–110.

Ali, K., Shahu, J.T. and Sharma, K.G. (2014). Model tests on single and groups of stone columns with different geosynthetic reinforcement arrangement. *Geosynthetics International,* 21(2): 103-118.

Barksdale, R.D. and Bachus, R.C. (1983). Design and construction of granular columns. *Report No.*

FHWA/RD-83/026. Federal Highway Administration Office of Engineering and Highway Operations

Research and Development, Washington, D.C.

Chummar, A.V. (2000). Ground improvement using stone columns: problems encountered. *An International Conference on Geotechnical and Geological Engineering, GeoEng2000*, *Melbourne, Australia.*

Chen, X., Zhang, J. and Li, Z. (2014). Shear behaviour of geogrid-reinforced coarse-grained soil based on large-scale triaxial tests. *Geotextiles and Geomembranes,* 42(4): 312-328.

Deb, K. and Mohapatra. S, R. (2013). Analysis of stone column-supported geosynthetic-reinforced embankments. *Applied Mathematical Modeling*, 37(5), 2943–2960.

Dash, S.K. and Bora, M.C. (2013). Influence of geosynthetic encasement on the performance of stone columns floating in soft clay. *Canadian Geotechnical Journal,* 50(7): 754-765.

Elsawy, M.B.D. (2013). Behaviour of soft ground improved by conventional and geogrid-encased stone columns, based on FEM study. Geosynthetics International, 20(4): 276-285.

Fox, Z.P., 2011. Critical state, dilatancy and particle breakage of mine waste rock. Master's thesis.

Colorado State University, Fort Collins, USA.

Gniel, J. and Bouazza, A. (2009). Improvement of soft soils using geogrid encased granular columns.

Geotextiles and Geomembranes, 27(3): 167-175.

Gniel, J. and Bouazza, A. (2010). Construction of geogrid encased granular columns: A new proposal based on laboratory testing. *Geotextiles and Geomembranes,* 28(1): 108-118.

Ghazavi, M. and Afshar, J.N. (2013). Bearing capacity of geosynthetic encased stone columns. *Geotextiles and Geomembranes,* 38: 26-36.

Hughes, J.M.O., Withers, N.J. and Greenwood, D.A. (1975). A field trial of the reinforcing effect of a stone column in soil. Geotechnique, 25(1), 31-44.

Han, J. and Gabr, M.A. (2002). Numerical analysis of geosynthetic-reinforced and pile-supported earth platforms over soft soil. *ASCE Journal of Geotechnical and Geoenvironmental Engineering*, 128(1): 44-53.

Hosseinpour, I., Riccio, M. and Almeida, M. S.S. (2014). Numerical evolution of a granular column reinforced by geosynthetics using encasement and laminated disks. *Geotextiles and Geomembranes,* 42: 363-373.

Hosseinpour, I., Almeida, M. S.S. and Riccio, M. (2015). Full-scale load test and finite-element analysis of soft ground improved by geotextile-encased granular columns. *Geotextiles and Geomembranes,* 22(6): 428-438.

IS 15284-Part1 Indian Standard - Design and construction for ground improvement – Guidelines. Part – 1 Stone Columns.

Itasca (2005). Fast Lagrangian Analysis of Continua (FLAC3D 5.00). Itasca Consulting Group Inc, Minneapolis, USA.

Indraratna, B., Basack, S. and Rujikiatkamjorn, C. (2013). Numerical solution of stone column- improved soft soil considering arching, clogging and smear effects. *Journal of Geotechnical and Geoenvironmental Engineering*, 139(3): 377-394.

Khabbazian, M., Kaliakin, V.N. and Meehan, C.L. (2010). Numerical study of the effect of geosynthetic encasement on the behaviour of granular columns. *Geosynthetics International,* 17(3): 132-143.

Keykhosropur, L., Soroush A. and Imam, R. (2012). 3D Numerical analyses of geosynthetic encased stone columns. *Geotextiles and Geomembranes,* 35: 61–68.

Lo, S.R., Zhang R. and Mak, J. (2010). Geosynthetic-encased stone columns in soft clay: A numerical study. *Geotextiles and Geomembranes,* 28(3): 292–302.

Murugesan, S. (2007) Geosynthetic Encased Stone Columns as Ground Reinforcement of Soft Soils, thesis submitted in partial requirements for the award of the Ph.D.degree, IIT Madras, Chennai, India.

Murugesan, S. and Rajagopal K. (2006). Geosynthetic-encased granular columns: Numerical evaluation.

Geotextiles and Geomembranes, 24(6): 349-358.

Murugesan, S. and Rajagopal, K. (2007). Model tests on geosynthtic encased granular columns.

Geosynthetics International, 14(6): 346-354.

Murugesan, S. and Rajagopal, K. (2008). Shear load tests on granular columns with and without geosynthetic encasement. *Geotechnical Testing Journal*, 32(1): 35-44.

Murugesan, S. and Rajagopal, K. (2010). Studies on the behavior of single and group of geosynthetic encased granular columns. *Journal of Geotechnical and Geoenvironmental Engineering*, 136(1): 129-139.

Mohapatra, S.R., Rajagopal, K., and Sharma, J. S. (2014). Analysis of geotextile-reinforced stone columns subjected to lateral loading. in: 10th International Conference on Geosynthetics, Berlin, Germany.

Mohapatra, S.R., Rajagopal, K., and Sharma, J. S. (2016). Large direct shear load test on geosynthetic encased granular columns. *Geotextiles and Geomembranes,* 44(3), 396-405.

Mohapatra, S.R. and Rajagopal, K. (2016). Analysis of failure of geosynthetic encased stone column supported embankments, In: Proceedings of 3rd Pan-American Conference on Geosynthetics, April, Miami Beach, USA.

Pulko, B., Majes B. and Logar, J. (2011). Geosynthetic-encased stone columns: Analytical calculation model. *Geotextiles and Geomembranes,* 29(1): 29–39.

Raithel, M., Kempfert, H.G. and Kirchner, A. (2002). Geotextile-encased columns (GEC) for foundation of a dike on very soft soils. *Seventh International Conference on Geosynthetics, Nice, France*, 1025-1028.

Stoeber, J.N., (2012). Effects of maximum particle size and sample scaling on the mechanical behavior of mine waste rock; a critical state approach. Master's Thesis. Colorado State University, Fort Collins, USA.

Shahu, J.T. and Reddy, Y.R. (2014). Estimating long-term settlement of floating stone column groups. *Canadian Geotechnical Journal,* 51(7): 770-781.

Thorburn, S. (1975) Building structures supported by stabilized ground. *Geotechnique*, 25(1): 83-94. Yoo, C. and Kim, S.B. (2009). Numerical modeling of geosynthetic-encased granular column-reinforced ground. *Geosynthetics International,* 16(3): 116–126.

Zhang, Z., Han, J. and Ye, G. (2014). Numerical investigation on factors for deep-seated slope stability of stone column-*supported* embankments over soft clay. *Engineering Geology*, 168: 104-113.

