A Tryst with Geosynthetics

Annual Lecture

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Shahrokh graduated from Indian Institute of Technology, Bombay and continued there for his post-graduation. He is a Fellow, Indian Geotechnical Society and Fellow, Institution of Engineers (India), and Chartered Engineer. He has more than 49 years'' experience in geotechnical and civil engineering, of which the past ten years have been entirely devoted to the field of geosynthetics.

At Strata Geosystems, he has been instrumental in streamlining the designs of geosynthetics and has brought innovative applications of geogrids and geocells to the forefront. Such innovations include vertical containments for hazardous waste landfills, check dams constructed from geocells, use of mechanistic empirical methods for the design of flexible pavements reinforced with geocells and geogrids, and the use of two-layer theory for the analysis of load bearing systems incorporating geocells. Shahrokh's expertise in geosynthetics is well recognised in India and he is a member of prestigious committees for flexible pavements and bridges of the Indian Road Congress, a body for the formulation of specifications, guidelines, and codes of practice of the Ministry of Road Transport and Highways. Shahrokh is also Member of the TX30 Committee of the Bureau of Indian Standards. He has been appointed to the executive committee of the Initiative for Geotechnical Research and Innovative Practice (iGrip), an initiative of Indian Institute of Technology, Gandhinagar, for the advancement of geotechnology and geosynthetics for various engineering applications. He has also been appointed to the Board of Studies of the Mukesh Patel School of Technology Management and Engineering of the NMIMS Autonomous University.

Recognitions to Shahrokh include:

1. Life Time Achievement Award for 2015-16 of the IGS Delhi Chapter.
2. The Mancherji Edalji Joshi Memorial Trust "Outstanding Contribution Award 2017 for Excellence in Geosynthetics and Environmental Preservation".
3. Appreciation Award for contributions to the Indian Chapter of the International Geosynthetics Society.

Shahrokh has 45 publications to his credit.

43rd IGS Annual Lecture

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A Tryst with Geosynthetics

A: ABSTRACT

(Code A)

Geosynthetics are progressively playing a dominant role in geotechnical engineering and geotechnology. The engineering aspects of geosynthetics are a discipline unto themselves, based on the principles of Applied Mechanics. Through this Lecture Paper, the Author highlights his experiences with geosynthetics.

To date, much literature is available on various aspects and applications of geogrids and other geosynthetics. Not much can be reviewed on geocells. The Author has intensely worked on several applications with geogrids, geocells, and drainage systems, with nonwoven and woven geotextiles as indispensable requirements of detailing. While only innovative applications with geogrids and drainage geocomposites have been illustrated, the Author has majorly focussed on geocells.

Geocells were at one point of time, the least understood geosynthetic materials. Geocells are one example where technology preceded the understanding of its engineering mechanics. The Lecture Paper *inter alia* attempts to project theories defining the mechanics of geocells and analyses by simple methods that do not require sophisticated and expensive software. To highlight the practicality of the geocell systems, the Lecture Paper includes several case studies.

The objectives of the Lecture Paper include projecting diverse applications of geosynthetics, the need to consider any geotechnical aspect of a project holistically rather than in isolation, encourage out-of-the-box solutions and, most importantly, emphasise conservation of natural resources.

THE SECTIONS

For easy perusal, the Paper has been discussed section-wise as follows:

A: (Code: A)	Abstract
P: (Code: P)	Prologue
Section I: (Code: LF)	**Landfills - Containments with Vertical or Green Steep Side Slopes - Innovations at Donzi Ga, Vapi and Ghazipur**
Section II (Code: GC)	The Geocell and Its Basic Essentials
Section III (Code: MIGC)	**The Microlevel Mechanics of Load Bearing Geocells**
Section IV (Code: MAGC)	**The Macrolevel Mechanics of Load Bearing Geocells– Elastic Proposal**
Section V (Code: RD)	**The Road System**
Section V - A (Code: EM)	**The Embankment Component of Roads: Geogrids and Geocells as Basal Reinforcement**
Section V - B (Code: AP)	**Apparatus to Determine Shear Strength of Geocell Strap**
Section V - C (Code: PV)	**The Pavement Component of Roads**
Section V - D (Code: CNRD)	**Conclusions – Geosynthetics for Roads**
Section VI (Code: RW)	**Geosynthetics as Load Bearing Systems for Railways**
Section VII (Code: CY)	**Geosynthetics as Load Bearing Systems for Container Yards**
Section VIII (Code: WL)	**Geocells for Gravity Walls**
Section IX (Code: FS)	**Geocells as Fascia for Reinforced Soil Structures**
Section X (Code: CD)	**Geocells for Construction of Check Dam**
Section XI (Code: SL)	**Slope Erosion Protection**
Section XII (Code: RE)	**Reservoir Linings**
Section XIII (Code: DR)	**Geocomposite Drainages for Reinforced Soil Systems**
Epilogue (Code: E)	**Epilogue**
	Acknowledgements

P: PROLOGUE

(Code P)

GENESIS

Basically, the application of Geosynthetics boils down to:

1. Conservation of natural resources, and
2. Innovation

Ever since the existence of humankind, Earth has been exploited off its natural treasures. It started with the construction of habitats, using wood for fire, making tools for agriculture, and hunting for survival; forging weapons for territorial rights, greed, and ambitions; building of fortifications; transportation; and so, it went on and on all through Time; the Industrial Revolution; exploiting sources of forms of energy; and most tragically, major wars that have scant respect for the elements of Nature. Ancient Vedic and Avesta Scriptures revered Nature and extoled the virtues of the treasures of Earth *"Gêti"* with its soil, *"Zamo"* as one of the five vital elements. The Scriptures underscore the need to respect and preserve Nature's bounties – air, water, soil and rock, fire, and space. Yet humankind is the only species which never contributed to Nature, while systematically and exponentially exploiting it. Realisation of the folly has finally dawned upon us since the last quarter of the previous century. We marched into the new millennium burning our fingers, but much the wiser about the nuances of climate change. Indirectly, Geosynthetics is an offshoot of such realisation, at least as one of the objectives.

Soil reinforcement is not a new concept and it prevailed back into history. The earliest known structure is believed to be the terraced Hanging Garden of Babylon constructed around 600BC. The Romans constructed roads around 300BC using reeds as reinforcement for traversing weak subgrade soils. Sections of a wall along the Northern borders of ancient China were constructed around 200BC. Construction continued piecemeal into the 14^{th} to 17^{th} centuries during the Ming Dynasty. Bamboo strips were used as reinforcing elements. Thereafter globally, the reinforced soil concept was given a go-by till the last 30 years of the 20^{th} century.

During those ancient and baroque periods, vegetation such as reeds and bamboos, suitably seasoned were used as reinforcement. The Industrial Revolution heralded the age of quality grade steel, and the soil reinforcement of the 1970s was essentially steel. This was followed by the age of polymer plastics, oil derivatives way down in the crude refining hierarchy, and polymeric soil reinforcement has come to stay as "geosynthetics" with all its advantages over steel for this application.

By 1970s, the geotechnical fraternity of Europe launched into geosynthetics. Initially, the concept was more towards cost savings and space constraints, but the major advantage was the judicious use of natural resources.

GENESIS IN INDIA

In India at the outset, getting acceptance of soil reinforcement and geosynthetics has been an uphill task. But there were a few determined pioneers, and the Author pays tributes to these visionaries – academicians, manufacturers, even individuals – who persevered to eventually bring the geosynthetics technology and its engineering to the current level, despite odds. We made a modest beginning in the 80s with the introduction of woven and nonwoven geotextiles. As in the case of any new innovative development, there were several roadblocks which included:

1. Limitations of the characteristics of the available products in India.
2. Limitations of that time, when the repertoire of applications was cautiously restricted; that innovative spirit was lacking.
3. Basic inertia among geotechnical engineers to gain knowledge of the new technology.
4. A suspicious outlook by consultants and users towards the new technology; those who have a problem for any solution.
5. Lack of guidelines and standards from global statutory agencies.
6. The "I don't want to be a guinea pig" Syndrome, an unfortunate trait among owner-engineers.

The first bold exposure to the engineering fraternity at large was in 1985, with the first workshop on geotextiles and geomembranes conducted by the Central Board of Irrigation and Power (CBIP) at New Delhi. The CBIP forecasted and identified geosynthetics as an important area relevant to India's need for infrastructure development, including roads. This was followed by the First Indian Geosynthetics Conference (FIGC) in 1988 at IIT Bombay, under the auspices of the International Geosynthetics Society and the International Society for Soil Mechanics and Foundation Engineering with none other than Prof J P Giraud delivering the Keynote Lecture.

The International Geosynthetics Society (IGS) Council approved the formation of the Indian Chapter in October 1988. The India Chapter was registered under the Societies Registration Act 1860 in June 1992 with its Secretariat at CBIP. This was a landmark for propagating the knowledge of geosynthetics in India.

Earliest applications in India have been recorded in a publication entitled "Use of Geosynthetics in India – Experiences and Potential" brought out by the CBIP (Venkatappa Rao and Saxena, 1989). The earliest major initiatives where geosynthetics were used include (Vivek Kapadia, 2021):

1. Ukai-Kakrapar Canal Lining, Gujarat with Grouted Geo-mattresses (1983-1985).
2. Airport Runway, Ahmedabad, Gujarat (1988).
3. Salal Hydro-electric Project, Jammu and Kashmir, Reinforced Soil Systems (1990).
4. Loktak Hydro-electric Project, Manipur, Reinforced Soil Systems (1990).
5. Rammam Hydro-electric Project (Stage II), West Bengal, Reinforced Soil System (1991).

Geosynthetics, whatever be the type and style, as a class come under the domain of Technical Textiles. Other Technical Textiles had been well established for diverse applications such as air conditioning and climate control filters, air and oil filters, dust collection bags, vehicle heat insulation linings, sound insulation, medical applications such as surgical gowns and face masks, and defence applications such as parachutes, protective suits for nuclear, biological, and chemical warfare, and fire retardants. There were only three established manufacturers: Tata Mills Ltd (then in the private sector), Dinesh Mills, and Porritts & Spencers (Asia) Ltd. Yet geosynthetics had very few takers.

Despite all the reluctance to accept geosynthetics, there were academic stalwarts like Prof. M Madhav, then of IIT Kanpur and Prof G V Rao, then of IIT Delhi; individuals dealing in Technical Textiles for different applications, like Ms Aruna Lall and Mr Yogesh K Kusumgar to name two, who realised the potential of Technical Textiles in geotechnical engineering. These academicians and individuals were persuasive through workshops, conferences, and seminars. Notwithstanding a dearth of appropriate types of material of quality, they established the concept that Technical Textiles do have a major role tole to play in the civil and geotechnical engineering arena. It was Ms Lall along with the Author in 2001, who persuaded reluctant project leaders in a premier consultancy organisation to consider woven geotextiles for flooring and rand nonwovens for retaining structures for a paper warehouse in Kerala (Shahrokh Bagli et al, 2004).

There were pioneering Indian manufacturers with vision, who were broadminded enough to smelt the coffee and take the risk. Their venture into geosynthetics may as well be charted majorly as the progress of geosynthetics in India during the 21st century. Credit is also due to statutory bodies such as the Bureau of Indian Standards (BIS), India's Ministry of Road Transport and Highways (MoRTH) and the Indian Roads Congress (IRC) that realised the need of the hour and brought out several standards, guidelines, and codes of practice to rationalise the applications of geosynthetics in general civil engineering and highways practice.

INDIA'S SUCCESS STORY OF GEOSYNTHETICS

The Author is a member of the Family of Strata Geosystems (India) Pvt Ltd. Strata's history could well be the success story of major Indian geosynthetics players. The firm was established jointly with Strata Systems Inc, USA. Initially, geosynthetics (geogrids) were imported till a manufacturing unit for geogrids was set up in Daman in 2009. An innovative product was added, the geocell, in 2012. The geocell was honed to greater refinement of its characteristics and over the years, design methods have been evolved and adopted. Various aspects of the geocell are the major feature of this Lecture Paper.

Strata secured larger projects and proved that Indian geosynthetics can comfortably rival global contemporaries in quality. With several global orders on its platter, Strata became a major international player. Its geogrid sales touched 30 million m^2 in 2016. Considering the growing global demand for the Indian product, Strata set up the largest geosynthetics manufacturing unit in South Asia at Daheli, Gujarat in 2019. The Plant manufactures 1 lakh m^2 geogrids per day.

This is also India's success story of geosynthetics. Other contemporary Indian manufacturers also have similar success stories to tell in no small measure. In less than 20 years, from being a reluctant user and importer of geosynthetics, India is today an exporter of geosynthetic products of globally accepted quality. The credit for raising the bar of quality standards goes not only to the manufacturers but also to the BIS, MoRTH and IRC. A significant role has been played by various international accreditation agencies including India's NABL, and certainly the academicians from IITs, NITs and autonomous institutes that have been a constant source of guidance and inspiration; all in all, a joint, harmonious effort.

A TRYST WITH GEOSYNTHETICS

The Author has covered extensively from his trysts with geosynthetics. It was a casual curiosity to begin with in the 1980s, but this grew into an obsession over the last decade.

The Lecture Paper attempts to cover geosynthetics across the board holistically. However, there are constraints of length for the Paper and time limitations for a lecture (to be read between the lines as "attention span"!), hence the Lecture Paper essentially concentrates on experiences with geogrids and geocells with a smattering of drainage, no doubt essential in its own right.

But the *Prima Donna* of this Lecture Paper is the Geocell. Several aspects of geogrids have been extensively covered by numerous authors; but there is comparatively little on geocells. The Author well recalls that in 2012, the Geocell was least accepted. However, persuasion through successful case studies, field tests and trials in typical Indian conditions, as well as evolved analyses and design methods have finally won the day for the Geocell.

Geocells have proved their worth in diverse applications. This geosynthetic has been well accepted by BIS, MoRTH and IRC by bringing forth guidelines and rigid specification requirements. The Indian Geocell may be outpriced by certain global cheaper products of low standards. However, notwithstanding the higher price, the Indian product has scored over inferior quality since clients, both domestic and global, can be quite discerning over quality and are willing to pay that extra price for value addition.

In his tryst with geosynthetics, the Author has had enlightening interactions with four prominent personalities – Mr Narendra Dalmia and Mr Ashok Bhawnani, both Directors of Strata Geosystems, whose innovative thinking and foresight has come a long way in the advancement of geosynthetics in India; prof Manoj Datta of IIT Delhi has provided valuable opinions at the Vapi landfill and also his ideas and insights while designing the green verneer with geocells at Ghazipur landfill; Prof GV Rao, formally of IIT Delhi and now with IIT Gandhinagar; the Author was in close association with him on various IRC and BIS Committees, truly a patriarch of geosynthetics; and Prof K Rajagopal of IIT Madras, who is a friend, philosopher and guide to the Author, he is the sounding board on not only project related issues but also for this very Lecture Paper. My deep gratitude to these four tall, outstanding personalities and their valuable contributions to the advancement of geosynthetics in India.

And the Development Team of Strata, Gautam Dalmia, Suraj Vedpathak, Yashodeep Patil, Prashant Guda, Harsh Rajput and Pragya Mishra for putting in long hours of contributions to this Lecture Paper. The Author cannot forget the support from Strata Geosystem's Daheli Plant, Mr Chandrashekhar Kanade (CVK to us all) and his laboratory Team, particularly for their ideas and inputs for devise the Shear Test Apparatus for geocells.

SECTION I: LANDFILLS – CONTAINMENTS WITH VERTICAL OR GREEN STEEP SIDE SLOPES - INNOVATIONS AT DONZI GA, VAPI AND GHAZIPUR

(Code: LF)

PREAMBLE

Considering the significance of conservation of nature, the Author deems it prudent to start with the application of geosynthetics to landfills. While the scope of application to landfills is vast, for the sake of brevity, the Author has confined to only certain innovative applications of geosynthetics.

With massive scaling up of manufacturing and construction activities, as a developing nation, India is saddled with an enormous amount of waste. Development produces waste, be it village or a megapolis, a stand-alone plant or an industrial estate. Each produces its respective class of waste, ranging from municipal wet and dry wastes to normal or hazardous industrial wastes. Whatever, these wastes require careful handling and storage to prevent mingling with the enviro-system, being a hinderance to normal life, and damage the environment.

To facilitate the logistics and reduce enroute hazards, the landfills should be located as close as possible to the waste generator nodes, and yet far enough so as not to hamper normal activities. Hence such dumps are invariably on premium land.

The demand for storage space increases daily. Horizontal spread of landfills is expensive, and the only other option is a vertical expansion on the original footprint itself. To store additional industrial waste with minimal land space in the urban region of the industrial town of Vapi in Gujarat, a vertical landfill containment was designed and constructed for the first time in India using the design and construction philosophy of reinforced soil systems. This pioneering task in India of design and construction of the landfill came with a fair share of challenges and unpredictable encounters.

The idea of reinforced soil containment for landfills germinated from the landfill at Donzi, Atlanta in Georgia US. It was here that necessity invoked innovation by Strata US. The main issue was lack of space that necessitated vertical expansion of the landfill. The success of this idea in the US was a cue for Strata India to hone this innovation further, using its geogrids and its unique modular and segmental concrete block fascia.

This Section is confined to the innovations on landfill containments and not landfill environmental statutory matters and features. The features for containments include applications of reinforced soil technology and an out-of-the-box concept for greening steep embankment slopes shrouded with untextured geomembrane. Greening steep containment embankment slopes has been innovatively resolved at Vapi and Ghazipur in the Delhi National Capital Region, cases highlighted in this Section.

REINFORCED SOIL SYSTEM

Reinforced soil structures are commonplace today. In India, these structures have essentially been used for roads and highways, generally as grade separators and approaches to underpasses and bridges. Essentially two reinforced soil structures are constructed with their rear several metres apart depending on width required at the top and infilled with unreinforced compacted material in-between. Unlike the traditional earth embankment with a trapezoidal cross-section, the structure is slim, occupying a land footprint only to the extent of its utility requirement, with very steep side-slopes. And compared to a reinforced concrete or structural steel system, its cost is just a mere fraction in comparison.

Considering the advantages of a reinforced soil system, when land is at a premium, it is prudent to have a containment structure for a landfill comprising of a reinforced soil system with a narrow footprint rather than a conventional trapezoidal earth embankment with a wide footprint, not utilised for landfill material storage.

DONZI, ATLANTA GEORGIA IN USA

There were several concerns at Donzi that guided the decision towards a reinforced soil containment system. The population growth and development in and around Atlanta put pressures on the existing landfill, requiring an increase in the capacity of the landfill. However, the landfill was laterally confined to its existing battery limits and no additional land could be made available for expansion. While vertical expansion seemed to be the only option, there was no space available even to construct an appropriate containment embankment in accordance with traditional designs. Vertical expansion was possible only if the slopes were steep. Another factor that required the attention of the designer was the constraints of high-tension power transmission lines traversing across the landfill. These lines posed a danger to rear tilting dumpers which could come within the hazardous arcing distance of the high-tension power lines while emptying their loads.

Considering the dire requirement of increased capacity, aesthetics was not considered as a governing aspect. A reinforced soil slope was provided (StrataSlope™), comprising of geogrid-reinforced steep slopes (1V: 0.5H). Fascia of stone filled steel wire-baskets were adopted.

Fig. LF-1(a) shows the concept of the expansion before the filling would commence. Fig. LF-1(b) shows the landfill profile conceptually after full height filling has been attained. It may be noted that the old fill was not provided with any containment but was graded to slope as seen in Fig LF-1(a).

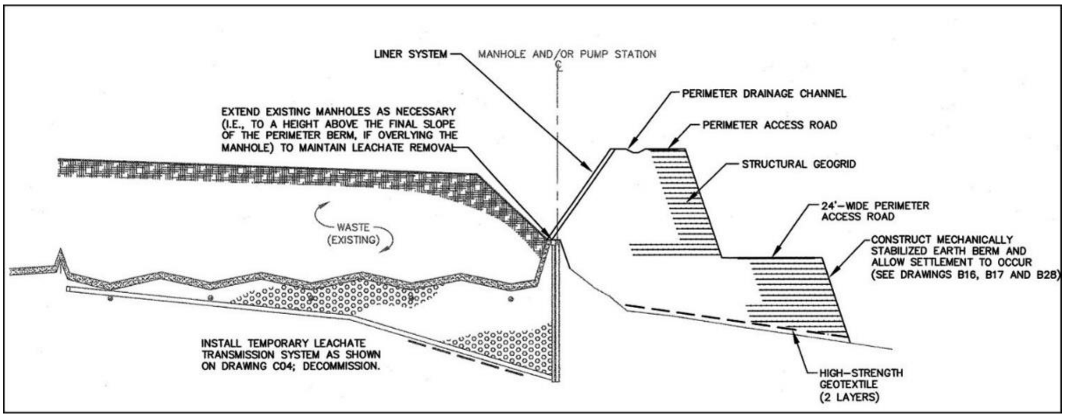


Fig. LF-1(a): Schematic of expansion prior to fill

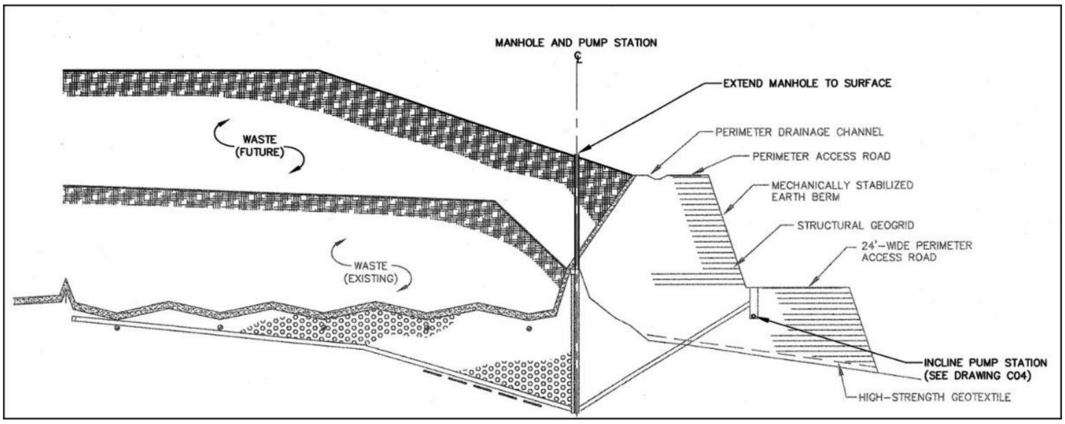


Fig. LF-1(b): Schematic of expansion after full fill

Fig. LF-1: Concept of vertical expansion of Donzi Landfill

The initial landfill dump was sloped and given the appropriate environment treatment and backed up with well compacted soil stabilised by a reinforced soil slope of 1V: 0.3H. The extension is an earth embankment with an inside slope of 1V: 1H and is supported on the outer side by a reinforced soil slope with wire basket fascia defining a slope of 1V: 0.5H. A road berm has been provided along the outer face of the reinforced soil slope to access the crest.

The finished height of the containment structure is 40’ (12.2m).

The maximum length of the geogrid is 28’ (8.5m). The vertical spacing of the geogrids is 460mm over the initial height of 1800mm, and thereafter the spacing has been increased to 900mm.

The high-tension transmission towers and catenaries were relocated appropriately. The maximum permitted height for the filling below the power line catenary was marked on the towers. The height of the marking was based on the safe distance between the highest point of a standard unloading tipper of a dumper and the lowest point on the catenary to safely prevent arcing.

Fig LF-2 illustrates the progressive construction. The old landfill is seen behind the construction of the reinforced soil slope that would contain the additional material.

Fig. LF-2(a) The lower-level stone filled wire basket fascia

Fig. LF-2(b) Wire basket fascia in collapsed form

Fig. LF-2© Progressive construction of reinforced soil slope with wire basket fascia

Fig. LF-2(d) Progressive construction of reinforced soil slope

Fig. LF-2© Completed reinforced soil slope

Fig. LF-2 Progressive construction of reinforced soil slope with wire basket fascia

The vertical extension of Donzi Landfill was the inspiration behind the vertical containment structure of the landfill at Vapi.

LANDFILL EXPANSION AT VAPI, GUJARAT FOR VAPI GREEN ENVIRO LIMITED

The three Cells of the landfill of Vapi Green Enviro Ltd had almost reached their capacities and required immediate expansion to cater to waste generated over yet more months. Adequate space for expansion within the existing battery limits was difficult if conventional trapezoidal embankments were constructed as containment. The footprint of such embankments would occupy valuable area leaving inadequate space to accommodate the anticipated additional volumes of waste. A reinforced soil system option was the only solution.

The Three Facets of the Project

Ultimately, the project was a harmony of three outstanding facets:

a) Land space optimisation despite a steep inner slope:

The landfill was built 14m high from the ground level. The containment was a reinforced soil structure with precast concrete modular block fascia (StrataBlock™). The uniqueness was that the inner reinforced soil slope had a steep angle of 70⁰. PET uniaxial geogrids (StrataGrid™) were wrapped around soil bags and stretched to and connected with the precast concrete modular blocks that formed the outer fascia. A composite lining of non-woven geotextile and geomembrane was provided along the slope, which would be in contact with the landfill material.

b) The reinforced soil wall on the original earth embankment containment structure:

For a portion of the landfill extension, it was necessary to vertically extend atop the original embankment containments. This required astute intricate designing for strengthening the original earthen embankment with soil nails.

c) Greening of the Existing Geomembrane Covered Steep Slopes:

The outer slopes of the embankments containing the old landfill cells are covered with untextured geomembrane shroud. The slope is as steep as 1V:1.5H. It was required to vegetate the slope. To vegetate a steep and smooth slope required an out-of-the-box solution. Geocells were considered along with geogrids as a support system, a creative method since the geomembrane could not be spiked to support the geocells along the slope. A unique solution was also proposed for anchoring at the crest since a conventional anchor trench was not feasible owing to the geomembrane cover at the crest of the closed Cell.

Project Overview

This Project was taken up turnkey by Strata Geosystems, which designed, supplied all geosynthetic material, and played the role of general contractor along with construction supervision. The client is in the business of storing industrial wastes from the nearby Vapi industries. The landfill is located within the urban industrial belt of Vapi, an industrial town in South Gujarat, west on the Indian peninsula, about 120km from Mumbai (Bombay). The client required more volume for storage at its present landfill site to cater to waste accumulating for at least two additional years. The additional capacity that could be generated by conventional containment was 150,000m^3 only whereas the required increase in capacity was 260,000m^3. The landfill is in a congested zone where there was no scope for adequate horizontal expansion. Fig. LF-4 shows the location of the landfill within the town, and a close-up, to underscore that the final completed expansion leaves no more scope for any more areal expansion. The Client had very limited space for expansion and a conventional solution did not provide adequate capacity to last two more years.

Fig. LF-4: The landfill in congested industrial zone within Vapi town

Falling back on its R&D team and the Donzi experience, Strata Geosystems devised a containment system with the reinforced soil concept. The lean structure would generate more space for waste, the vertical expansion making it possible without the need for a voluminous earth retaining structure.

The landfill is divided into cells, and each cell is filled one at a time. Fig.LF-5 shows the cells after completion of the reinforced soil vertical containment wall for Cell 4. The first three cells were closed and covered according to statutory requirements, shown in Fig. LF-5 as “Existing Cells”. Only Cell 4 could be developed for accommodating the additional volume.

Fig. LF-5: Drone shoot showing the division of landfill for stage-wise development; Cell 4 optimised by reinforced soil containment structure

The essence of this innovative technique was simplicity in design and construction, utilising geotechnical and geosynthetic solutions to their fullest extent. There were various facets and challenges for which geosynthetics have been innovatively used, highlights of which are discussed below.

Land Space Optimisation

The expansion portion of the Project began with Cell 4 – Part 2, away from the earlier closed cells. The design and construction of this portion was a normal reinforced soil structure. The landfill expansion is over a horizontal spread, and up to a height of 14m above the ground (Fig. LF6). With the reinforced soil concept using StrataGrid™ PET geogrids, a slim vertical containment was constructed.

The inner face of the wall was a wraparound steep slope and the outer facia comprised of precast concrete modular segmental StrataBlock™. Thus, the landfill could be extended vertically to contain more waste than was previously envisaged. The containment slimmer than the conventional trapezoidal section greatly contributed towards better land use for waste storage rather than mere containment bulk.

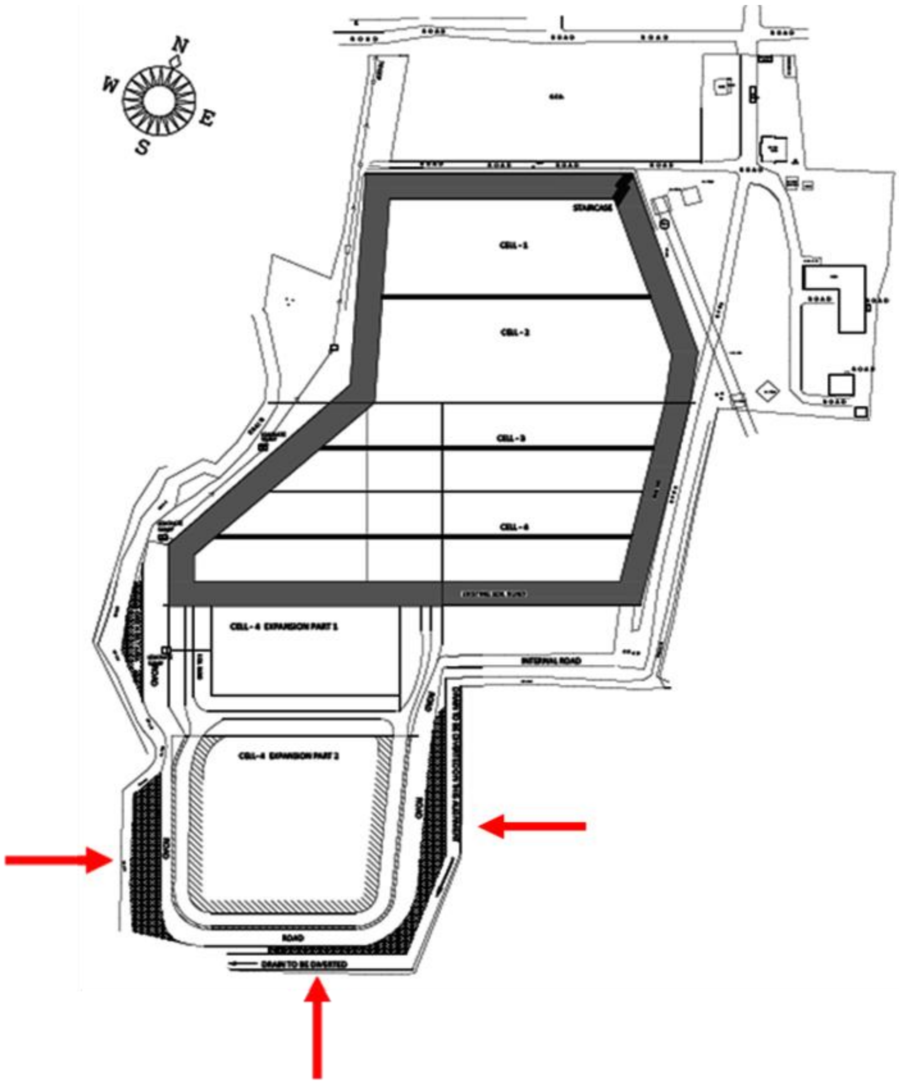


Fig. LF-6: The reinforced soil wall section resting directly on the ground, marked by arrows

The reinforced soil containment comprised of two reinforced soil systems:

1. The inner slope that abuts the landfill material was a rudimentary form of fascia with geogrids wrapped around soil bags (Fig. LF-7(a)). This face was draped with a composite of non-woven-geomembrane-non-woven sandwich to prevent landfill leachate from seeping into the containment (Fig. LF-7(b)). The uniqueness of this face is that the slope was at a steep angle of 70^0 virtually falling under the classification of a wall. Two berms were provided (Fig. LF-10) to break the slope at every 5m vertical height, also to provide anchorage for the non-woven-geomembrane-non-woven composite. Judicious detailing was very essential to provide for a leak-proof system that would prevent seepage of leachate out of the landfill system.

Fig. LF-7(a) The geogrid wraps for the inner face

Fig. LF-7(b): Shroud of nonwoven-geomembrane-nonwoven sandwich laid over the inner slope

2. The outer facia of segmental precast concrete modular blocks, which lent an imposing appeal to the structure (Fig. LF-8).

Fig. LF-8: The outer fascia of segmental precast concrete modular blocks

There were challenges that arose during the excavation for the foundation. Landfill waste was observed to almost 3m depth at some locations, forcing the Designers back to the drawing board for a rework, to consider replacing the organic fill with engineered backfill. The enormity of the problem faced can be seen in Fig. LF-9, which shows a typical pit displaying landfill material.

Fig. LF-9: Pit excavated for reinforced soil wall foundation at Cell 4 – Part 2, exhibiting landfill material

The cross section of the reinforced soil wall is shown in Fig. LF-10. The top width is 8.270m whereas the width at founding level, 1m below ground level, is 16.3m. Non-plastic soil (SP) as material for reinforced soil was used. Since the design and detailing required the reinforcing geogrids through and through the cross section, the non-plastic soil was placed over the entire section. No zoning was required, except for the gravel drainage bay behind the precast modular concrete blocks (Fig. LF-11).

The consolidated drained strength parameter φ_{CD} of the non-plastic SP soil was initially considered as 34^{o}. However, in view of the vagaries of availability of the material from a single location and with the prospect of having to utilise material from several discreetly located borrow areas, it was decided to judiciously use a conservative φ_{CD} of 30^{o}.