STABILIZATION OF EARTHEN RIVER EMBANKMENT USING JUTE-SYNTHETIC HYBRID GEOTEXTILES – A CASE STUDY

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ABSTRACT

A multi-fibre approach has been adopted in conventional geotextiles to engineeraneco- sustainable product(s) possessing the improved geotechnical properties for enhancement of performance of earthen structure through soft revetments. Hybrid geotextiles comprising of juteand synthetic (polyolefin) were used together to minimize the disadvantages of each of the components by incorporation of their complementary advantageous properties. Hybrid geotextile was developed and compared with conventional 100% synthetic geotextiles. The geotextile contains at least 60% natural component having tensile strength10 kN/m and apparent opening size (AOS) of 180-200 micron. The above geotextile initially acts as a filtration layer and thereby assist in the formation of hard filter cake at the inner face of the geotextiles and after 6 months of degradation of jute, open structure synthetic part acts as durable reinforcing element for the entire soil structure and higher porosity of the cloth assists faster and better vegetation. Placing of seamless geotextile tube made of polypropylene tape filled with moist river sand as anchor trench - cum- toe guard with graded aggregates and back-filled of soil and sand, adds further reinforcement to the earthen structure. A cushion of soil to encourage vegetative growth, through turfing/grass sodding gave additional revetment of top soil geotextile and bottom soil by deep insertion and interweaving of grass roots. The woven fabric with apparent opening size (O95) of 180-200 micron successfully resisted the soil erosion thus helps in better confinement of the bund. Blended geotextile seliminates clogging off, and reduces the phase difference, thus are less slippery nature and provides better point to point contact for top soil and back soil. The developed geotextiles were tested through a large field trial for reconstruction of a part of the bank of river Mayurakshi at Birbhum, W B, India. The performance of the geotextiles is being monitored for last 10 years.

INTRODUCTION

Geotextiles of certain quality parameters, made from 100% polyolefin slit-film are presently being extensively used world-wide for solving different geotechnical problems (Pillips and Ghosh, 2003; Veldhulizen Van Zanten, 1986). Most of the 100% synthetic geotextiles are closed-mesh cloth or/and continuous sheet .The major advantage with the synthetic are reflected in its non-biodegradebilty which in turn provides it higher durability, high strength, very good processibility and it is easier to engineer for specific uses with cost competitiveness. Very low tensile modulus and photo-degradation are among the major disadvantageous properties of polyolefins. Steadily depleting resources of crude along with continuous increase in its price at a high rate are among the major concerns for the use of synthetics. The other problems of using all-synthetic geotextiles are its non-biodegradability and negative effects on biodiversity of the embankment soil due its closed structure. So, there has been considerable recent interest in the use of natural fibres for soil reinforcement at embankments and retaining walls (Lekha and Kavitha, 2006; Rawal. and Anandjiwala, 2007; Sarsby, 2007; Subaida et al 2008; Chauhan et al 2008) as well as its use as a separator for construction of paved and unpaved roads (Ahn, et al., 2002;Datta, 2007; Datye and Gore, 1994;Ghosh, et al., 2005;Mandal, 1987; Ranganathan, 1994; Sanyal and Chakraborty,1994; Shenbaga Kaniraj and Venkatappa Rao, 1994; Venkatappa Rao, 2000). In embankments, use of blended geotextiles will initially provide the filtration/separation and reinforcement function to the soil and by the time (at least two rainy season cycles) the jute is degraded and mixed with earth, the soil has been stabilized. Longevity of natural fibre under the soil depends upon the composition of the soil, pH, moisture content throughout the year and organic content. To increase the longevity, the jute geotextiles are bituminized to impart rot resistance property. However, possibility of leaching-out of bitumen (mostly composed of polycyclic aromatic hydrocarbons) is a threat to the environment by eventual contamination of nearby agricultural fields and river/pond water in the long run. So blending of synthetic fibrous material may increase the durability to a substantial extent of the jute-based geotextiles and at the same time the geotextiles may provide a harder layer (due to the presence of lignin) after degradation of untreated jute. Sustainable resources, (*viz*., jute, sisal, abaca, coconut-fibreetc) also have great role to play to mitigate the rapidly increasing global need of technical textiles, including geotextile. Coconut fibre having the highest lignin content in all lignocellulosicfibre is a much better option for application in geootextile due to it durability of about two years under soil. But the fibre is struggling for its own existence due to its processing limitations (mainly due to short fibre length, wide variation in fibre length, bigger diameter and rigidity) physical properties and lack of diversified products. Geotextiles made from hard fibres (*viz.,* coconut-fibre) sometimes fail to achieve some desirable property parameters for strength, porosity and drapability due to its coarseness, high flexural rigidity and low modulus. Major advantages of coconut fibre are its abundant availability, very slow biodegradability in contact with soil. However coconut fibre nonwoven, woven and knotted nets are quite popular now a days in slope stabilization.

The jute fibre has high tensile strength and modulus, good dimensional stability, anti-slip nature, high moisture absorption, and biodegradable. However, low extensibility, stiffness, long hairs protruded from the yarn surface, and fibre shedding are major drawbacks which restrict its processibility in subsequent high-speed machines. High biodegradability of jute also sometimes poses problems in some geotechnical uses. Along with this, its high moisture content sometimes reduce it durability itself during the storing and also makes it heavy and difficult during the laying. Further, as jute is an agricultural product, its productivity mostly depends on the vagaries of climate.

So in conjunction of jute, coconut fibre and/or other natural fibres with synthetic, may resolve that problem of both the component, to a notable extent without much hampering ecological balance. In addition, it is noteworthy that in most of the cases, a single fibrous element may not offer all the essential/desirable properties to a fabric for a specified end use. The disadvantageous properties can be minimized to some extent, if not eliminated, by incorporating certain percentage of other natural or man-made fibre(s).

The novelty of the present work was to combine the advantageous properties of jute, coconut-fibre and synthetics for location specific applications depending on boundary conditions. The prime objectives of the work were (i) preparation of standard geotextiles using natural fibres as the major element with synthetic materials having comparable or improved property performance suitable for construction of earthen river embankment, (ii) to lay down a range of specifications of jute-based geotextiles which can easily be produced in most economic wayindigeniously, and (iii) evaluating the performance of the developed geotextile by actual field trails. For this, a hybrid/composite-structured geotextile sample (jute-HDPE plain woven) were developed in a high-speed automatic circular weaving machine. Some important property parameters for geotextiles were evaluated at the laboratory as per standard methods to identify the fabric specification(s). The geotechnical property of developed geotextiles was also compared with plain-weave 100% synthetic geotextiles prepared from polyolefin slit-film. Finally, a field-trial was carried out using our developed geotextiles as separation-cum-reinforcing material for protecting rain-fed river embankment.

MATERIALS AND METHODS

Jute fibres (*Corcorusolitorius*) of TD 4 grade (BIS, 1987) (Bundle tenacity, 28-30 cN/tex) were used for preparation of jute yarns and nonwovens. 111 texHigh density polyethylene (HDPE) and 200 tex polypropylene (PP) flat-tape used for preparation of hybrid fabric sample.

Preparation of Jute Yarn and Fabric Samples

Jute yarn (Table 1) were prepared for this work in conventional jute spinning system using a non-mineral oil based spinning additive (Basu, et al 2008).Plain-weave fabric of different structural parameters was woven at a pick insertion rate of 480 pick/min in a fully automatic high-speed circular weaving (number of weft carrier/shuttle-4) machine supplied by Lohia Starlinger Ltd., Kanpur, India (LSL, 1989).

Fig. 1: Composite geotextiles

Evaluation of the Yarn and Fabric Property at Laboratory Stage

Environment for testing geotextiles was maintained at a relative humidity of 65+5% and a temperature of 27+ 2°C as per BIS (BIS, 1972) recommendation. The yarns are conditioned for 48 h prior to the tensile testing.