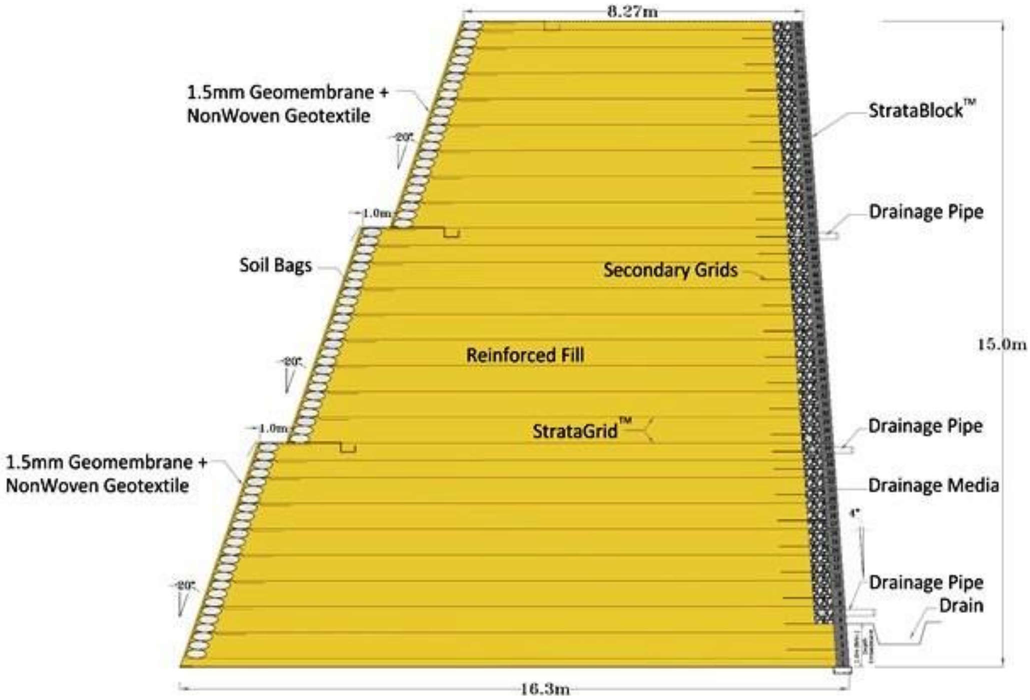


Fig. LF-10: Cross-section of the reinforced soil containment wall with precast modular concrete blocks as outer fascia and geogrid wrap as inner slope fascia

Fig. LF-11: The gravel drainage bay and the non-plastic soil

Reinforced Soil Wall on Earth Embankment

Referring to the drone shoot in Fig. LF-5, Cell 4 – Part 1 includes a portion of the side slopes of the containment embankments of the covered and closed Cell 3, both to the East and West of the landfill, as seen in Fig. LF-12. Considering the higher top level of Cell 4 as compared to that of the cover top of Cell 3, this required vertical expansion of the new containment system on the existing side slopes and up to a height of 14m above the ground level. Construction of a reinforced soil wall on the side slopes of an existing earth embankment was an astute task; it required intricate engineering, and spot judgement and decisions during construction.

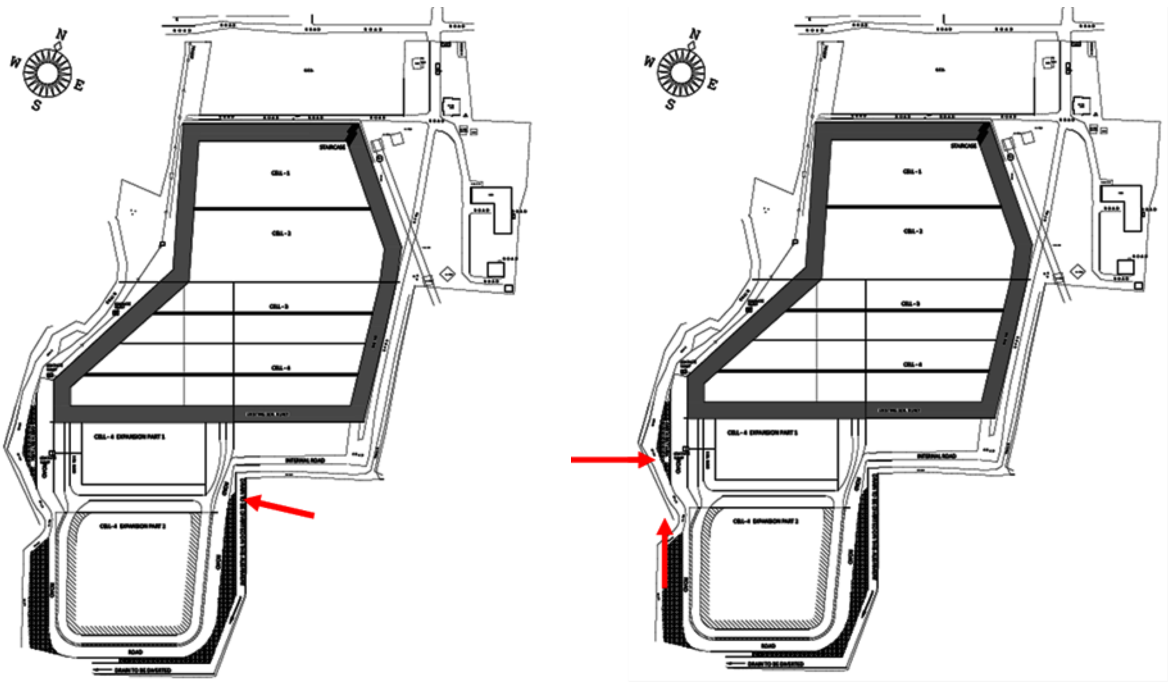


Fig. LF-12: RS containment walls constructed on old embankments to the East and West of landfill

The earth embankment structure required strengthening with soil nails. Geotechnical parameters were conservatively determined on the basis that the existing structure was “just safe” under the existing conditions. Slope stability analysis was carried out with these conservative strength parameters and with the anticipated loads of the reinforced soil structure systems on both East and West sides of the landfill (see Fig. LF1-2). Accordingly, series of soil nails were devised such that the safety factors exceeded 1.5.

Nailing on the East Side

On the East side, the embankment required cutting and the soil nails were required to retain a vertical cut in the soil embankment (Fig. LF-13) till the reinforced soil structure was constructed.

The pictorial narrative of the reinforced soil containment structure on the earthen containment embankment strengthened by soil nails is shown in Fig. LF-13. The exposed earth surface was shotcreted and weep holes were provided for pore water pressure relief, to cater to percolated water during the monsoon rains.

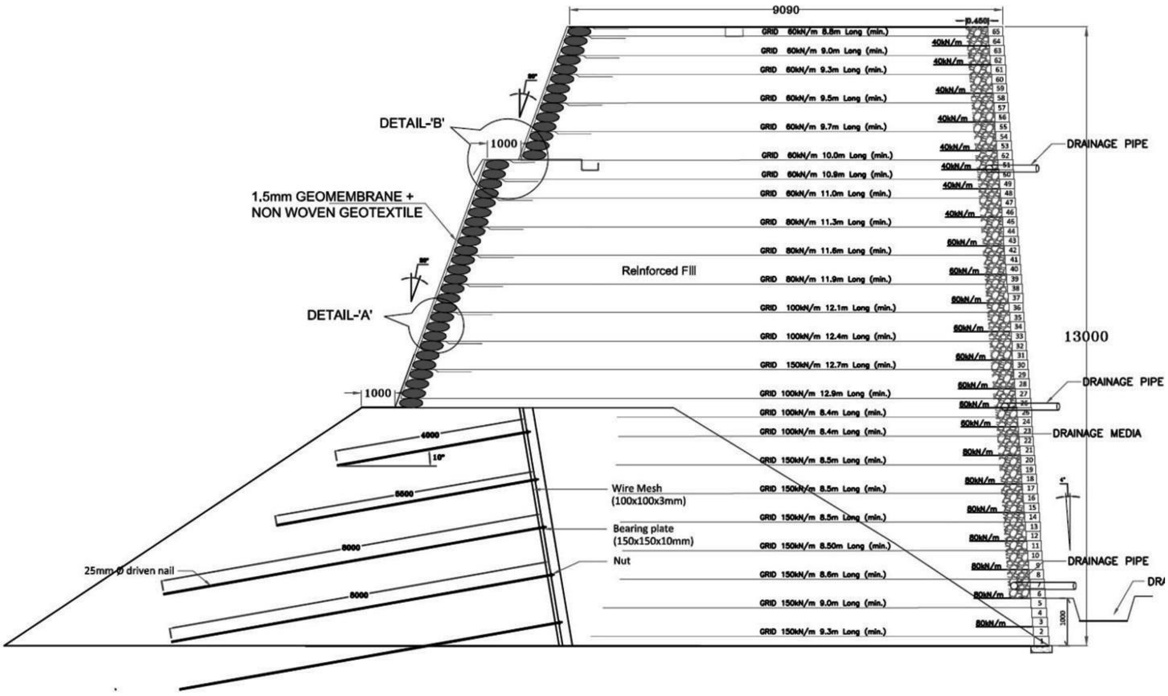


Fig. LF-13(a) East Side cross section. With West direction to left: Design of soil nails for exposed excavation

Fig. LF-13(b) East Side and facing West: Work in progress, soil nails on the exposed earth face during excavation; Note the steel mesh to hold the shotcrete

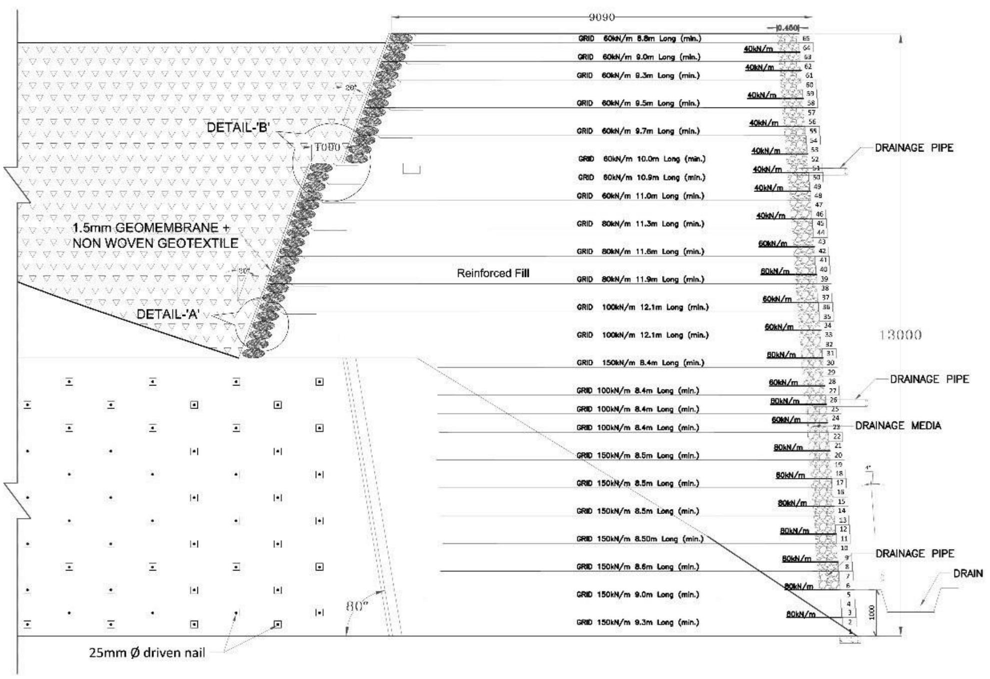


Fig. LF-13©: Cross section of East side and facing North: Design of soil nails for excavation

Fig. LF-13(d): East Side and facing North: Shotcrete work in progress, soil nails for excavation

Nailing on the West Side

On the West side, the earth embankment is required to support the reinforced soil containment wall (Fig. LF-14). The design is illustrated in Fig LF-14(a).

The reinforced compacted granular fill at the base of the reinforced soil containment wall in Fig. LF-14(a) was to provide a capable foundation for the reinforced soil structure, as well as to ensure that no pore water pressures develop within due to any unlikely percolation of water from the surface of the landfill.

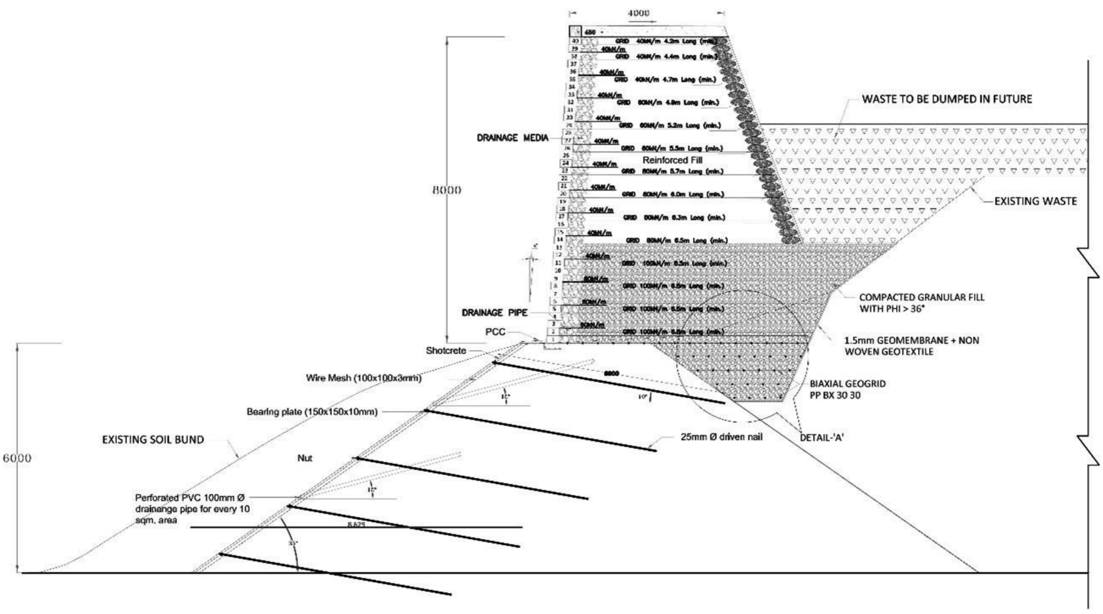


Fig. LF-14(a): West side: Design of soil nails for excavation

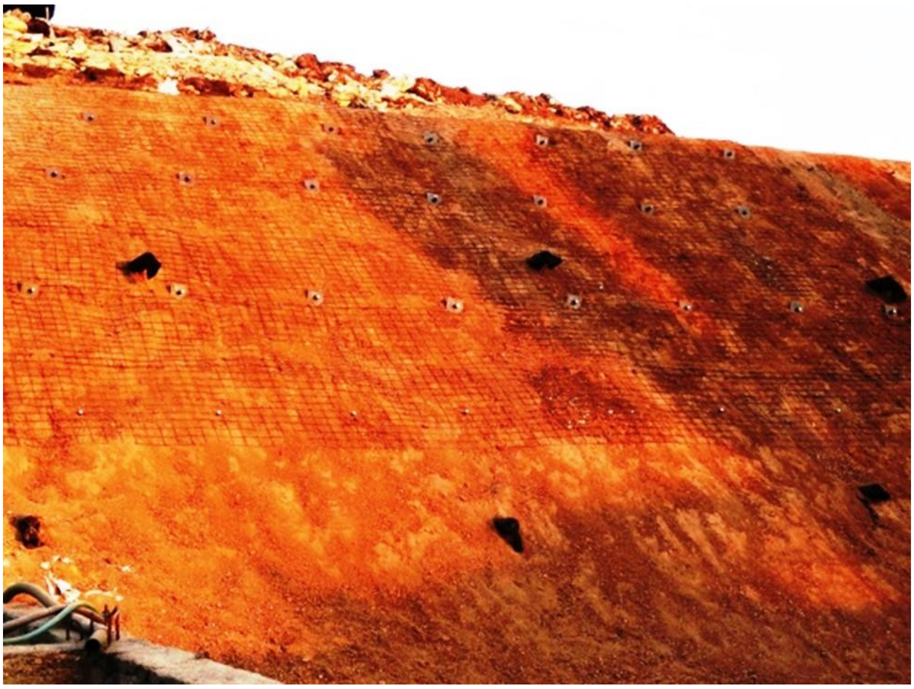


Fig. LF-14(b): West side: Soil nails for excavation in progress

The reinforced soil containment on the earthen containment embankment strengthened by soil nails is shown in Fig. LF-15. The exposed soil slope has been shotcreted and weep holes have been provided for pore water pressure relief.

Fig. LF-15: West side: The completed transition at the West side; reinforced soil containment structure atop the old earth embankment strengthened by soil nails, and shotcreted; Note the weep holes through the shotcrete veneer

Nailing Details

Fig LF-16 illustrates driving of soil nails. The work was executed by a specialist agency's team.

Fig. LF-16: Typical soil nail driving by specialist agency's team

GREENING OF STEEP CONTAINMENT EMBANKMENT SLOPES DRAPED WITH GEOMEMBRANE

Conventional containments of landfills are traditional trapezoidal embankments. The outer slopes are shrouded with geomembrane. Quite often, the outer slopes are steep, and the geomembrane is untextured. From aesthetics and mandatory requirements, these outer slopes require to be vegetated.

While erosion control and greening of steep slopes is discussed in a separate Section, greening a slope shrouded with geomembrane is generally unique to landfill containments and is therefore discussed in this Section. Provision of greenery on a steep slope shrouded with geomembrane which is sometimes untextured, requires innovative techniques.

VAPI: GREENING OF GEOMEMBRANE COVERED STEEP SLOPES OF CONTAINMENT EMBANKMENT

The concerned area is the outer slope of the North-side containment embankment of the closed Cell 1. The innovative approach of greening the untextured geomembrane shrouded slope has been successfully repeated at the Ghazipur landfill in the National Capital Region of Delhi.

Initially, Strata Geosystems was not contracted to work on the outer slopes of the existing earth embankments containing the closed Cells. The outer slope of the containment is steep, 1V: 1.5H and shrouded with smooth, untextured HDPE geomembrane. The slope was required to be greened.

Earlier, the Client had made several attempts to place soil on the untextured geomembrane on the 1V: 1.5H slope but was unsuccessful. Based on case study reports of its past successes, Strata Geosystems was approached during its work on Cell 4.

Previously, Strata had successfully used StrataWeb® geocells spiked to the sloping surface to retain soil along with vegetation on steep slopes for slope erosion protection. However, the current case posed a unique challenge since the geomembrane shroud could not be spiked to support the geocells. The solution proposed was innovative. Geogrids were used to support the geocells along the slope.

The entire system of geocells along with the infilling and vegetation and supporting geogrids needed to be anchored at the crest. An anchor trench was also not feasible owing to the completed cover of the cell at the crest. Hence, an anchor mound was designed to resist the sliding forces. The anchor mound is shown in Fig. LF-17. There was only 3m space between the top peripheral drain and the crest edge. The anchor mound had to be accommodated within the space and yet provide the required resistance against sliding of the soil infilled geocells. The earth mound was designed as a reinforced soil structure to avoid flat side slopes. The entire mound was shrouded in nonwoven geotextile carefully sealed by double needle stitching as can be seen in Fig. LF17.

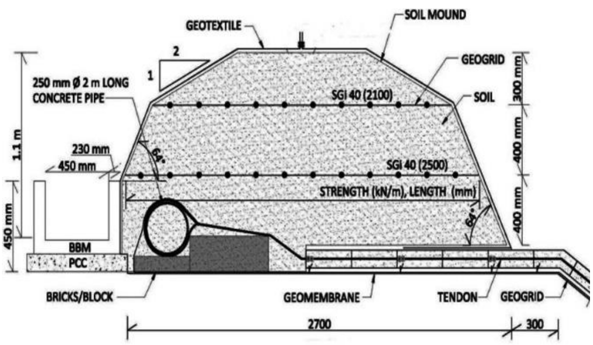


Fig. LF-17: Anchor mound at the crest holding the infilled geocells, the holding geogrid and the enabling tendon

StrataGrid™ geogrid Sgi 40 (PET flexible knitted and coated geogrids with 40kN /m strength in machine direction and 30kN / m in cross-machine direction) was temporarily secured but temporary weights and dropped down the slope, machine direction along down-slope, as shown in Fig. LF-18. Two adjoining widths were secured by cable ties as seen in Fig. LF-18.

Fig. LF-18: StrataGrid™ Swi40 geogrid laid down the slope; adjacent widths secured together with cable ties

StrataWeb® geocells SW445 125 (weld spacing 445mm, depth 125mm) was laid along the slope, and tied to the geogrid by cable ties (Fig. LF-19). StrataCord® tendons were used to guide the geocells down the slope. The geocells were initially temporarily secured at the crest. After the geocells were properly secured to the geogrids, the anchor mound was constructed at the crest to its permanent profile as seen in Fig. LF-17.

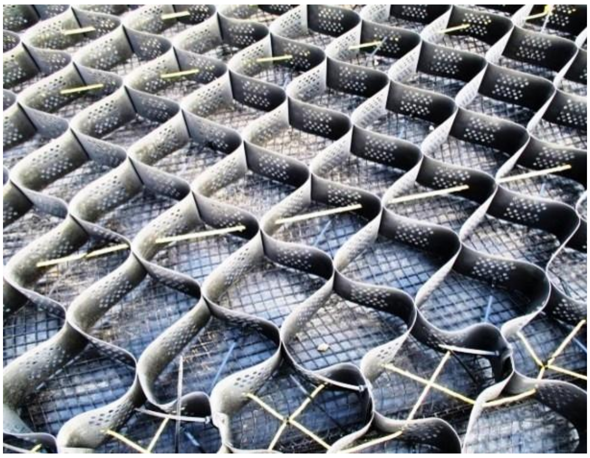


Fig. LF-19: Tying StrataWeb® geocells to StrataGrid™ geogrids

Thereafter, the geocells were infilled with organic soil. The soil was turfed (Fig. LF-20). With the monsoon rains, the slope was covered with lush greenery. The system has survived the heaviest of monsoon rains prevalent in this part of the country.

Fig LF-20(a): Geocells being infilled

Fig LF-20(b): The greenery after the initial monsoon showers

Conclusions

The three features that make this Project unique are:

a) land optimization, with vertical reinforced soil containment structure,
b) reinforced soil walls constructed atop earth embankments,
c) steep green slopes over untextured geomembrane shrouding.

Despite the challenges and hurdles that arose during this Project, astute innovations saved the client time, money, and the burden of procuring additional land in a prime region.

A bird's eye view of the complete Vapi Green Enviro landfill is seen in Fig. LF21. The picture clearly tells the narrative; reinforced soil technology makes it possible to optimise land space for landfills, particularly in and around urbanised and industrialised zones, where land space is at a premium.

Fig LF-21: Bird's eye-view of the two types of containment structures; the conventional earth structure (above, North) and the reinforced soil structure (below, South); Note the differences in footprint areas

Geosynthetics also helped solve many other challenges that came up along the way – the greening of steep slopes, the reinforced soil structure on the earth embankment, optimising the anchor mound, etc. The Project showcases how judicious innovations created a first of its kind landfill in India.

EROSION PROTECTION FOR ENCASEMENT SLOPE AT GHAZIPUR LANDFILL

In containment systems for landfills, geomembranes are invariably used as leachate barriers, to seal off the waste from the ground onto which it is dumped, as well as the environment in general. After the filling is completed to capacity, the landfill is sealed off using layers of appropriate soil and geomembranes. Along the sides, the earthen embankments confining the landfill must be rendered stable with a factor of safety higher than conventional, considering the consequences of a breach. Outer slopes of such containment can be of the order of 1V: 4H.

Real estate within the National Capital Region of Delhi (NCR) is at a premium. The landfill at Ghazipur is located on prime land within the NCR. To generate a small footprint for the containment structure, the outer side slopes of the containment were constrained to be steeper than conventional. The slopes of the containment embankment, constructed from local silt, were adequately stable.

The surface of the slopes was lined with geomembrane. The geomembrane was required to be protected against physical damage and UV by soil cover. In addition, the soil cover was required to foster vegetation since the client desired an aesthetic, green and environmentally friendly façade. Soil cover of local silt laid on geomembrane could not be sustained since the friction between the soil and the geomembrane was low. Also, the length along the inclination was as much as 30m in many sections. Furthermore, strict deadlines required that the lining protection be completed to satisfaction within six weeks. The tasks within the time frame of six weeks included design, Optimize and execution.

Fig. LF-22 shows the initial condition of the slope.

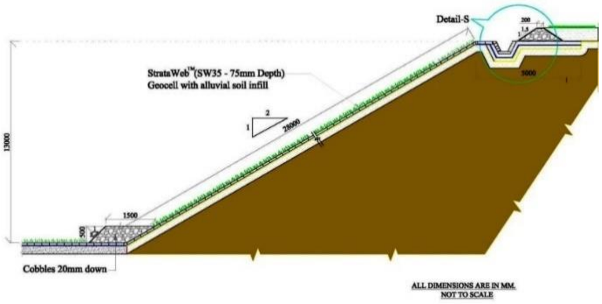
Fig. LF-2-2: Original condition of the containment slope at the Ghazipur landfill

There were several major issues which needed addressing:

1. The slope was 2H: 1V and steeper at places.
2. The geomembrane over which the soil was to be placed was untextured.
3. Geomembrane negated use of steel spikes for supporting geocells.
4. The inclined lengths of the slopes ranged from 25m to 30m.

With these constraints, the solution lay in providing geocells with a supporting system quite different from what is normally adopted, but similar to that adopted at Vapi. The solution was engineered with a combination of geogrids and geocells, schematically shown in Fig. LF-23(a). The geogrid-geocell system is anchored at the top as schematically shown in Fig. LF-23(b) and draped down the slope with the machine direction along down-slope. The geocells are connected to the supporting geogrids with high strength cable ties shown in Fig LF-24(a). Fig LF-24(b) and (c)) show fixing details with cable ties.

The style of geocell considered for the purpose was SW356 75 (weld spacing 350mm, depth 75mm) and the geogrid used was Sgi 30 (uniaxial with tensile strength 30kN/m).



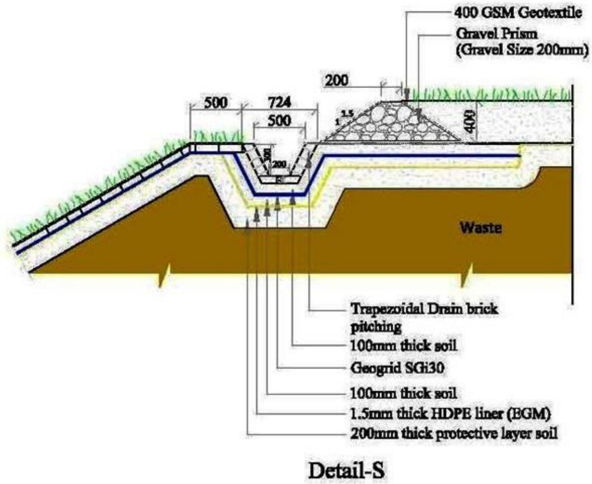


Fig. LF-23(a): Slope cross section

Fig. LF-23(b): Anchor and drain details at crest

Fig. LF-23: Typical slope details

Fig. LF24(a): High strength cable tie

Fig. LF-25(b): Adjacent geogrids connected with cable tie

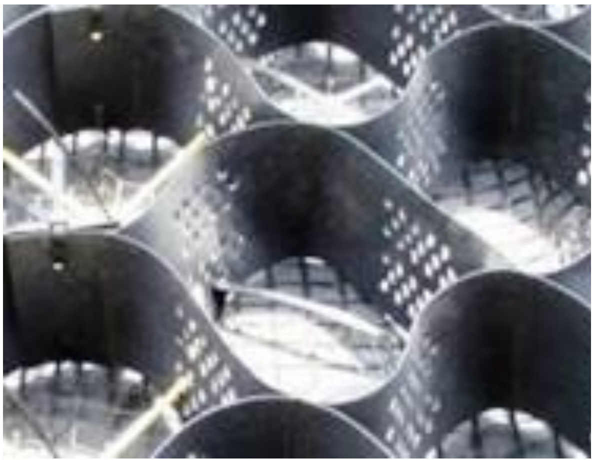


Fig. LF-24©: Geocells connected to geogrids with cable ties

Fig. LF-24(d): Two adjacent panels of StrataWeb® connected together with cable ties

Fig. LF24: Use of cable ties for connecting system components

The geocells and geogrid composite was anchored at the crest by weighing down with a mound of gravel along the crest, the schematics of which is shown in Fig. LF-23(b). For this, a factor of safety of 1.5 was considered.

Soil filling of the geocell panels was done from top and spread manually taking substantial care that each cell was fully filled. The soil was dressed and compacted (Fig. LF-25).

Fig. LF-25(a): Geocells laid out

Fig. LF-25(b): Soil in-filling

Fig. LF-25: Laying out StrataWeb® and in-filling

Grass seeds were sown along the slope soil cover. Intermittent watering was done and within a few days' time, a green cover was fostered making the slope aesthetic, blending well with the landscape (Fig.LF-26).

It may be significant to note two flaps of HDPE placed upright and parallel to the strike of the slope perceived as black traces in Fig. LF-26. This was yet another innovation on the Project to serve two basic purposes, essentially to prevent erosion:

1. The flap breaks the energy of surface run-off and reduces the magnitude of erosion of soil infill.
2. It prevents sliding of soil further down the slope.

Fig. LF-26(a): Seeded infilling

Fig. LF-26(b): Vegetation

Fig. LF-26: Vegetated infill

Eight years have gone by since completion of the work. Visual inspections have indicated that the geocell-geogrid system has performed very well. The soil cover has shown no indication of creep or slide, and erosion is barely perceptible. In retrospect, while the problem was challenging, it was obvious that once the posers were resolved, the start-up was easy, and the work proceeded at a fast pace. The strict timelines were met with. The major advantage in the NCR was that there was minimal requirement of unskilled labour and no particular trade skills were involved.

Good natural construction material such as sand is available only at a premium. The technique with geocells could use locally available silt. The original solution was to use concrete tiles which did not prove successful. The geocell solution required no concrete.

While there was an overall savings of 15% on the liner, the solution was inarguably green and aesthetic as a landfill.

SECTION II: THE GEOCELL AND ITS BASIC ESSENTIALS

(Code: GC)

THE ESSENTIALS

Geocells are basically geosynthetic reinforcement. They are lightweight but strong, three-dimensional, curvilinear rhomboidal cellular confinement systems.

Geocells are often used for rigorous, heavy-duty usage as in load bearing, or even elementary applications such as level and sloping ground erosion protection and control.

Geocells are fabricated from ultrasonically welded HDPE strips that are expandable at site to form the cellular structure (Fig. GC-1). The cells (sometimes referred to as “pockets”) of a geocell system for load bearing applications are filled essentially with non-plastic soil and non-plastic marginal materials such as pond ash. If vegetation growth is desired within geocells particularly for erosion control applications, the cells are infilled with plastic and organic soils to sustain greenery, or gravel, or lean concrete. The cell walls are perforated basically for pore water pressure relief and for soil-to-soil interaction, or to maintain monolithic conditions fir concrete infills, as the case may be. The walls are also textured for better soil-cell wall interaction and on case of concrete infill, for better adhesion between concrete and the cell walls to minimise crack width.

Infilling is an essential requirement for functioning of geocells for any geotechnical application. The perforations and texturing of the cell walls along with the infilled non-plastic soil create a semi-rigid geo-composite of sorts, to provide a stiff mat, particularly for load bearing applications, and drainage as in gravity wall and fascia applications. Infilling of any soil including gravel and concrete is essential for erosion protection and control.

Fig. GC-1(a): Geocells brought folded to site for ease in packaging and transportation

Fig. GC-1(b): Expanded geocell panel

Fig. GC-1. Typical geocell panel

Basic Applications

The geocell is a versatile product and provides ample scope for innovative engineering. Geocell panels are deployed for diverse purposes including road (i.e., support embankment and pavement) reinforcement, foundation stabilisation, stability of embankments on weak soils, slope erosion protection, gravity walls, fascia for reinforced soil embankments, etc. The latest fad among interiors architects includes geocell panels on walls as décor and ceiling light shades, and wine cellars.

From engineering considerations, there are five basic applications:

1. Reinforcement for vertical loads in bearing.
2. Slope and level ground erosion protection.
3. Lining for water pondages and channels.
4. Gravity walls,
5. Fascia for a reinforced soil structure.

There are geocell styles for each of these applications to cater to different functions of the geocell. Basic key dimensions are highlighted in Fig. GC-2. Where Strata is concerned, a geocell is defined by two basic length-dimensional parameters as seen in Fig. GC-2(a):

1. Distance between two congruent welds along each strap, which is constant for a style of geocell panel; this has a particular bearing for a particular application.
2. The depth of the geocell, *h*.