1. Tensile Properties

Tensile properties (*viz.,* breaking strength and strain, tearing strength) of yarn samples were carried out in Instron tensile tester (model 4411) at a gauge length of 500 mm at a strain rate of 300 mm/min (Table 1). Mechanical properties of geotextile samples were tested in a tensile testing machine for testing geotextile materials as per ASTM recommendation maintaining test parameters (a) Breaking strength and strain of fabric (specimen width, 200 mm; test length, 100 mm; loading speed, 10 mm/min); (b) Trapezoid tearing strength; (c) Index puncture strength in terms of puncture resistance was measured with 8 mm diameter probe at a speed of 320 mm/min and static puncture strength was measured using 50 mm diameter probe at a speed of 50 mm/min using a standard constant rate of extension type machine and (d) Perforation resistance test was carried out using cone drop a tester.

2. *Hydraulic Property*

(a) Water permeability of the geotextiles was measured by flowing plain water perpendicularly to the plane of a 500 mm diameter fabric for 160 s maintaining a 500 mm constant water-head pressure. (b) Filtration property, *i.e.*, apparent opening size(O_{95}) of the fabric samples was evaluated following ASTM recommendation using dry sieve shaking method (sieve shaker diameter, 200 mm; weight of glass beads, 50 g; time, 10 min; shaking speed, 150 cycle/min).

3. *Fabric Construction/Structure Related Parameters*

Area density of the woven fabrics was evaluated maintaining the size of the each specimen as 500 mm x 500 mm. The mesh size of the fabrics, *i.e.*, number of threads per dm in both warp (machine direction) and weft (cross direction) was evaluated by using a pick counter (BIS 1963- 1981). The thickness of the fabric was evaluated under a pressure foot of 55 mm diameter at a pressure of 2 kPa for 60 s. The structural parameters of the plain-weave fabricwas:

(a) Hybrid/composite structuredgeotextile having PP-tape in machine direction and double plied jute thread 39 x 44 end/dm, thickness – 1.5 mm, area density of fabric – 330 g/m^2, width of the cloth – 1.5 m.

Table 1: Tensile parameters of yarn and slit-film

Sl. No.	Material	Tape width / yarn diameter, mm	Linear density, tex	Tenacity, cN/tex	Extension at break, %	Load at 1% extension, N	Specific work of rupture, mJ/tex-m
1	PP – flat tape	2.5	108	37.43 (7.07)	22.94 (11.94)	5.09 (12.5)	54.85 (18.25)
4	100% jute yarn	1.2	650	10.89 (15.0)	2.31 (18.76)	12.39 (37.0)	1.18 (30.2)
Tensile testing was carried out maintaining test length at 500 mm at a speed of 300 m/min.							

Table 2: Physical Properties of Yarn for Preparation of Geotextiles

Sample	Targeted linear density, tex	Actual linear density, Tex	Diameter/ width, Mm	Breaking tenacity, cN/tex	Breaking strain, %	Load at 0.5% extn., N	Load at 1% extn., N	Sp.work of rupture (mJ/ tex-m)	Yarn twist, twist/ dm	Shrinkage due to plying, %	Jute content, %	Specific Flexural rigidity
Single jute yarn	344.5 tex	349 tex (3.31)	0.74 (13.91)	11.19 (20.13)	2.22 (13.15)	4.82 (21.20)	11.96 (15.86)	1.03 (30.98)	14.72 (7.99)	NA	100	60.88 (26.63)
2 ply of 349 tex jute yarn	689 tex	655 tex (3.69)	1.42 (5.79)	10.12 (33.11)	2.17 (21.18)	8.43 (30.00)	20.73 (22.48)	0.94 (42.93)	7.79 (5.73)	2	100	36.37 (9.39)
PP tape (slit film)	180	185 tex	2.5 (2.4)	42.37 (5.2)	25.22 (11.9)	2.76 (14.2)	5.09 (12.5)	66.92 (18.9)	NA	NA	0	307.223 (30.576)

Property parameters of developed hybrid/composite geotextiles

Table : Physical property parameters developed geotextiles

Type of geotextiles	Apparent opening size (O_{95}), micron	CBR puncture resistance (50 mm probe), kN	Index puncture resistance (8 mm probe), kN	Permittivity at 50 mm constant water head, Per sec	Cone drop test, mm	Tensile strength, kN/m		Breaking elongation at break, %		Seam strength, kN/m	Trapezoidal tear strength, kN/m	
						MD	CD	MD	CD		MD	CD
Jute-polypropylene hybrid plain woven geotextile	180	2.3	0.46	0.27	10.4	22.6	18.07	27	7	12	6.93	4.98
Polypropylene tape woven seamless tubular geotextiles	165					21.8	22.0	24.4	23.8			

FIELD TRIAL

Brief Description of the Location

The proposed area of work is along a part of the river-bank of Mayurakshiriver flowing through Shatpalsha in Mayreswar-II block in the Birbhum district, West Bengal. The proposed length of the embankment/dyke to be treated under this project is about 200 m.

Birbhum

Birbhum is an important district in the Rarh region having a considerable area under undulating topography in the western part of the district. Most of the rivers enter through the western side of the district and pass through the eastern direction. There are innumerable numbers of rivulets locally known as “kandor” spreading all over the district. Excepting the monsoon dry weather prevails through outthe years in this district with variation of temperature from 12.7˚C to 28.3˚C in the winter and from 25.5˚C to 39.4˚C in the summer. The normal rainfall is 1430.5 mm. The predominant soil types are old alluvial and red lateritic with low to medium in organic carbon & phosphate content and medium to high in potash. The soil is acidic in nature with pH range of 5.0 to 6.5.

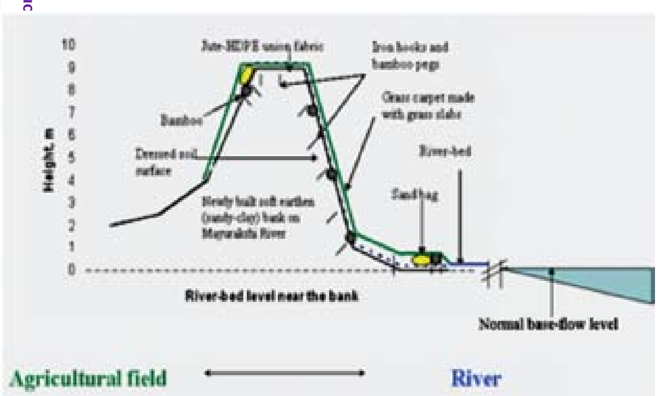


Fig. 2: Sectional diagram of the geotextile laid embankment

Fig. 3: Laying of geotextiles

Fig. 4: Sand filled Geotextile tube

Detail of Geotextile Tube

Calculation showing force imparted by the moist sand-filled geotextile seamless tube

1. The tube was filled with moist river sand and rested on the geotextiles for anchoring and also for guard wall.
2. The tube takes-up the shape at elliptical cross-section after sand filling.
3. Since, the tube do not have any side seam, chances of bursting of the tube is much less than stitched one.
4. Average length of the tubes was 40 m
5. It is flexible. So, impact of rolling water on it would be well distributed in multidirectional path. Ultimately, energy of river-side water current is dissipated/absorbed to a great extent.