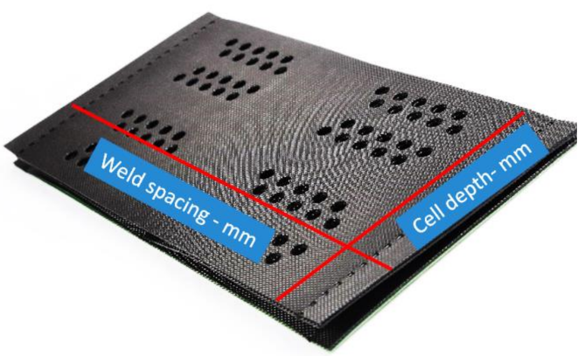


Fig. GC-2(a): The collapsed cell, defining weld spacing and height or depth

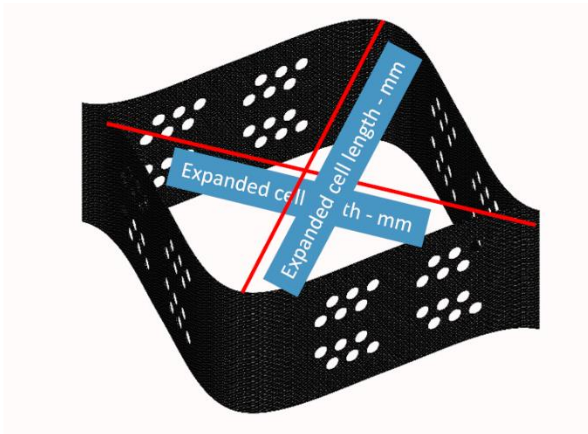


Fig. GC-2(b): The expanded cell defining the width and length

Fig. GC-2: Defining the basic key dimensions of a cell

A crucial ratio for geocells for load bearing applications is the ratio of depth of the geocell *h* and the length of the side of the rhomboidal cell, *d*. The length of the side of the rhomboidal cell, approximated to a straight line is defined as

$$d - \frac{1}{2}\sqrt{\left({d_1}^2 + {d_2}^2 \right)} \qquad \text{(GC-1)}$$

Where
d_1 and d_2 are measurable length and width along the rhomboid diagonals, as seen in Fig. GC-2(b).

Generally, geocell features for each of the above applications are a follow:

1. For geocells to be designed for vertical load carrying applications, the *h/d* aspect ratio should be close to 1. Weld spacing may be of the order of 330mm or 356mm. The depth of the geocell *h* may be 100mm, 125mm, 150mm and in extreme cases, 200mm. It is to be noted that generally, the depth does not exceed 200mm owing to compaction constraints.
2. In case of slope erosion protection, the depth *h* may be 50mm, 80mm or 100mm, and in some cases, 150mm. The weld spacing is generally high, of the order of 445mm and 660mm. The aspect ratio is of no consequence for this application.
3. In the case of geocells for gravity retaining walls or fascia for reinforced soil structures, the depth of the geocell *h* may be as high as 200mm, while optimization the *h/d* aspect ratio close to 1. However, it must be ensured that the infilled material is well compacted manually.

SECTION III: THE MICROLEVEL MECHANICS OF LOAD BEARING GEOCELLS

(Code: MIGC)

Basic Principles

Geocells have been in use even before engineers and researchers evolved the mechanics and the mathematics behind the principles of the load bearing geocell. At the outset, parameters were set based on tests and experimentation. But it was a proven fact that geocells infilled with non-plastic soils and placed over a subgrade, spread the load applied normal to the geocell mat plane over a wider area, as compared to that where there is no geocell layer. It is only recently that theories behind the functioning of geocells are being developed.

Load bearing geocells are filled with non-plastic material to form a semi-rigid mat, capable of distributing imposed loads over a larger area. Hence the magnitude of bearing pressure on the supporting subgrade is lower than that if there was no geocell reinforcement.

Consider a planar mat as in Fig. MIGC-1 (a), a geocell panel infilled with non-plastic soil. When a load is applied normal to the surface of the geocell plane, bending moments develop within the system. The bending is resisted within the system by the vertical cell walls as well as the infill non-plastic soil as seen in Fig. MIGC-1(b). The resistance offered by the surrounding infilled cells contributes to the ability of the geocell-soil system to spread the load over a larger area and thereby, the pressure bearing upon the subgrade is reduced.

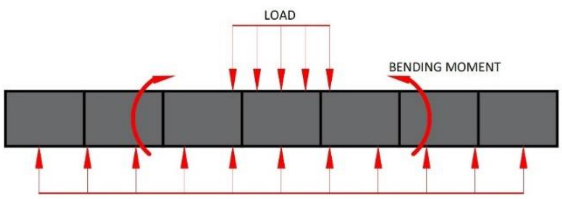


Fig. MIGC-1(a): Bending moment generated within geocell panel to resist imposed force

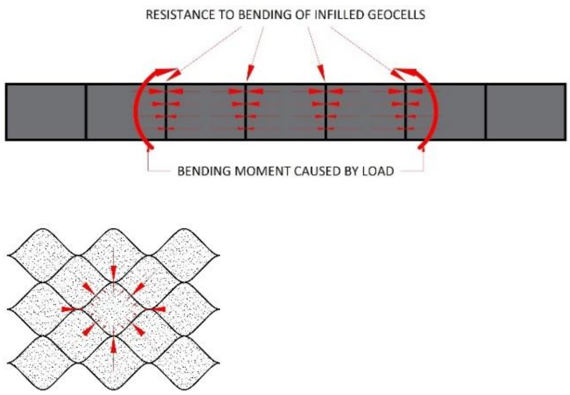


Fig. MIGC-1(b): Resistance to bending within the geocell panel

Fig. MIGC-1. Mechanics of geocells

The resistance offered by the surrounding infilled cells is explained considering the mechanics of pressures within the infilled cell, brought out by Neto *et al* [2013]. With reference to Fig. MGC-2, if q_o is the imposed vertical pressure on the non-plastic infill of the cell, lateral stress σ_{h0} is generated against the walls of the cell which is approximated to

$$\sigma_{h0} = k_0 q_0 \qquad \text{(MIGC-1)}$$

Where

k_0 is the "at rest condition" earth pressure coefficient since akk cells are infilled and the cell walls have no scope for lateral deformation.

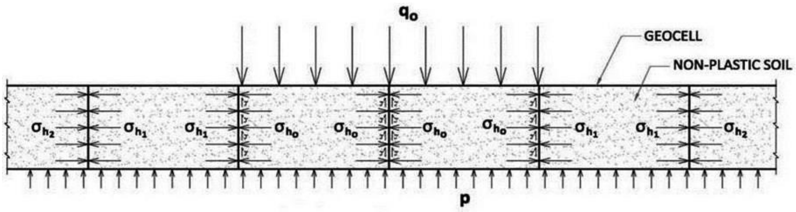


Fig. MIGC-2: Pressures invoked within cells of a geocell panel (after Neto *et al*)

Consider the cells just beyond the loaded area stressed vertically by pressure q_0. Due to lateral stress σ_{h0} generated against the cell wall within the cell stressed by vertical pressure q_0, lateral stresses are also generated in those congruent cells but which are not vertically stressed, as equal, and opposite reaction, whose magnitude would be less than (but marginally) σ_{h0} owing to resilience of the HDPE cell walls. These lateral stresses are generated in adjoining cells as reaction, one after the other. These pressures increase the shear strength of the confined non-plastic soil within the cells as elaborated later herein to create a semi-rigid mat which distributes the imposed vertical pressure q_0 over a larger area.

What has significantly been ignored here is the "deep girder effect" of the cell walls, particularly in the light of the aspect ratio approximately equal to 1 for load bearing geocells.

CREEP IN GEOCELLS

Considering the mechanics of transfer of pressures to generate load spread, it is significant to note there is no scope for creep in load bearing applications.

Creep is the tendency of any solid material to move slowly or deform permanently under external application of sustained stress over a period. While creep occurs in all solid polymers, it is necessary to understand:

1. the application of the polymeric geosynthetic,
2. the extent of the stress or strain vis a vis at yield,
3. a constant load sustained over time, and
4. an ability to undergo sustained deformation at all.

In the case of polymeric geosynthetics, when the material is subjected to an applied load, the molecules of the material tend to move apart and stretch. This leads to elongation of the material in one direction. There will also typically be thinning of the material thickness. However, it is a major aberration when the designer applies reduction factors due to creep when consideration of creep *per se* is of no relevance. Such is the case while designing for geocells.

Load bearing geocells as a geosynthetic comprise of three dismensional cells infilled with soil or aggregate. Moreover in a majority of the applications, the geocells are either subsurface as in foundations and pavements, or have a backing of soil as in a retaining structure, or are closely spiked as in the case of slope erosion contril.

As the geocells are installed on a separator over weak subgrades and infilled with good quality compacted non-plastic soil or aggregate, the cell walls of the geocells do not have any scope to undergo significant expansion. Even when loads are applied on a geocell through a wheel load, the cell wall membrane strains within the geocell are very low and of the order of less than 1%.

Numerical analysis has been carried out by Gedela R and Rajagopal, K (2020) where a wheel load on a pavement reinforced with a layer of geocell was simulated through a 300mm diameter plate. The exact replica of an expanded curvilinear rhomboidal shape of the HDPE geocell panel was incorporated in the numerical model as seen in Fig. MGC-3. The objective of the analysis by Gedela and Rajagopal was to determine the stresses and strains in the cell walls. The findings also adequately demonstrated that creep cannot take place in load bearing geocells.

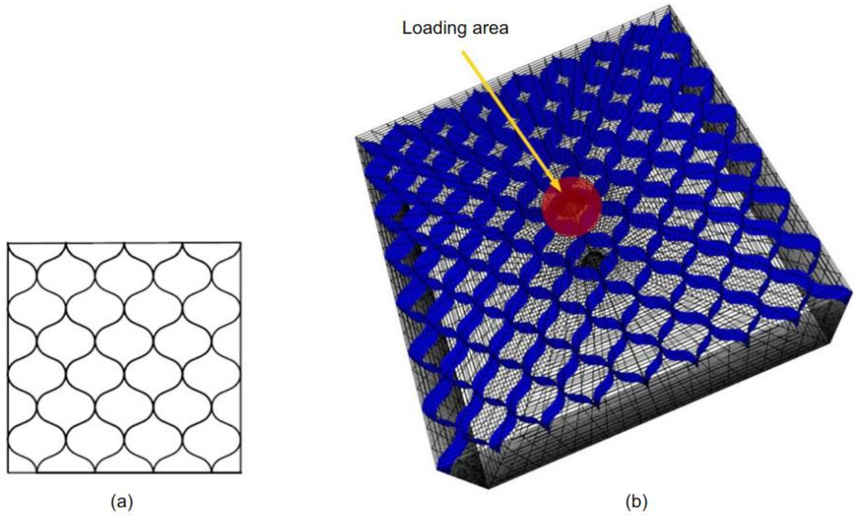


Fig. MIGC-3: Replica of curvilinear rhomboid shape of geocells in numerical model (after Gedela & Rajagopl)

The analysis considers that the mat is rigid, which is not a significant assumption since the objective is to prove that creep, where geocells are concerned, is not of relevance. Load was applied on a rigid plate and the response of the geocell layer was evaluated. The average of membrane (cell wall) stresses in the geocell cell directly below the loading plate was observed to be about 800kPa, when the plate settlement was equal to 10% of its diameter, and about 2,000kPa when the plate underwent very large settlement, equal to about 40% of the plate diameter (Fig. MIGC-4).

Three cell shapes of a cell have been considered for weld spacing of 356mm. It is interesting to note that for a given settlement of the system, the idealised square and “diamond” shapes demonstrate higher average cell wall stresses as compared to the realistic curvilinear rhomboid cell shape, indicated as “honeycomb” in Fig. MIGC-4.

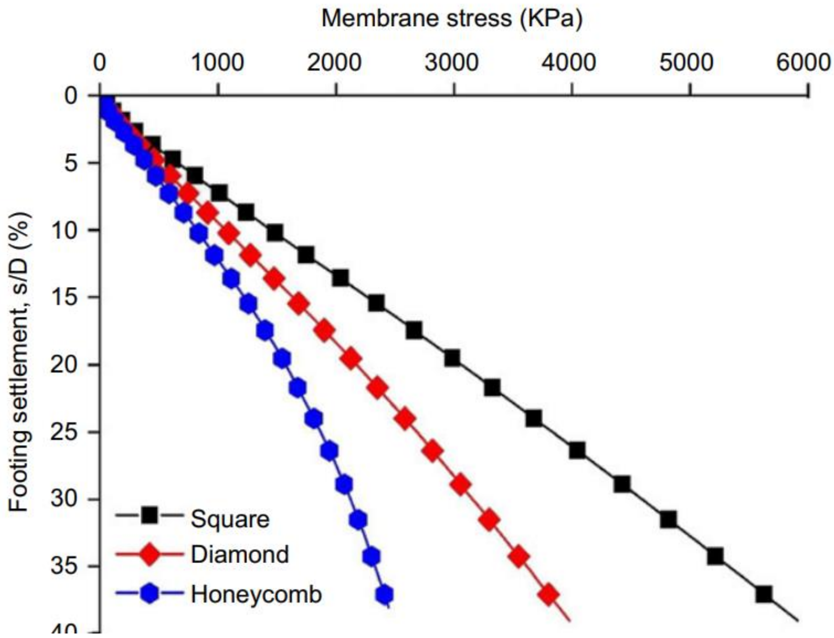


Fig. MIGC-4: Average cell wall stresses in the central cell below the plate, for various settlements and different idealised shapes and the realistic curvilinear rhomboidal (“honeycomb”) shape (after Gedela and Rajagopal)

Consider the stress-strain relationship of the HDPE straps that form the cells of the geocell, shown in Fig. MIGC-5, tested as per ASTM 6693-08. The cell wall stresses of 800kPa and 2,000kPa correspond to about 0.35% and 0.85%.

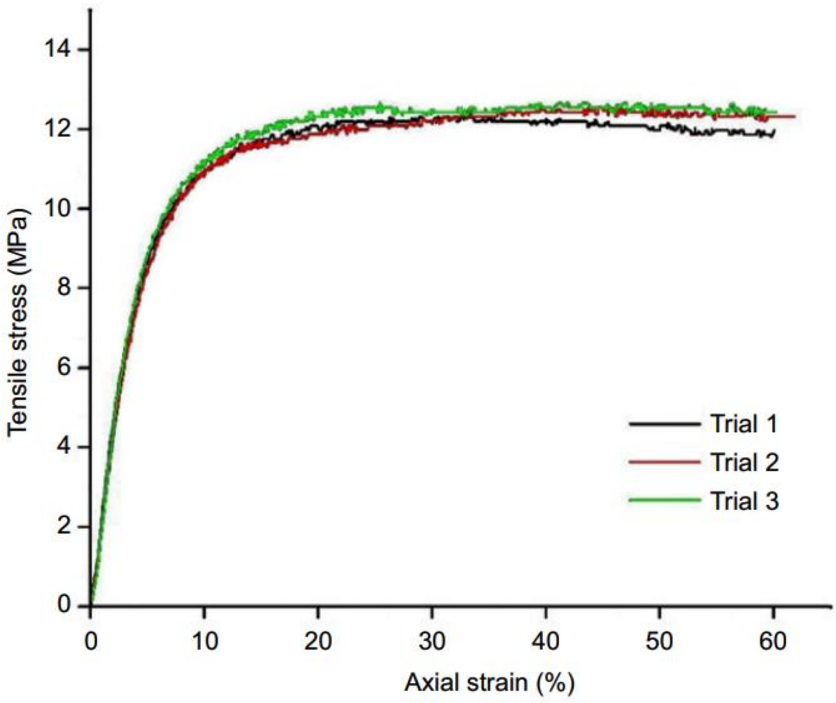


Fig. MIGC-5: Tensile stress – strain response of geocell HDPE strap (after Gedela and Rajagopal)

Hence it is proven that even when there is a direct load on a geocell panel cell and tending to enlarge it, the strains are very low since each cell is surrounded by other soil infilled cells. There is no scope for creep of the geocell straps.

When geocell panels are used to construct gravity retaining walls or used as fascia for reinforced soil systems, they are subjected to vertical compression due to weight of soil above and lateral thrust. Under such load conditions, significant cell wall strains / stresses will not develop within the geocell walls.

Creep of HDPE geocells is not a relevant design parameter.

SECTION IV: THE MACROLEVEL MECHANICS OF LOAD BEARING GEOCELLS– ELASTIC PROPOSAL

(Code: MAGC)

PREAMBLE

This Section is confined to analysis of geocells under direct normal vertical load. It highlights a simple method to evaluate deflection and pressure distribution below a layer of load bearing geocells, placed on subgrade. This analysis deems the geocell layer and the subgrade below as a two-layered system. Deflections are evaluated at the interface of these two layers. The objective essentially includes determination of the spatial extent to which geocells are effective below the loaded area, i.e., the distance at which the pressure due to imposed load die out.

The proposed technique does not require any sophisticated analytical tools or software other than Huang's curves which consider two layers with elastic characteristics.

When a vertical pressure over a limited area is applied onto a geocell panel infilled with non-plastic soil, a wide-angle load spread has been observed by several researchers and geocell promoter-organisations through field and laboratory tests. The geocell panel develops its characteristic to spread the load through infill of congruous perforated and textured cells. The vertical curvilinear cell walls have a depth: average cell diameter ratio almost equal to unity as highlighted in the previous Section. The cell walls also contribute to the rigidity of the geocell mat and to wider load spread.

The composite structure of the geocell mat is complex for analyses by conventional mechanics and requires techniques like the finite element method. There are three significant aspects to be considered:

1. The shear strength and modulus of a non-plastic soil significantly improve when it is confined within the cell walls. Such improvements in the infill soil parameters enhances the performance of the geocell-subgrade system.
2. The elastic characteristics of the HDPE geocell wall.
3. The infilled soil-geocell wall interaction.