Volume/linear meter = 3.14* 12cm * 70cm *100 cm/4 = 65940 cm^3
Bulk density of moist river sand = 2.0 g/cc
Weight of the 1m x 0.7 m geotextile tube = 137.34 kg

Mass of sand/linear meter of elliptical geotextile tube = 137.34 kg

A = 12 cm and B = 70 cm

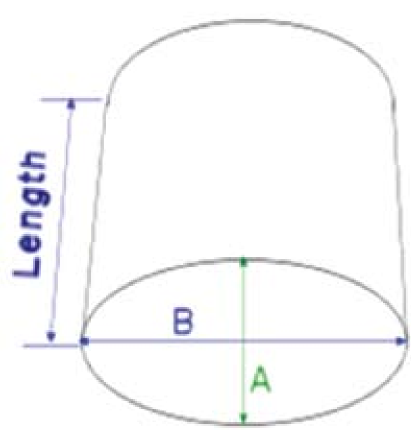


Fig. 5: Sketch of geotextile tube

1. Area of geotextile tube (when empty): 22/7 * radius (0.2386 m)^2 = 0.1789 m^2
2. Area of geotextile tube (Sand filled) : 22/7 * Minor radius (.06 m)*Major radius (.375 m) = 0.07 m^2
3. Extensibility %= 10%

Monitoring

- A spell of wide-spread rain started on 20th September, 2007 (due to deep depression created at Bay of Bengal and China Sea) and continued up to 26th night over entire Southern Bengal, Jharkhand and Bihar plateau.
- Rainfall at Mayureswar during this period (24th -26th September) was recorded as 290 mm.

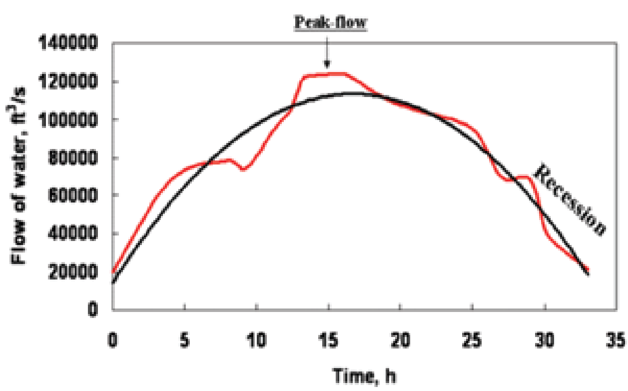


Fig. 6: Hydrograph of of water-flow from Tilpara barrage

Release of water from Tilpara barrage started on 25th morning at the rate of 60,000 cusec, increased upto 1,25,000 cusec at around 4'O clock. It was continued for 5-6 h, then slowly decreased to 60,000 cusec, it was continued for further 6-7 h, and then brought to the normal flow.

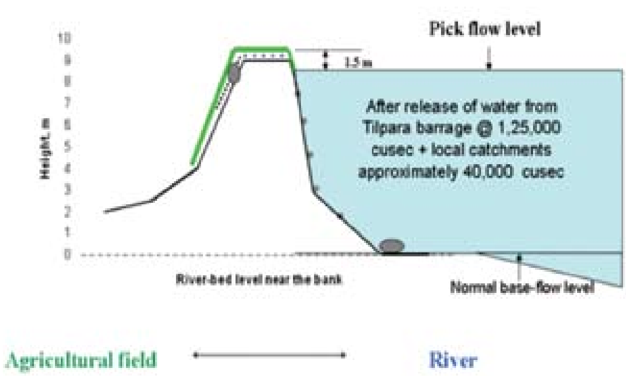


Fig. 7: Sketch showing level of raised water after release from barrage

Fig. 8: Condition of river-bank where geotextiles were not used

Fig. 9: Picture shows the comparative condition of the embankment after flood

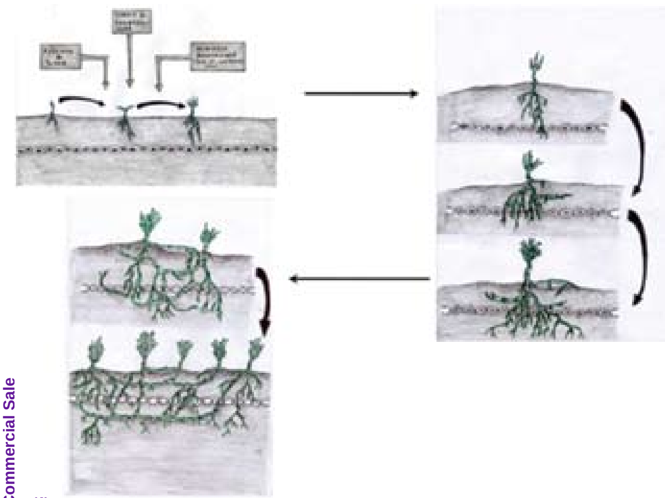
Fig. 10: Picture showing the condition of bank where all-PP geotextiles were used

Where all-PP geotextiles were used, torn-off spots from cattle hooves were observed; grass-sod slipped – down and vegetation was scattered and a very few.

Condition of River-bank After 2nd Year

No deformation/rain cut was observed; vegetation has been established; existence of worms is visible. Thickness of filter-cake formed - 7 cm (at the middle of the slant) in the 1st year and 15 cm in the 2nd year.

Principle of Reinforcement of Hybrid Geotextiles



CONCLUSION

Structure of fabric has been identified to be plain (1/1) woven of different construction parameters (*i.e.*, different mesh densities, linear densities of jute yarn/polyolefin tape) for making the hybrid geotextiles. Effort was made to prepare full-cover fabric. The laboratory test result, followed by the field trial of jute – HDPE woven fabric developed at NIRJAFT proved that the jute – synthetic (polyolefins) hybrid woven geotextile material could be the one of the most economic options for protection of earthen river bank. The developed geotextiles improved the serviceability of embankment. Evaluation of fabric samples were carried out as per ASTM/BIS recommendations and the test results found to be much satisfactory for using as geotextile in earthen embankment construction. The fabric can be woven at fully automatic high-speed weaving machine like, circular weaving machine as well as modern/conventional flat-bed weaving machine. Inclusion of synthetic results in much higher performance in productivity at weaving machine for manufacturing jute-based geotextiles as compared to all-jute geotextiles. The developed jute - polyolefin union fabric of specified yarn, tape and fabric parameters are suitable for using as separation layers, filtration medium and thereafter, as reinforcing materials. The developed poly-jute union fabrics are less costly than 100% jute geo-textiles, but little costlier and heavier than similar types of 100% synthetic fabrics, if 100% synthetic fabrics for geotextile-use is manufactured in India. Use of jute in making geotextiles, results in saving of valuable foreign exchange. In addition, presence of jute in the blended fabric imparts better filtration property which may not be achieved from 100% polyolefin tape fabric.

REFERENCES

Ahn, et al., 2002. Ahn, T. B., Cho, S. D. and Yang S. C., Stabilization of soil slope using geosynthetic mulching mat, *Geotextiles and Geomembranes*, **20** (2), (2002), pp 135-146.

ASTM, 2003 (a). Standard Terminology Relating to Textiles, D 123 – 03, Annual book of ASTM standards, Standards of D 13 Committee on Textiles, ASTM International, West Conshohocken, U. S. A., Vol. 07.01, (2003).

ASTM, 2003 (b). Annual book of ASTM standards, Standards of D 35 Committee on Geosynthetics, Construction, ASTM International, West Conshohocken, U. S. A., Vol. 4.13, Section 4, (2003).

Basu, et al., 2008.Basu, G., De, S. S., and Samanta A. K., Effect of bio-friendly conditioning agents on jute fibre spinning, *Industrial Crops Products*, DOI: 10.1016/j.indcrop.2008.05.008.

BIS, 1972. Indian Standard, IS:6359-1971, Method for Conditioning of Textiles (Bureau of Indian Standards, New Delhi, India), April (1972).

BIS, 1987. Grading of white, tossa and daisee uncut Indian jute. IS 271, Bureau of Indian Standards, New Delhi, India (1987).

Chauhan, M. S., Mittal, S. and Mohanty, B. 2008. Performance evaluation of silty sand subgrade reinforced with fly ash and fibre. *Geotextiles and Geomembranes, 26(5):429-435.*

Datta, 2007.Datta, U., Application of jute geotextiles, *J Natural Fibers*, 4 (3) (2007), pp. 67-82.

Datye and Gore, 1994.Datye, K.R. and Gore, V.N., Application of natural geotextiles and related products, *Geotextiles and Geomembranes*, 13 (6-7), (1994), pp 371-388.

Ghosh, et al., 2005. Ghosh, A., Ghosh, Ambarish, and Bera, A. K., Bearing capacity of square footing on pond ash reinforced with jute-geotextile, *Geotextiles and Geomembranes*, 23 (2), (2005), pp. 144-173.

Ghosh et al., 2005. Ghosh, G. K., Chattopadhyay G. N., and Chattopadhyay, S, Availability and forms of sulphur in red and lateritic soils of Birbhum District of West Bengal, *Indian J Agric Sc*, 75 (6), (2005), pp. 358-360.

JMDC, 2001. A manual on use of jute geotextiles in civil engineering, edited by N. Som, P. J. Rao and T. Sanyal, Jute Manufactures Development Council and Indian Jute Industries' Research Association, Kolkata, India (2001).

Lekha, K.R. and Kavitha, V. 2006. Coir geotextile reinforced clay dykes for drainage of low-lying areas, Geotextiles and Geomembranes, 24(1): 38-51

LSL, 1989. Poly-jute circular weaving machine, *Technical Leaflet*, LohiaStarlinger Ltd., Kanpur, India (1989).

Mandal, 1987.Mandal, J.N., Geotextiles in India, *Geotextiles and Geomembranes*, 6 (4), (1987), pp 253-274.

Mhaiskar, 2004.Mhaiskar, S. Y., Geosynthetics in pavement applications, In: *Geosynthetics-New Horizons*, edited by G. Venkatappa Rao, P. K. Banerjee, J. T. Shahu and G. V. Ramana, M/s Asian Books Pvt. Ltd., New Delhi (2004), pp. 173-182.

Phillips and Ghosh, 2003. Phillips, K. J. and Ghosh, T. K., The technology of polypropylene tape yarns: processing and applications, edited by P. W. Harrison, *Textile Progress*, 33 (1) (2003).

Ranganathan, 1994.Ranganathan, S. R., Development and potential of jute geotextiles, *Geotextiles and Geomembranes*, 13 (6-7), (1994), pp 421-433.

Rawal, A. and Anandjiwala, R. 2007. Comparative study between needlepunched nonwoven geotextile structures made from flax and polyester fibres, Geotextiles and Geomembranes, 25(1): 61-65

Sanyal and Chakraborty,1994. Sanyal, T. and Chakraborty, K., Application of a bitumen-coated jute geotextile in bank-protection works in the Hooghly estuary, *Geotextiles Geomembranes*, 13 (2) (1994), pp. 67-132.

Sarsby R.W. 2007 Use of 'Limited Life Geotextiles' (LLGs) for basal reinforcement of embankments built on soft clay. Geotextiles and Geomembranes, 25(4-5): 302-310

Shenbaga Kaniraj and Venkatappa Rao, 1994. ShenbagaKaniraj, R. and Venkatappa Rao, G., Trends in the use of geotextiles in India, *Geotextiles and Geomembranes*, 13 (6-7), (1994), pp 389-402.

Shome, 2006. Shome, T. K., Official Communication from Additional District Magitrate (Development), Demi Official Letter No. 1102/XXI -12/Dev, 12.06.2006, Government of West Bengal, India.

Subaida, E.A., Chandrakaran, S. and Sankar, N. 2008. Experimental investigations on tensile and pullout behaviour of woven coir geotextiles. *Geotextiles and Geomembranes, 26(5): 384-39.*

Veldhulizen Van Zanten, 1986. Veldhulizen Van Zanten, R., *Geotextiles and geomembranes in civil engineering,* A. A. Balkema, Roterdam, The Netherlands, (1986).

Venkatappa Rao, 2000. Venkatappa Rao, G., Sampath Kumar, J. P. Banerjee, P. K., Characterization of a braided strip drain with coir and jute yarns, *Geotextiles and Geomembranes*, 18 (6), (2000), pp 367-384.

Venkatappa Rao, and Pothal, 2006.Venkatappa Rao, G. and Pothal, G. K., Testing and evaluation of geotextiles, In: *Proc. Int. Seminar on Geosynthetics in India – Present and Future*, edited by G. Venkatappa Rao, G. N. Mathur and V. K. Kanjlia, Indian Chapter of International Geosynthetics Society And Central Board of Irrigation And Power, New Delhi, 8-10th November (2006), pp. 156-167.

COPPER SLAG POTENTIAL FOR USE AS BACKFILL / FILL MATERIAL IN CONVENTIONAL AND REINFORCED EARTH RETAINING WALLS

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ABSTRACT

Reinforced earth technique is being extensively used in construction of various civil engineering structures over the last two decades. The design and performance of reinforced soil structures depend mainly on the fill material interaction with the reinforcing material. Granular soils (frictional fills) are preferred in construction of reinforced soil structures, particularly in reinforced earth retaining walls and reinforced soil beds as they mobilise higher interfacial friction with reinforcing materials and the interfacial friction is not much affected in presence of moisture. Cost of conventional granular fill material namely sand and moorum has increased due to scarcity arising from continuous dependence. As the cost of reinforced earth retaining walls is largely affected by cost of fill material, efforts are being made to explore cost effective materials particularly non hazardous industrial wastes as fill material. Rock flour and Coal ashes have been tried to some extent in construction of reinforced earth walls based on the efforts of researchers. In this direction, the present work is aimed at exploring the potential of copper slag (a by-product from the matte smelting and refining of copper) for use as backfill /fill material in retaining structures. The previous studies on copper slag revealed that it is non hazardous and is a coarse grained material with good frictional characteristics, the suitability of the copper slag for use in conventional and reinforced earth (soil) retaining walls is assessed based on laboratory studies. Apart from evaluation of engineering properties of copper slag, it's interaction with woven geotextile fabric is evaluated from modified shear box and pullout tests. The results of the study indicated the potential of the copper slag for use as backfill material in conventional retaining walls and as fill in reinforced earth retaining walls. Copper slag has exhibited better interaction with woven geotextile as it mobilised angle of interfacial friction equal to its angle of internal friction as sand mobilises angle of interfacial friction of the order of 0.75 to 0.9 times the angle of internal friction.

Keywords: *Reinforced Earth, Retaining Wall, Frictional Fill, Interfacial friction angle, Copper slag, Woven geotextile, Pull out test, modified shear box test.*

INTRODUCTION

Reinforced earth retaining walls have emerged as effective alternative to conventional retaining walls in various civil engineering infrastructures due to faster construction, flexibility and low cost of construction. The construction of conventional retaining wall uses concrete and steel whereas reinforced earth retaining wall uses fill material, reinforcement and facing. The cost of reinforced earth retaining wall is governed by fill material as it is the major component. The success of the reinforced earth/ soil technology mainly depends on the interaction between the fill material and the reinforcing material used (Vidal, 1969; Sridharan and Hans Raj Singh, 1988; Koerner, 1990). The frictional (granular) fills, defined as good quality, well-graded, non-corrosive cohesionless materials with good frictional characteristics are preferred in reinforced soil constructions, particularly reinforced earth retaining walls. The gradation specifications for use of any soil/ material as frictional fill in reinforced earth constructions are that the percentage of fines shall be less than less than 10, uniformity coefficient shall be greater than greater than 5 and effective angle of internal friction shall not be less than 25°(Jones, 1985; Koerner, 1990).