While considering each of these factors would require a complex mathematical model, the method of analysis suggested herein considers the composite geocell infilled with soil as one homogeneous entity.

THE TWO-LAYER SOLUTION

When vertical pressure over a limited area (as from a footing) is applied onto a geocell panel, there is a wide-angle load dispersion by the geocells.

As explained above through Fig. MiGC-1, the geocell panel develops its rigidity through infilled congruous geocells as well as the vertical geocell walls, where the aspect ratio is close to unity. While the composite structure is complex for conventional mechanics analysis, one solution is the application of the Burmister concept, considering the infilled geocell mat as the top homogeneous layer over the subgrade, which would be the lower second layer. Both these layers are considered elastic.

ASSUMPTIONS

To relate the Burmister concept to geocells, the following assumptions need to be considered:

1. Material properties at any point within the geocell layer are homogeneous. Likewise, within the subgrade, material properties at any point are similar.

2. Stress – strain solutions are characterised essentially by two material properties, elastic modulus, and Poisson's ratio for both geocell layer and the subgrade respectively.

3. The cellular configuration of the geocell layer is ignored and the infilled geocell is considered as a homogeneous and isotropic layer, notwithstanding the compartments of soil segregated by vertical HDPE cell walls. Holistically, the elastic modulus of this layer is assumed to be an isotropic E_{GC} and the Poisson's Ratio as μ_{GC}.

4. While the geocell layer has a finite depth (or thickness), the subgrade is of "infinite" depth for the sake of analysis. Horizontally, both geocell layer and the subgrade are considered to be of infinite extent.

5. Properties within the geocell layer as well as within the subgrade are assumed to be isotropic. In other words, at any specific point, the property is the same in every direction or orientation.

6. The geocell layer and the underlying subgrade are in continuous contact and at no location is there any loss of contact.

7. Full friction is developed at the interface between the geocell infill material and the underlying subgrade. There is no slippage between the layers. This assumption is justified by the fact that a 50mm layer of the infill material is placed below the geocell layer, considering that the characteristics of the geocell composite extend to that layer. A relatively thin nonwoven geotextile separation layer is generally provided at the interface which also justifies this assumption.

8. There are no horizontal shear forces at the surface, a reasonable assumption.

ELASTIC CHARACTERISTICS OF GEOCELL SYSTEM

The objective is to determine the spatial extent to which the geocell layer is effective. This is best determined by computing the stresses along the interface between the geocell layer and the subgrade. The horizontal distance from the externally imposed load to the point where the stress due to the imposed load tend to zero, is determined. This distance is indicative of the effective extent of the geocell layer.

Tests conducted in the Dandeli Forests (# Saride *et al*) highlight load spread. However, the extent of effectiveness of geocells need to be determined for various parameters and their combinations, such as:

1. Various infill types.

2. Subgrade characteristics, considering project site inputs based on basic geotechnical investigation data.

3. The areal geometry of the external imposed load.

One particular aspect that needs to be highlighted is the improvement in the *E* value of the infilled material, and the vertical extent to which this improvement is effective to generate E_{GC}. This has been earlier proven through tests by Prof K. Rajagopal [2012] of IIT Madras as well as Dr. Chandan Basu [2013]. Tests have also been conducted by Strata Geosystems and the Author. All these tests have proven that the E_{GC} value of the system improves anywhere between 2.3 and 3 tines, and sometimes >3. This improvement is cited as the "*Modulus Improvement Factor*", *MIF*. The improvement in *E* extends beyond the depth of the geocells (i.e., thickness of the geocell layer) and is recommended as 50mm above the geocell and 25mm below the geocell, provided that the material above and below the geocell is the same as the non-plastic infill. With the *E* value of the infill, one can estimate the holistic modulus value of the geocell layer. The Author has considered a *MIF* of 2.5 in many cases. *However, it is advisable to compute the* E_{GC} *value directly from cyclic plate load /* Ev_2 *tests on the infilled geocell layer, on the prototype subgrade itself in the field, along with the appropriate nonwoven geotextile separator for a realistic* E_{GC} *of the holistic geocell layer as a composite system.*

While conducting load tests, in order to obtain realistic moduli values for infilled geocell, it is suggested that the plate for the geocell tests should not exceed 300mm diameter. While the zone of influence should preferably be limited to the depth of the geocell, the entire cell area along with the walls of the geocell and beyond should be covered by the plate area such that the test is more or less representative of the geocell structure. The zone of influence is bound to cover the subgrade below the geocell layer also. Hence it is necessary to conduct the tests at the project site with the geocells placed on the subgrade to be considered.

For tests on the subgrade to determine the subgrade elastic modulus, a plate of 600mm diameter is preferred so as to cover maximum depth within its zone of influence.

The two-layer theory assumes that Poisson's ratio $\mu = 0.5$. Considering that non-plastic soil is well confined within the geocell system, this is arguably not a good assumption, since lateral deflection due to vertical stress is negligible, more so due to confinement. However, in elastic solutions of this type, the contribution of μ is not significant.

BURMISTER'S TWO-LAYER SOLUTION [1945]

The solution for a geocell on a subgrade may be approached through Burmister's solution for a two-layer problem. Stress and deflection values as obtained by Burmister are dependent on the ratio of moduli of the two layers, i.e., geocell layer at the top and the subgrade below, E_{GC}/E_S. Fig. MAGC-1 indicates stress values below the centre of a circular loaded area over a two-layered system, which one may consider as an infilled geocell overlying the subgrade.

Total surface deflection *Δ* for a flexible plate is:

$$\Delta = 1.5\frac{pa}{E_S}F_2$$

[MAGC-1]

where

p is the pressure from the circular flexible plate,

a is the radius of the circular loaded area,

F_2 is a dimensionless factor depending on the EGC/ES ratio as well as the depth to plate radius ratio (z/a) at the point where the deflection is measured. Curves for F_2 are shown in Fig. MAGC-1.

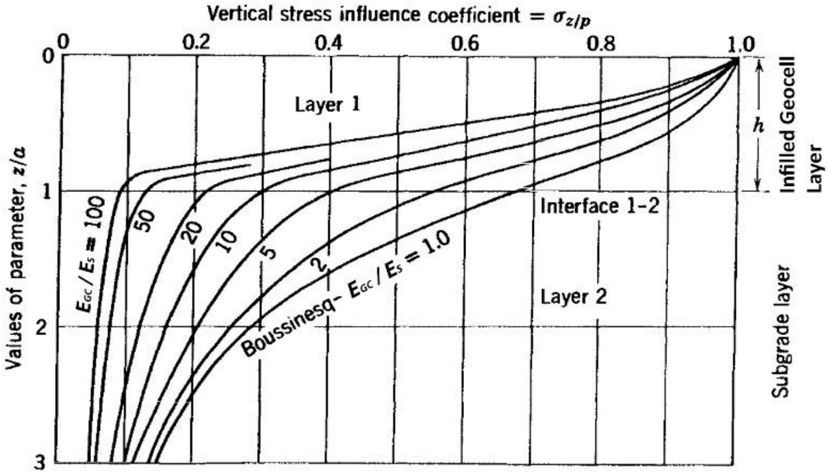


Fig. MAGC-1: Burmister's influence curves for points below the centre of a loaded area on infilled geocells

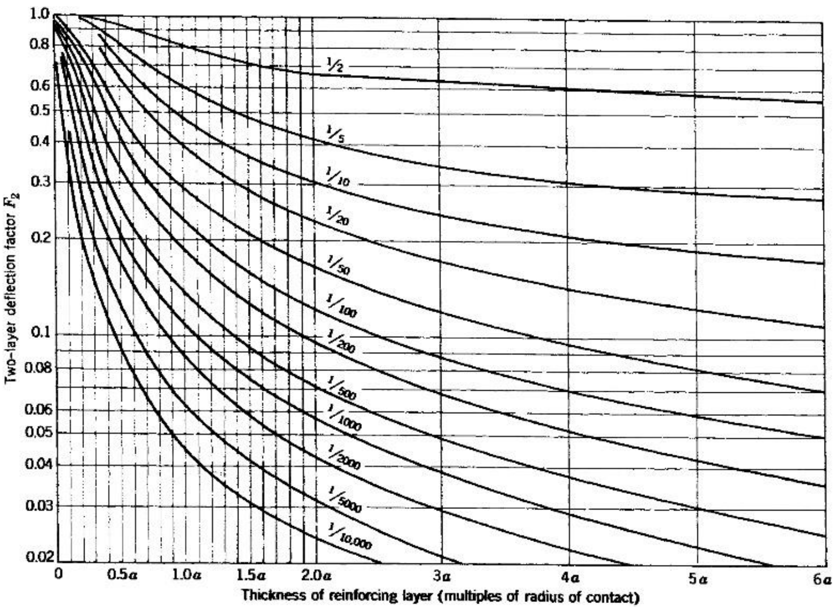


Fig. MAGC-2: Two-layer influence factor F2 for Burmister's two-layer theory considering infilled geocells (Equation MAGC-1)

However, Burmister's two-layer solution does not provide the extent to which the geocell is effective from the centre of the loading.

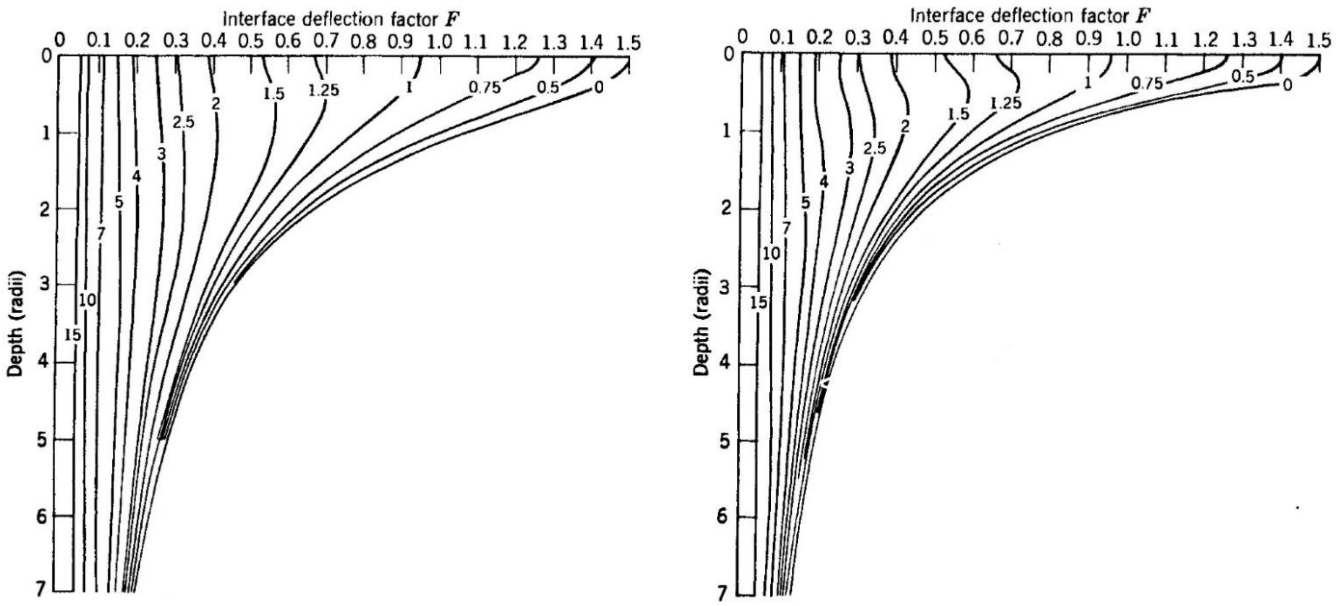
HUANG'S EXTENSION OF BURMISTER'S TWO-LAYER SOLUTION [1993]

Huang's Basic Extension

As an extension to Burmister's two-layer derivations, Huang [1993] developed charts for deflection factor F to address deflections along the interface of the two layers. As in the case of Burmister's two-layer analysis, F is determined on the basis of the assumption that μ is 0.5. The deflections Δ_{IF} at points along the interface are given by the equation:

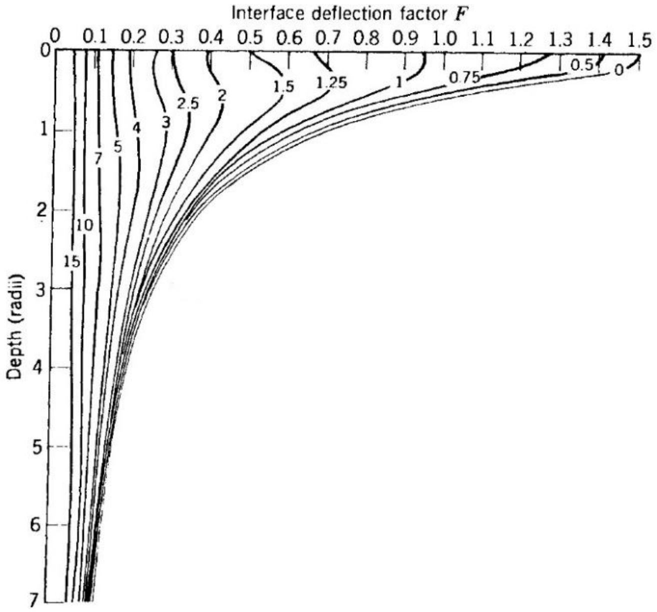
$$\Delta_{IF} = \frac{pa}{E_S} . F \qquad \text{(MAGC-2)}$$

The factor F can be determined from the charts in Fig. MAGC-3. Each chart in Fig. MAGC-3 is for a specific EGC/ES ratio.

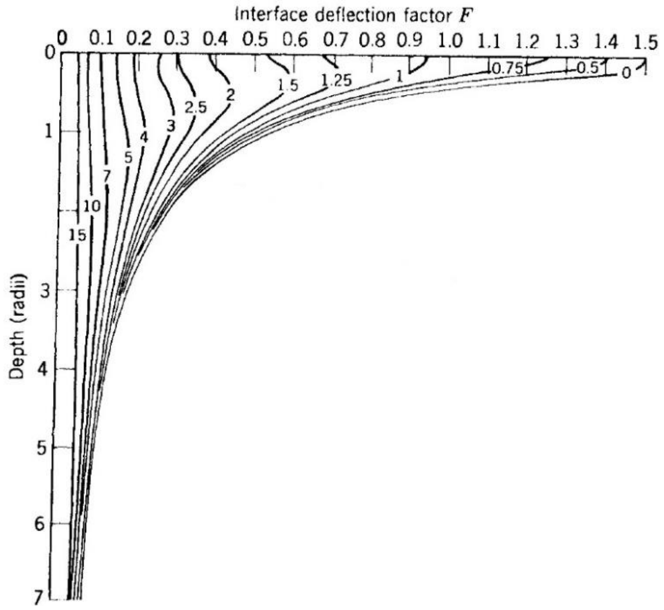


a) F for E_{GC}/E_S=1 b) F for E_{GC}/E_S=5

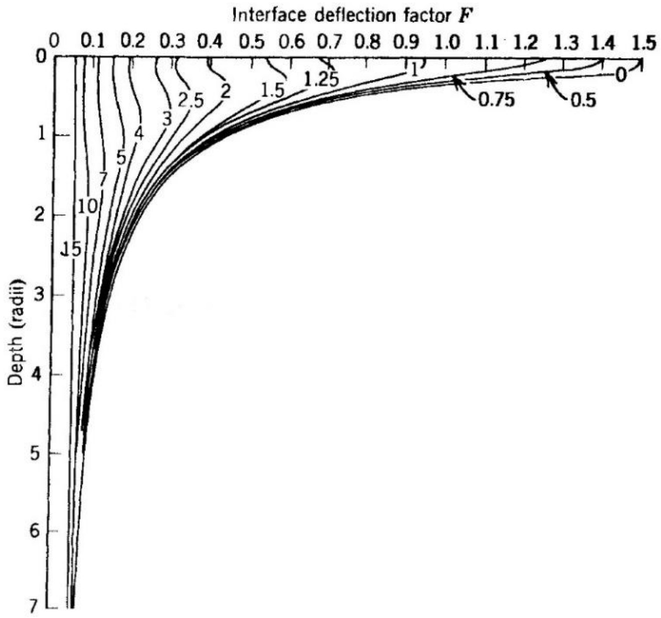
Fig: MAGC-3. F Factor for various E_{GC}/E_S. Numbers on curves indicate the Distance Ratio, D_R, i.e., distance from loading center in terms of loading radius (after Huang) …. (Continued)



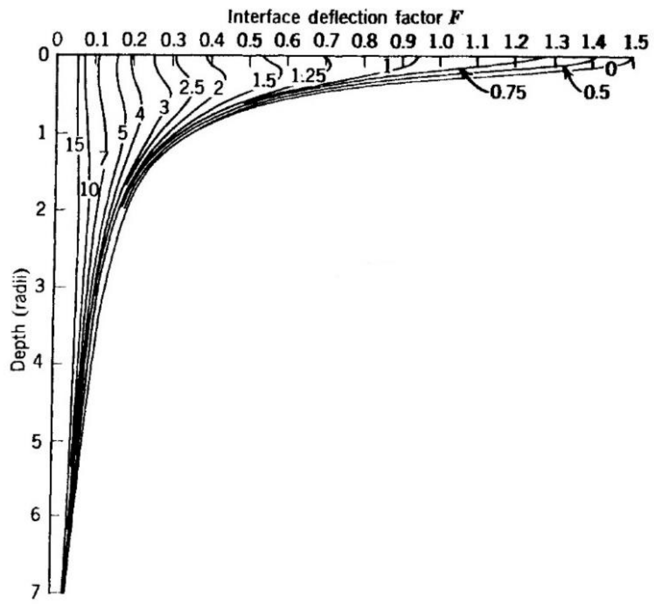
c) *F* for E_{GC}/E_S=10



d) *F* for E_{GC}/E_S=25



e) *F* for E_{GC}/E_S=50



f) *F* for E_{GC}/E_S=100

Fig. MAGC-3: (Continued). F Factor for various E_{GC}/E_S. Numbers on curves indicate the Distance Ratio, D_R, i.e., distance from loading centre in terms of loading radius (after Huang)

Determination of Extent of Effectiveness of the Geocell Layer

The curves are also characterised by the Distance Ratio D_R, which is the distance from the loading centre in terms of the loading radius.