Moorum (silty gravel) and river sand are the preferred backfill in conventional retaining walls and as fill material in construction of reinforced earth retaining walls as they possess good frictional characteristics and free drainage ability. Increased use of these materials has resulted in scarcity of these materials and increased the costs. To reduce the demand on large quantities of conventional fill materials and enable cost effective construction, alternative materials are to be identified for use as backfill and fill material in conventional and reinforced earth retaining walls.

The rapid rate of industrialization has generated large quantities of several wastes. Most of the wastes are not finding any effective use and thereby dumped at the sites resulting in environmental and ecological problems besides occupying large areas of valuable land. Efforts of researchers enabled some of the wastes such as rock flour, fly ash, pond ash, granulated blast furnace slag utilization in construction of highway embankments, fill material in reinforced soil structures, improved subgrades for pavements, basement filling, back filling behind retaining structures etc.

The interfacial friction angle of fill materials with reinforcing material is determined by various researchers by performing modified shear box or pull out tests. The investigations on soil-geofabric system indicate that the mobilized friction parameters of synthetic fabrics are influenced by surface texture, thickness of fabric as well as type, density, grain size, gradation characteristics and moisture content of fill materials (Makiuchi and Miyamori, 1988). It has been reported that the mobilised friction angle for the geofabrics ranges from 0.75 to 0.90 times the friction angle of sand. Studies on geotextile-sand friction evaluation (Venkatappa Rao and Pandey, 1988) revealed that the pullout tests yield higher values of interfacial friction angle than that obtained from modified shear box tests. Hence, the design of reinforced soil structures based on interfacial friction angles determined from modified shear box tests will be uneconomical. As pull out tests simulate field conditions in a better manner, usage of interfacial friction angles determined from pull out tests shall be encouraged. In the present study, geotechnical properties of the copper slag and its interaction with woven geotextile reinforcing fabric from modified shear box tests (Hussaini and Perry, 1978) and pull out tests are evaluated through laboratory investigations and suitability of Copper slag for use as fill/backfill in reinforcing structures is evaluated.

COPPER SLAG

Copper slag is a by-product obtained from the matte smelting and refining of copper and about 2.2-3.0 tons of copper slag is generated as waste for every ton of copper produced, (Maria Mavroulidou, 2017). The world copper production is currently about 20.47 million tons (International Copper Study Group, 2018). The copper slag generation in India is estimated to be about 1.63 million tons.

Fig. 1: Copper Slag

Based on the U.S. environmental protection agency regulations, copper slag is classified as non-harzardous material (Sreeramarao, Lavanya and Darga kumar, 2011). Hence, research studies are being done on investigating the use of copper slag as a cost effective material in various geotechnical engineering applications to derive potential environmental as well as economic benefits for related industries. Sterlite Industries, Hindalco and Hindustan copper are the major producers of copper in India. Das et al. (1982) conducted laboratory investigations on copper slag and reported it to be consisted of medium sand sized angular particles and good frictional characteristics ($\Phi = 40^0$ at relative density of 20% and $\Phi = 53^0$ at relative density of 100%). Further, it is reported that slag is non hazardous in terms of ground water pollution. The angle of interfacial friction of copper slag with geogrids from pull out test results is reported to vary from 35° in loose state to 49° in dense state (Prasad and Ramana, 2016).

EXPERIMENTAL WORK

Engineering Properties of Copper Slag

Copper slag used in the present work is collected from Sterlite Copper, Thoothukudi, Tamil Nadu. Extensive laboratory investigations are carried out to obtain the engineering properties of copper slag. As the copper slag is a coarse grained material, specific gravity is determined using Pycnometer. The grain size analysis is done as per IS 2720 (Part 4) – 1985 from sieve analysis and the gradation curve of copper slag is presented in Fig. 2. The compaction characteristics are determined from compaction curve plotted from I.S. heavy compaction test results as per IS 2720 (Part 8)-1980. The values of angle of internal friction of the copper slag are determined in OMC-MDD and saturated conditions. The test specimens are prepared at OMC and respective maximum dry unit weight corresponding to the I.S. heavy compaction conditions. The direct shear tests are conducted as per IS 2720 (Part 13)-1986. The failure envelopes of copper slag obtained from direct shear test are presented in Fig. 3. The soaked CBR value of the copper slag is evaluated by testing specimens prepared at OMC and respective MDD. CBR tests are performed as per IS: 2720 (Part 16)-1987. The coefficient of permeability is determined from constant head permeability test as per IS 2720-(part 17)-1986. The engineering properties of copper slag determined from the laboratory investigations are presented in Table 1.

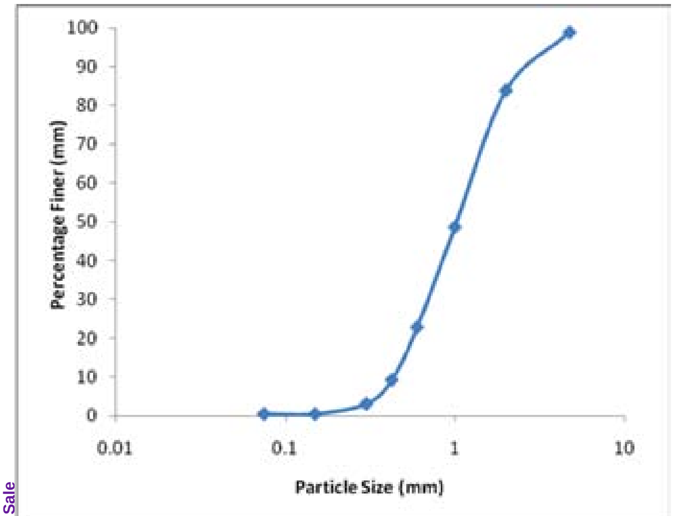

Fig. 2: Gradation curve of Copper Slag

Table 1: Engineering properties of copper slag

S. No	Engineering Property	Value
1.	Specific Gravity	3.34
2.	Grain Size Analysis	
	a. Gravel Size (%)	1
	b. Sand Size (%)	99
	i) Coarse sand size (%)	15
	ii) Medium sand size (%)	75
	iii) Fine sand (%)	9
	c. Fines (%)	0
	d. Coefficient of Uniformity	3.2
	e. Coefficient of Curvature	0.8
3.	Plasticity Characteristics	
	a. Liquid Limit (%)	NP
	b. Plastic Limit (%)	NP
4.	Equivalent IS Classification	SP
5.	Compaction Characteristics	
	a. Optimum Moisture Content (%)	7
	b. Maximum Dry Density (g/cc)	2.35
6.	Shear Strength Parameters	
	a. OMC - MDD Condition	
	(i) Cohesion (KN/m^2)	0
	(ii) Angle of Internal Friction (Φ)	41^0
	b. Soaked condition	
	(i) Cohesion (KN/m^2)	0
	(ii) Angle of Internal Friction (Φ)	38^0
7.	Coefficient of Permeability (m/s)	$2.8x10^{-5}$
8.	Soaked CBR (%)	10.2