Based on *F* determined from the appropriate curve in Fig. MAGC-3, vertical deformations Δ_{IF} are computed along the interface of the geocell layer and the subgrade using Equation MAGC-2. The computations for Δ_{IF} are carried out below the area of application of the load where the Distance Ratio D_R is <1, and also beyond the loaded area where D_R is >1. The settlement curve along the interface is plotted as in Fig. MAGC-4 for the illustrative example below. To facilitate further computation, the deflection should be computed at equal, regular intervals, as closely spaced as practical.

The plot of vertical deflection Δ_{IF} will indicate the spatial extent to which the geocell layer is effective, i.e., as $\Delta_{IF} \rightarrow 0$. If the objective of the designer is only to determine the extent to which the geocell layer is effective, the analysis may be terminated here. However, it is also essential to determine the stress profile below the geocell from considerations of stability of the subgrade and also for the purpose of detailing the system being designed.

Determination of Stresses at the Geocell-Subgrade Interface

It would be significant to take cognisance that the profile of the vertical deflection curve is similar to the profile of the stress pattern at the interface of the geocell layer and the subgrade. From equilibrium requirements, the area under the vertical stress curve is equal to the imposed vertical force. These two basic premises form the basis of determination of vertical stress profile along the interface.

To continue the solutions towards determination of stresses along the interface, the area under the curve of Fig. MAGC-4 is computed by dividing the curve into vertical strips of equal width to facilitate computation or scaling off of the deformations at equal spacing.

The area under the vertical stress curve will be the vertical force on the geocells. The ratio of stress at any given point at the interface and the total downward force will be the same as the ratio of vertical deflection at that point and the area under the deflection curve. Hence the magnitude of stress at that given point can be evaluated. When several such points at the interface are considered and the stresses are computed, one can draw the stress diagram as seen in Fig. MAGC-5, which relates to the solved example below.

Solved Example

An example has been shown below using the proposed theory. For this example, a circular footing of 1m diameter has been considered. The footing exerts a uniform pressure of 100kPa onto the geocell reinforced layer. The two layers for the two-layer theory are:

1. the geocell reinforced layer of which total thickness is considered as 200mm,
2. the subgrade of infinite depth.

The elastic modulus improvement factor for geocells has been considered as 2.5 which will be applied to the elastic modulus of the compacted infill material. In this case the E_{GC} = 125Mpa and E_S = 5Mpa hence, E_{GC}/E_S = 25.

For E_{GC}/E_S = 25, from the chart in Fig. MAGC-3(d), interface deflection factor F has been obtained and the deflections at the interface of subgrade and geocell reinforced layer are calculated at various points using Equation [MAGC-2].

The deflection values are plotted to obtain the deflection profile at the interface which has been shown in Fig. MAGC-4.

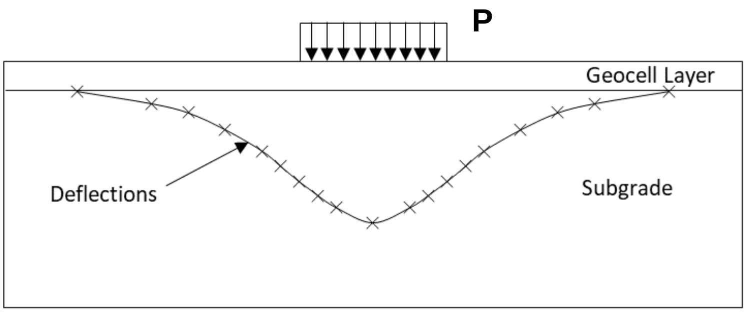


Fig. MAGC-4: Interface deflection profile (Y scale has been increased for illustration)

Based on these deflections, stresses at various points are calculated. The total area under the stress curve, from equilibrium considerations, shall equal to the total vertical force, ***P***, imposed on the geocell system.

The area under the Deflection Profile Curve is divided into vertical strips of conveniently small width. The Deflection Ratio at each strip is computed. Deflection Ratio is the ratio of the area of each strip and the total are under the Deflection Profile Curve.

These Deflection Ratios are the same in magnitude as the Stress Ratios at the respective points along the interface. Stress Ratio is the ratio of the area of each stress strip at any location along the interface within the Stress Profile Curve, and the total area under the

Stress Profile Curve. The area under the Stress Profile Curve shall equal the total vertical force P imposed on the geocell layer (Fig. MAGC-5).

Accordingly, considering Stress Ratios equal to Deflection Ratios, stresses are computed at the midpoint of each vertical deflection strip to obtain stress at the respective points along the interface. Accordingly, the Stress Profile Curve is drawn as shown in Fig. MAGC-5.

To demonstrate the effectiveness of the geocell reinforced layer, stress profiles at the interface with geocell reinforcement is compared with the stress profile at the interface without geocells in Fig. MAGC-5. The diagram compares not only the stress magnitudes but also the spread of stress profile with geocells. W

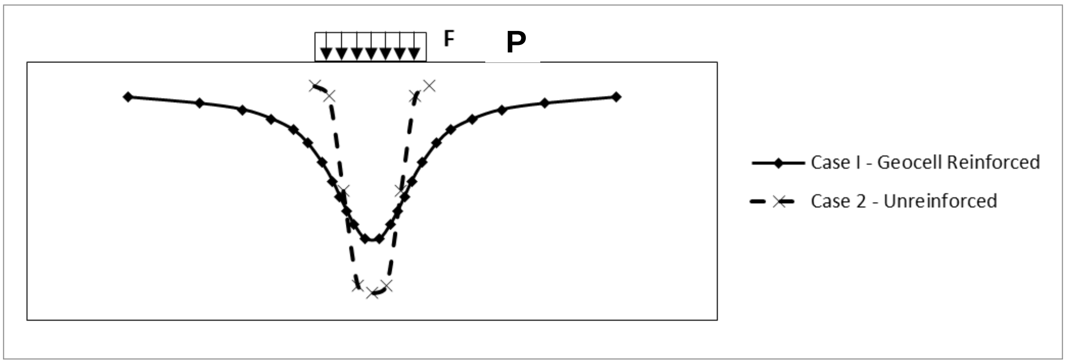


Fig. MAGC-5: Stress Distribution in case of Geocell reinforced and unreinforced sections

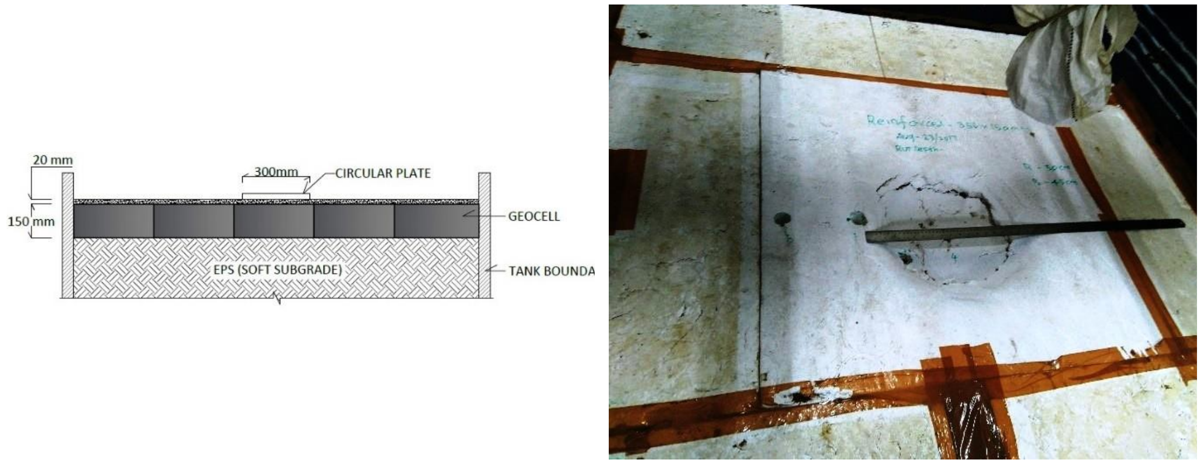
Angle of Dispersion

Based on the solved example results illustrated in Fig. MAGC-5, the Angle of Dispersion with respect to the vertical at the centre of the load is of the order of 70°.

Laboratory load tests have been conducted on geocells at the Indian Institute of Technology, Madras (IITM) by Prof K. Rajagopal. For repeated tests with consistency, in order to simulate a clay subgrade, expanded polystyrene (EPS) blocks were used. The ultimate bearing capacity of the EPS block was 100kPa. These blocks exhibited California Bearing Ratio (*CBR*) values ranging from 1.35% to 1.55%. These parameters represent parameters for a weak subgrade also.

The findings of the experiments conducted will be published separately by Prof Rajagopal. However, one aspect of the tests conducted needs to be highlighted here.

The schematic of the tests is illustrated in Fig.MA GC-6(a). After the load tests were conducted with geocell reinforced layer over the EPS subgrade in an appropriately sized tank, the indentation on the EPS block was an approximate indicator of load spread. The indentation is illustrated in Fig MAGC-6(b). Depth of the geocell used was 150mm.



a) Schematic of the test setup

b) Measurement of the indentation on the EPS block

Fig. MAGC-6: Indentation on the EPS block after laboratory load test on geocells

Prof Rajagopal recommends that dispersion be measured in terms of Load Spread Index (*LSI*). *LSI* is defined as

$$LSI = D_r/D_0 \tag{MAGS-3}$$

where

D_r is the diameter of the settlement bowl on the EPS surface for the geocell reinforced section

D_0 is the diameter of the settlement bowl on the EPS surface for an unreinforced section.

The dispersion angle for the tests with geocells is of the order of 70º. This compares well with the dispersion angle of about 70° from the solved example.

CONCLUSIONS

While designing for load on geocell systems, the extent to which geocells need to be provided beyond the loaded area has always been an enigma for the designer. This Section of the Lecture has evolved a simple method to determine not only the extent of geocells required, but also recommends how interface vertical deflections and stresses can be evaluated, using Huang's solution for two layered elastic systems. Direct application of curves recommended by Huang based on his solution for elastic two layered system helps in arriving at these three requirements by a simple method without having to resort to complex and time-consuming techniques.

The essence of the solution is a basic assumption that the subgrade is an elastic material. More reasonably, geocell mat infilled with non-plastic is considered as an elastic layer.

The load spread through analysis of a single layer of 200mm depth is about 70° with the vertical.

The solved example illustrates that a wide loaded areas would require either a thicker layer of geocell, or multiple layers of geocells. Considering the need for proper compaction of non-plastic material infilling, the depth of the geocells is normally restricted to 200mm. If thicker layers are required, multiple layers of geocells should be used.

The proposed method of evaluating deflections, stresses, and the operative extent of a geocell layer will enable the designer to design an adequate geocell system as reinforcement below loaded areas.

SECTION V: THE ROAD SYSTEM

(Code: RD)

PREAMBLE

Roads are an essential system for transfer of goods and passengers across the country and, unlike the railway network, various classes of roads, from expressways to rural roads, assure the last mile connectivity. The growing economy requires expansion of existing carriageways into multi-lane expressways, new road routes and development of rural roads to service motorised vehicles instead of the traditional animal-drawn carts. However difficult subsoil conditions along with high economic and social costs of diversions, and dearth of good construction material coupled with environmental constraints pose major challenges to development of the road network.

COMPONENTS OF A ROAD SYSTEM

A typical road as a system has two major components:

1. The embankment supporting the carriageway.
2. The pavement, which is the cases operation entity of the carriageway, though there are several roads where the pavement is supported directly on untreated / treated and dressed natural subgrade.

The two components of the road system are structural entities and should be designed appropriately.

The mechanics of the two components of a road system are different, each with its own nuances. Hence the two components are treated in separate Sections.

SECTION V – A: THE EMBANKMENT COMPONENT OF ROADS:

GEOGRIDS AND GEOCELLS AS BASAL REINFORCEMENT

(Code: EM)

PREAMBLE

This Section essentially addresses embankments on weak subgrades deriving stability with basal reinforcements. The embankment may be a typical earth structure with a trapezoidal profile without or with side berms, or a reinforced soil slopes steeper than 27°, or reinforced soil walls with batters 70° or higher. As in any other structure, two essential conditions need to be satisfied: strength and serviceability.

TERRAIN

Road systems need to traverse all types of terrain, from hills to mud flats, sometimes all along a single stretch, as an example, the NH-48 over its traverse along the West Coast frio Thane (Mumbai Metropolitan Region) to Bharuch). NH-48 traverses' hilly terrains, mudflats, zones of good residual soils and rock, and expansive soils, all within those 310km.

Each type of terrain poses its unique challenges. Roads along the coastline of India majorly traverse through mudflats. The groundwater table is high, and the lands are susceptable to tidal flooding. The upper strata of mudflats are invariably very soft to soft soils of high plasticity, with very low shear strengths and have tendencies to undergo large settlements over time with sustained loads such as from embankments and reclamations. As the weak plastic subsoil consolidates, it develops shear strength. However, to develop shear strength which is adequate enough to carry the load of the embankment structure, the structure will have to be constructed in judiciously calculated stages. Extended time for construction to allows for build-up of shear strength may not always be amenable to project economics, even if the consolidation process is accelerated by methods such as prefabricated vertical drains (PVDs).

THE EMBANKMENT

The embankment makes up for the difference in levels between the bottom of the pavement system and natural (dressed) ground to maintain the required top level of the pavement. It also spreads the loads from the pavement such that the stresses at ground (subgrade) level are within sustainable limits. The embankment also maintains the pavement system from predicted flooding and tidal waters.

The embankment, whether it is a conventional trapezoidal earth structure or a reinforced soil structure, needs to be checked for its structural integrity (stability) as well as deflections /

deformations (serviceability). These two aspects must be holistically considered in conjunction with the supporting foundation subsoil, untreated or treated.

Various methods of subsoil treatment for embankments are shown in Fig. EM-1(a) to (g). These procedures may be used appropriately, separately or in conjunction with each other. The Lecture Paper in particular addresses construction of the structure over untreated, weak subsoils. The subsoil may be improved to a required condition by any method or combination of methods shown in Figs EM-1 (a) to (f), The safety factor is further raised to the required magnitude with basal reinforcement as shown schematically in Fig EM-1(g). Basal reinforcement is an essential component where the embankment is supported on piles or stone columns to safely transfer loads to the piles or stone columns.

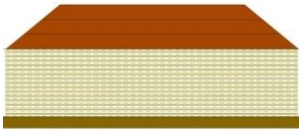


Fig. EM-1(a): Stage construction

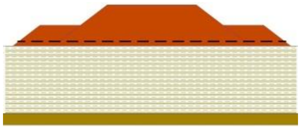


Fig. EM-1(b): Berms

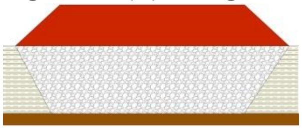


Fig. EM-1©: Full replacement

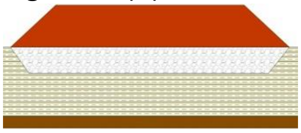


Fig. EM-1(d): Partial replacement

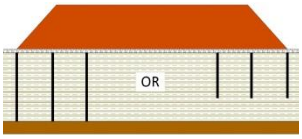


Fig. EM-1©: Prefabricated vertical drains

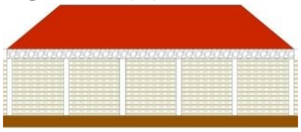


Fig. EM-1(f): Stone columns

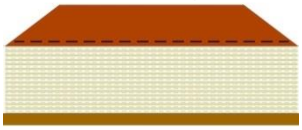


Fig. EM-1(g): Basal reinforcement

Fig. EM-1: Methods of constructing embankments over weak soils

As explained later, the purpose of treating the subsoil prior to application of basal reinforcement is to enhance the safety factor against global shear failure to 1 or more. The basal reinforcement would enhance the safety factor equal to, or beyond the required safety factor.