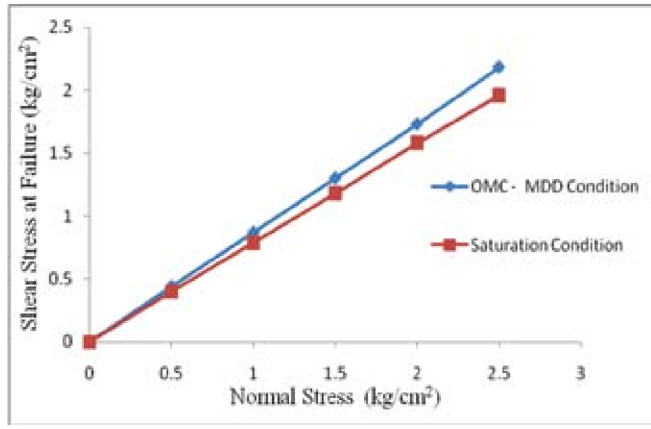

Fig. 3: Failure envelopes of copper slag from direct shear tests

SEM (Scanning Electron Microscope) Analysis is carried out at Advanced Analytical Laboratory, Andhra University and the results of analysis are presented in Fig. 4 Copper Slag particles are observed to be angular. The texture of the slag is observed to be rough in contrast to the relative flat texture of sand. X Ray Diffraction (XRD) analysis yielded the major constituents of copper slag as presented in Table 2.

Table 2: Major Constituents present in Copper slag under study

S. No	Element	Percentage by weight
1	Fe_2O_3	58.4
2	SiO_2	29.2
3	CaO	1.7
4	MgO	1.3
5	Zn	2.2
8	C_u	0.8

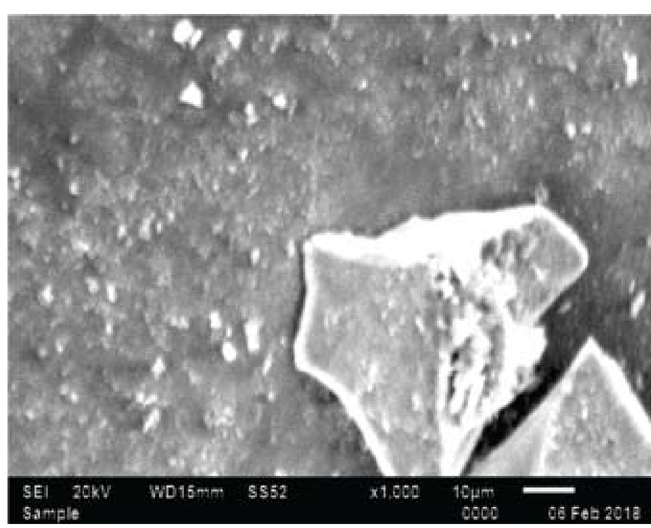

Fig. 4: Scanning Electron Micrograph of Copper Slag

Reinforcing Material

Polypropylene woven geotextile procured from M/s Garware Wall Ropes, Pune is used as reinforcing material with copper slag as fill material in the present study. The physical and mechanical properties of' geotextiles determined from laboratory tests are given in Table 3 and Table 4.

Table 3: Physical properties of woven geotextile

S.No	Property	Value
1.	Specific Gravity	1.10
2.	Mass per Unit Area (g/m^2)	240
3.	Thickness (mm)	0.61
4.	Apparent Opening Size (μ)	110

Table 4: Mechanical properties of woven geotextile

S. No	Property		Test method	Value
1.	Tensile Strength (KN/m)	Warp	ASTM D 4595	60
		Weft		58
2.	Puncture Strength (N)		ASTM D 4833	621

Evaluation of Interfacial Shear Parameters of Copper Slag with Woven Geotextile Modified Direct Shear Test

The test is performed in conventional shear box test by filling upper half with the copper slag material at OMC and MDD and placing woven geotextile on metallic block in lower half as per the procedure suggested by Hussaini and Perry (1978). During shearing, the slag slides along the reinforcement. The test is done in a way similar to conventional direct shear test by finding shear strengths mobilized under different applied normal stress. The failure envelope is plotted against the values of applied Normal stress Vs Shear strength mobilised at interface of geotextile and slag. The failure envelopes of copper slag with geotextile fabric in OMC-MDD state and saturated state are presented in Fig. 5 and the determined values of interfacial shear parameters are presented in Table 5.

Pull Out Test

Pull out tests are conducted in a modified direct shear box (Pull out box), which is assembled to a conventional direct shear test apparatus by sandwiching woven geotextile strip (size 12 cm x 4 cm) between copper slag compacted in OMC-MDD condition. By clamping one side of the reinforcement by a pair of jaws and sandwiching the other side between copper slag in the modified shear box, the reinforcement is subjected to pullout force. The pull out force is recorded by tension proving ring. The tests are also conducted on slag in saturated state. For fully saturated condition, the prepared test specimen at optimum moisture content and respective MDD in the modified shear box is saturated for one hour. The strength envelopes of copper slag with geotextile fabric (Fig. 6) are drawn by plotting normal stress on x axis and pull out resistance on y axis. From strength envelopes, interfacial shear parameters are determined and presented in Table 5. The friction coefficient is determined as tan ϕ_μ, where ϕ_μ is angle of interfacial friction angle of copper slag with geotextile fabric.

Table 5: Interfacial shear parameters of copper slag with geotextile fabric

Interfacial Shear Parameters	Modified direct shear test		Pull out test	
	OMC-MDD Condition	Saturated Condition	OMC-MDD Condition	Saturated Condition
Adhesion (kN/m^2)	0	0	0	0
Angle of Interfacial Friction	34°	32°	44°	41°
Friction coefficient	0.67	0.63	0.97	0.87

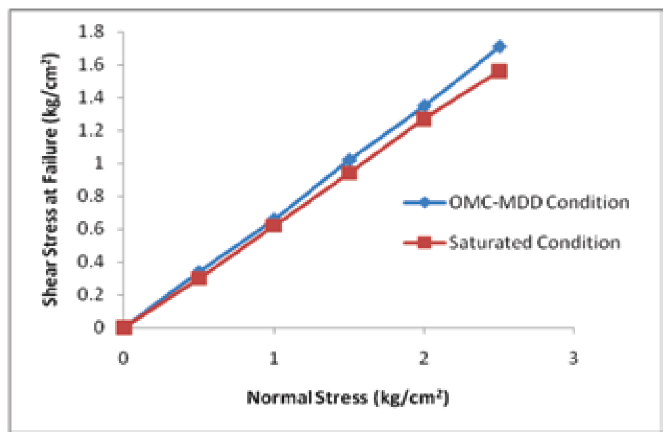

Fig. 5: Failure envelope of copper slag with woven geotextile from modified direct shear test.

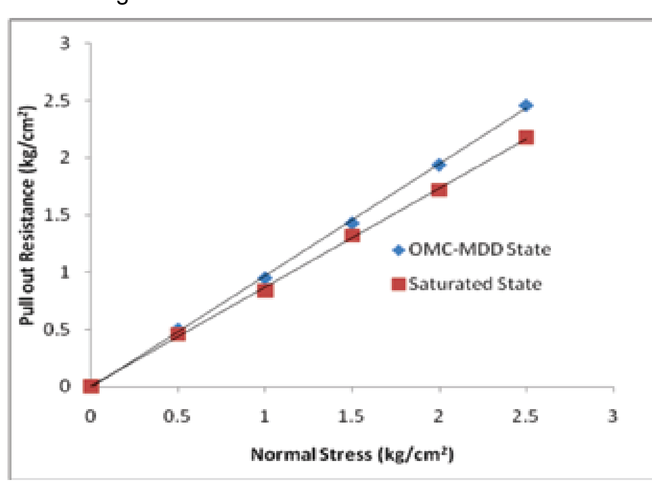

Fig. 6: Failure envelope of copper slag with woven geotextile from pull out test

DISCUSSION

Based on the gradation and plasticity characteristics, the equivalent ISSCS (Indian Standard Soil Classification System) classification group is observed to be poorly graded sand (SP). The copper slag exhibited good frictional characteristics both in OMC - MDD ($\Phi=41^0$) and wet conditions ($\Phi=38^0$). Copper slag has good drainage characteristics with coefficient of permeability of 2.8×10^{-5} cm/s. So, it can be concluded that copper slag is a coarse grained material with good frictional and drainage characteristics and is suitable for use as backfill material in retaining structures and basements of the buildings.

In comparison to conventional moorum backfill material ($\Phi=32^0$ and $\Upsilon = 21$ kN/m^3) behind retaining walls, the usage of copper slag as backfill results in reduced active thrust and At- rest thrust on retaining walls as detailed in Table 6 in spite of its higher unit weight due to its exceptionally good frictional characteristics. The active earth pressure coefficient (k_a) is determined from Rankine's theory (1857) and At –rest earth pressure coefficient (k_o) is determined using Jaky's equation (1944) based on angle of internal friction of material (ϕ) as detailed below.

$$k_a = \frac{1-\sin\phi}{1+\sin\phi} \quad \text{... (1)}$$

$$K_o = 1-\sin\phi \quad \text{... (2)}$$

Table 6: Lateral thrusts on retaining structures due to conventional (moorum) and copper slag backfills

Backfill Material	Earth Pressure Coefficient		Lateral Thrust (kN)		Reduction in thrust caused by Copper slag over conventional fill (%)	
	active state	at-rest state	active state	at-rest state	active state	at-rest state
Copper slag	0.208	0.344	$2.61H^2$	$4.315H^2$	19.8	12.6
Moorum	0.31	0.47	$3.255H^2$	$4.935H^2$	-	-

Where, "H" is height of backfill

Since copper slag has no fines and high friction angle ($\Phi=38^0$ in wet condition), its interaction with geotextile reinforcing fabric is evaluated through modified shear box and pull out tests in OMC and MDD condition and also in saturation condition. Copper slag exhibited higher values of interfacial friction angle with woven geotextile in pull out test compared to modified direct shear test. This is due to better butting of the slag material on geotextile surface due to rough textured angular particles. The pull out test based interfacial shear parameters shall be preferred as the test simulates field condition. However, the interfacial friction angle of copper slag with reinforcing material shall be limited to that of angle of internal friction of slag otherwise, shearing occurs in fill material itself. So, the interfacial friction angle of copper slag and woven geotextile under study shall be considered equal to the angle of internal friction of copper slag. Hence, copper slag is effective as fill material with geotextile fabrics in reinforced soil constructions compared to sand which mobilizes interfacial friction angle to the extent of 0.75 to 0.9 times angle of internal friction (Makiuchi and Miyamori, 1988).

CONCLUSIONS

Based on the results of tests conducted on copper slag and copper slag-woven geotextile fabric specimens under the study, the following conclusions are drawn.

1. Copper slag is a coarse grained poorly graded material with predominantly medium sand size particles with free draining nature. It has good frictional characteristics with angle of internal friction of 38^0 in wet condition
2. Copper slag is suitable for use as backfill material behind retaining structures as its usage reduces lateral thrust in active and at-rest conditions by about 20% and 12% respectively in comparison to conventional moorum backfill.
3. Copper slag can serve as better fill material compared to river sand as it mobilised higher friction angle with geotextile fabric under study.
4. Copper slag enables to adopt interfacial friction angle equal to its angle of internal friction with woven geotextile fabric and hence, it can be used as fill material in construction of reinforced soil structures, particularly reinforced earth retaining walls

REFERENCES

1. ASTM D 4595-2017: Standard test method for tensile properties of geotextiles by the wide-width strip method, ASTM International, West Conshohocken, PA, USA.
2. ASTM D 6241-14: Standard Test Method for Static Puncture Strength of Geotextiles and Geotextile-Related Products Using a 50-mm Probe, ASTM International, West Conshohocken, PA, USA.
3. ASTM D 6706 e-01, 2013: Standard test method for measuring geosynthetic pullout resistance, ASTM International, West Conshohocken, PA, USA.
4. Das BM, Tarquin AJ, and Jones AD. Geotechnical properties of copper slag, Transportation Research Record, Issue 941 (1982).

5. Hussaini A and Perry EB. Analysis of rubber membrane strip reinforced earth wall, Proceedings of Symposium on Soil Reinforcing and Stabilising Techniques, Sydney, pp. 59-72 (1978).

6. IS 2720 (part-III/Sec II) -1980: Methods of test for soils- specific gravity, Bureau of Indian Standards, New Delhi.

7. IS 2720 (Part 8)-1980: Determination of water content-dry density relation using heavy compaction, Bureau of Indian Standards, New Delhi.

8. IS 2720 (Part 4) – 1985: Methods of test for soils-grain size analysis, Bureau of Indian Standards, New Delhi.

9. IS: 2720 (Part 13)-1986: Methods of test for soils-Direct shear test, Bureau of Indian Standards, New Delhi.

10. IS 2720 (part 17)-1986: Methods of test for soils-Laboratory determination of permeability, Bureau of Indian Standards, New Delhi.

11. IS 13326 (Part 1)-1992: Evaluation of interface friction between geosynthetics and soil, Bureau of Indian Standards, New Delhi.

12. Jaky, J. The coefficient of earth pressure at rest, J. Soc. Hung. Eng. Arch. pp 355-358 (1944).

13. Jones CJFA. Earth reinforcement and soil structure, Advanced Series in Soil Engineering, London: Butterworths, 1985.

14. Koerner RM. Designing with geosynthetics, New York, Mc. Graw Hill publishers, 1990.

15. Makiuchi K and Miyamori T. Mobilisation of soil-geofabric interfacial friction, Proceedings of International Geotechnical Symposium on Theory and Practice of Earth Reinforcement, Fukuoka, Japan, 1988.

16. Maria Mavroulidou. Mechanical properties and durability of concrete with water cooled copper slag aggregate, Waste biomass valor 8, pp. 1841-1854 (2017).

17. Potyondy JG. Skin friction between various soils and construction materials, Geotechnique, Vol. II, No.4, pp. 339 – 353 (1961)

18. Prasad PS and Ramana GV. Feasibility study of copper slag as a structural fill in reinforced soil structures, Geotextiles and Geomembranes 44, pp. 623-640 (2016)

19. Rankine, W. On the stability of loose earth. Philosophical Transactions of the Royal Society of London, Vol. 147 (1857).

20. SreeramaraoA, Lavanya C, and Darga Kumar, A. A Review on utilization of copper slag in geotechnical applications, Proceedings of Indian Geotechnical Conference, Kochi, H-212 (2011)

21. Sridharan, A. and Hans Raj Singh. The effect of soil parameters on friction coefficient in reinforced earth, Indian Geotechnical Journal, Vol. 18, No.4, (1988).

22. Venkatappa Rao, G. and Pandey, SK. Evaluation of geotextile-soil friction, Indian Geotechnical Journal, Vol.I8, No.1, pp. 77-105 (1988).

23. Vidal, H. The principle of reinforced earth, Highway Research Record No. 282, pp. 1-16 (1969).