While basal reinforcement is considered for global stability of the embankment and subsoil, it also reinforces the earth structure against lateral slide.

TYPICAL **EMBANKMENT** FAILURES

An embankment on weak subsoil is susceptible to the following modes of failure:

1. Foundation bearing capacity failure.

2. Lateral sliding of embankment.
3. Global failure.

A single solution with geosynthetic reinforcement may cater to two or all three potential failure modes.

In addition, settlements during the service life of the embankment structure should be within limits permissible to the application.

FOUNDATION BEARING CAPACITY FAILURE

Like in any structure particularly founded on weak soils, it is essential to check the adequacy of ultimate (and corresponding safe) bearing capacity of the foundation of the embankment. This will not be confined to shear failures from embankment loads; short-term and long-term settlements also need to be reviewed to check for serviceability.

In case of bearing capacity inadequacies, there are basically two solutions:

1. Providing a basal reinforcement to act as a rigid layer to spread the load from the embankment.
2. Improving the shear strength of the weak soil below by consolidation.

Both solutions may be adapted simultaneously to advantage. An embankment with a rigid basal reinforcement layer may be considered equivalent to a footing with a rough base on weak soil (Almeida *et al* – 2013). Notwithstanding using reinforcement at the base, it is recommended that the bearing capacity considering the embankment without the reinforced basal layer be only marginally lower than, or equal to, or higher than the permissible stress of the underlying strata so that the factor is safety of the unreinforced embankment is at least marginally lower than, or equal to one, or higher. The basal reinforced layer is required only to increase the factor of safety to the specified magnitude.

Regarding settlements, the earth embankments, both unreinforced and reinforced are flexible structures and can tolerate differential settlements better than a rigid concrete or masonry structure. Even then, service conditions will require limiting these settlements during operations, and major percentage (preferably 90%) time related (consolidation) settlements are best taken place during construction of the embankment. THIS is highlighted in the case study for embankments presented in this Section.

FAILURE DUE TO LATERAL SLIDING OF THE EMBANKMENT

This is an often-neglected check for both unreinforced as well as reinforced embankments. This is also relevant to embanks atop reinforced soil structures (partial walls). Forces that come to play in sliding are illustrated in Fig. EM-2 (Almeida *et a*l – 2013).

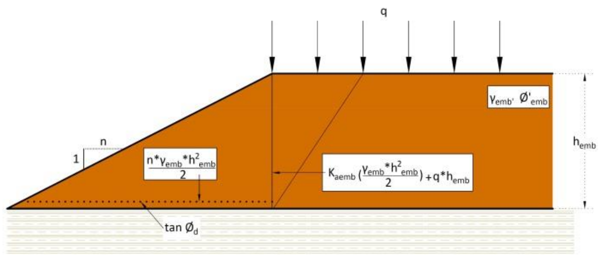


Fig. EM-2: Sliding in an embankment (Almeida *et al)*

With respect to Fig. EM-2, Equation (EM-1) and EQUATION (EM-2) give the factors of safety against lateral sliding for the two conditions of embankment base, i.e., unreinforced and reinforced respectively.

$$F_s = \frac{0.5 * n * \gamma_{emb} * h_{emb} * \tan\varphi_d}{K_{aemb}(0.5 * \gamma_{emb} * h_{emb} + q)} \tag{EM-1}$$

$$F_s = \frac{n * S_{clay} + T}{K_{aemb}(0.5 * \gamma_{emb} * h_{emb} + q)} \tag{EM-2}$$

where
n is the horizontal component of the slope ratio $1V\ : nH$;
K_{aemb} is the active earth pressure coefficient for the embankment material;
φ_d is the soil friction angle;
h_{emb} and γ_{emb} are height and material unit weight respectively of the embankment;
q is the surcharge on top of the embankment;
T is the long-term design strength of the geosynthetic reinforcement;
S_{clay} is the interactive shear resistance at the base; as a term, it is explained below.

Equation (EM-1) clearly underscores that the resisting force is by virtue of friction within the embankment material.

Equation (EM-2) highlights the interactive force between the underlying weak soil and the reinforcement, S_{clay}, and the long-term design strength of the reinforcement T. A word of caution here; as a matter of detailing in the design drawing, the designer is bound to place a nonwoven geotextile at the interface of the embankment and the underlying weak soil as a separator. Hence S_{clay} must be the force mobilised between the underlying weak soil and the geotextile separation layer.

Where geocells are concerned, total lateral resistance is by virtue of friction between the underlying soil and the infill material (if there is no separation geosynthetic in between), plus the tensile characteristic of the geocell material, symbolically T. If there is a separation layer, the preceding paragraph applies.

For geocells, the philosophy of considering T will be governed by the orientation of the geocells with respect to the embankment cross section:

1. When the straps of the geocell are oriented along the cross section of the embankment – Fig. EM-3(a), lateral forces from the embankment are transferred to the infill through friction, and the infill transfers these forces to the geocell expanded profile. Hence the

design tensile strength of the straps is to be considered. It is to be noted that the strap is parallel to the lateral force exerted by the embankment; and also, roughly at 45° at the most. Hence the resistance offered by the straps alone would be an average $0.85T$.
All cells of the geocells are infilled and the infill, being totally confined, will transfer the forces to the geocell straps. With the transfer of forces, the weld seam is stressed. Hence the weld seam strength is also significant.

T needs to be checked not only with respect to the design tensile strength of the perforated strap, but also for the weld seam peel strength. In this case, weld seam peel strength should be determined by “Method A” as per EN ISO 13426-1 “Geotextiles and geotextiles related products – Strength of internal structural junctions – Part 1: Geocells” The “Method A” style of testing is schematically shown in Fig. EM-3 (b).

1. When the straps of the geocell are oriented along the embankment longitudinal axis – Fig. EM3©, as in the previous case, lateral forces from the embankment are transferred to the infill through friction, and the infill transfers these forces to the geocell profile. However, in this case, tensile resistance from the straps will not be significant, and the lateral forces (other than the component resisted by friction between infill and underlying soil) will be resisted essentially by the geocell weld seams. In this case, weld seam peel strength should be determined by “Method B” as per EN ISO 13426-1 shown schematically in Fig.EM 3(b).

2. When the straps of the geocell are oriented along the embankment longitudinal axis as in Fig. EM-3©, as in the previous case, lateral forces from the embankment are transferred to the infill through friction, and the infill transfers these forces to the geocell profile. However, in this case, tensile resistance from the straps will not be significant, and the lateral forces (other than the component resisted by friction will be resisted essentially by the geocell weld seams. To simulate this case, weld seam peel strength is determined by “Method B” as per EN ISO 13426-1m schematically shown in Fig. EM-3 (d).

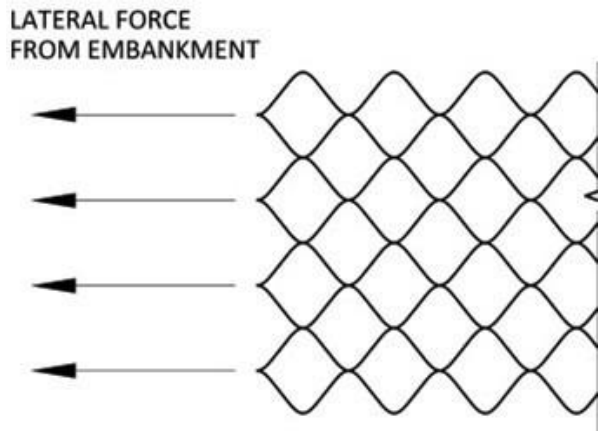


Fig. EM-3(a): Geocell straps along embankment lateral direction

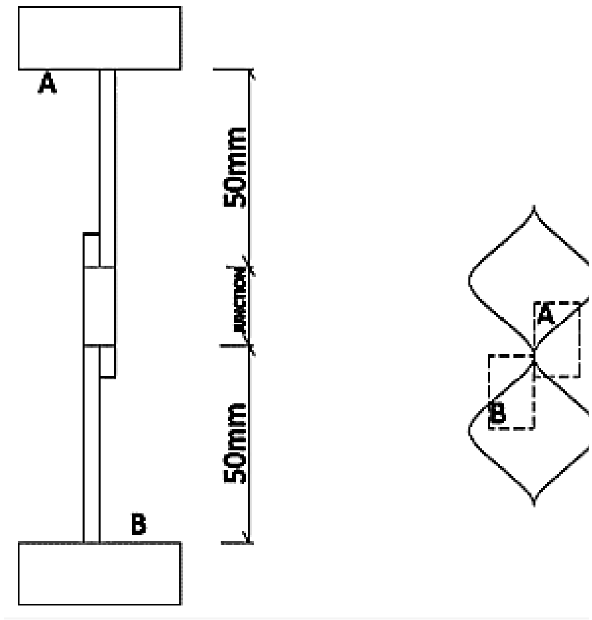


Fig. EM-3(b): Weld seam peel strength – "Method A"

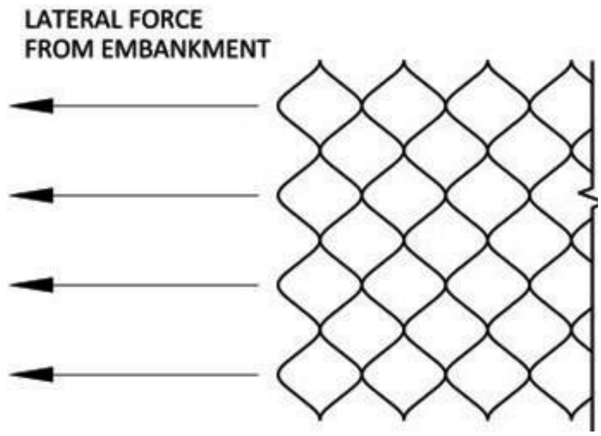


Fig. EM-3©: Geocell straps along embankment longitudinal direction

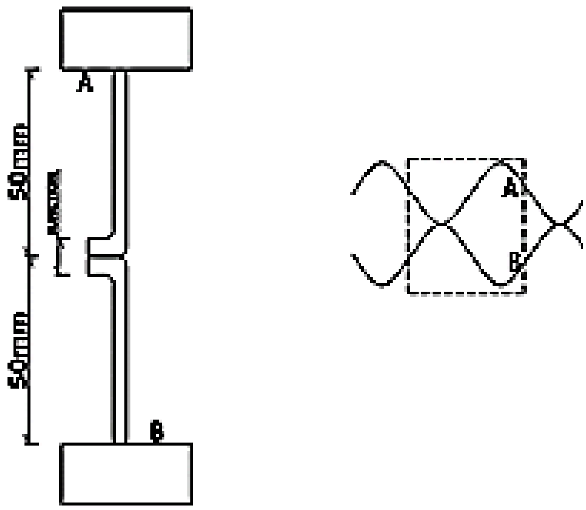


Fig. EM-3(d): Weld seam peel strength – "Method B"

It may be noted that stresses on the welded seams will be reduced owing to the confined in filling in all the cells. The reduction can be significant but difficult to determine at this juncture. Not much work has been concluded regarding geocells stressed along the panel plane in either direction. Further work is required through mathematical modelling, which will have to be backed up by experimental research, where appropriate instrumentation will be the key factor.

In case of reinforced soil structures with steep, near vertical sides (> 70^o), lateral stability is not an issue.

GLOBAL FAILURE

Several computer software's are available for global stability, also incorporating geosynthetic reinforcement within the embankment and along the base. In the case of embankments, Bishop's Method, considering circular slips would suffice.

GEOGRIDS AS BASAL REINFORCEMENT

Geogrids are essentially two dimensional with their major strength in one direction (uniaxial geogrids) or two orthogonal directions (biaxial geogrids). When uniaxial geogrids are used as basal reinforcement, the machine direction is invariably placed in the lateral direction of the embankment cross section.

The long-term design tensile strength T is considered as an additional factor as resisting force along the test shear surface.

It must be noted that the total tension to be taken by the geogrid must be a sum of the tension induced in the geogrid due to it providing lateral resistance *plus* the tension induced in the geogrid by the incipient shear failure surface determined by global failure analysis. Hence

$$T_{gr\ design} \geq T_{gr\ lateral} + T_{gr\ critical\ global} \qquad \text{(EM-3)}$$

where

$T_{gr\ design}$ is the long term design tensile strength of the geogrid;

$T_{gr\ lateral}$ is the tension developed in the geogrid due to resisting incipient lateral slide of the embankment for the required safety factor;

$T_{gr\ critical\ global}$ is the tension developed in the geogrid due to resisting incipient global shear failure of the embankment for the minimum safety factor as analysed (Fig. EM-4).

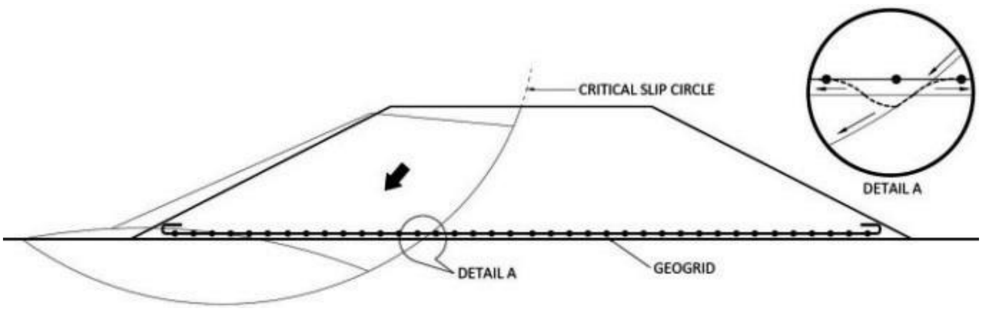


Fig. EM-4: Geogrid as basal reinforcement and critical slip circle

GEOCELLS AS BASAL REINFORCEMENT

When a soil structure is constructed on weak soil with three-dimensional geocells along the subgrade level as basal reinforcement, the slip surface will have to pass through the geocell reinforced section.

The mechanism for resisting shear failure when geocells are used as basal reinforcement is different from geogrids. Polymeric geogrids are two dimensional and are relatively flexible. Geocells are on the contrary, stiffer three-dimensional panels with the geocell straps fabricated upright, orthogonal to the plane of the panel. The stiffness furthermore increases with infilling with non-plastic soil. Hence geocell layers below the soil structure behave as a stratum with layer shear strength. Fig. EM-5 illustrates the global shear failure surface through the geocell layer.

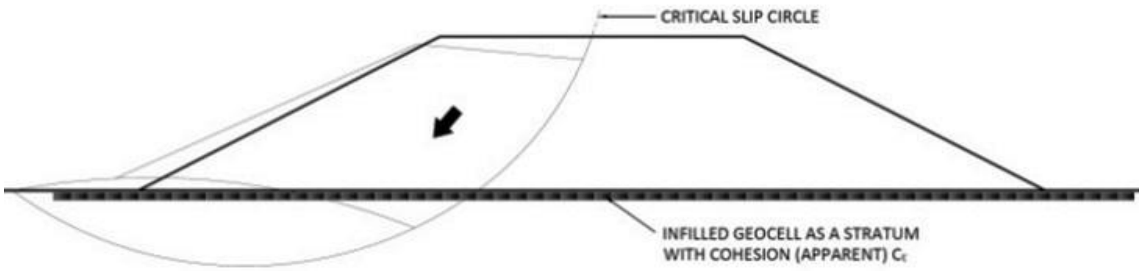


Fig. EM-5: Geocell as basal reinforcement, considered as a stratum with shear strength c_τ

As highlighted earlier and ever so often, the geocell is a three-dimensional geosynthetic material with interconnected curvilinear rhomboidal cells. The interconnected cells form a cellular confinement unit when expanded and infilled with well compacted non-plastic granular infill material. Geocells are manufactured using High Density Polyethylene (HDPE) material of grade which has a shear strength value of the order of 12Mpa. Geocell elements have a characteristic depth and an effective diameter which is a function of its weld spacing. Hence, in numerical analysis it is essential to consider geocells as three-dimensional structure and *should not* be modelled as two-dimensional.

Shear Parameters of Geocell Infill:

Apparent Cohesion and Friction Angle:

Bagli (Ref.2018) recommends an “apparent cohesion” parameter. This parameter is in addition to the angle of internal friction of the compacted non-plastic infill. “Apparent cohesion” has been derived by Bathurst and Rajagopal (1993) and it has been confirmed by laboratory tests that a geocell layer infilled with non-plastic soil can be considered as a stratum with an equivalent cohesion term c_τ (besides its friction angle φ), initiated by virtue of enhanced strength of the soil infill confined by the geocell walls. The geocell-infill system is best explained by the normal stress versus shear stress diagram in Fig. EM-6.

Fig. EM-6 illustrates the effect of lateral confining stress within the cell, $\Delta\sigma_3$ on confined soil to generate a larger circle, whose parallel tangent intercept is at c_τ on the shear stress axis; c_τ is also called "apparent cohesion". The slope of the tangent of the larger circle is the angle of internal friction φ.

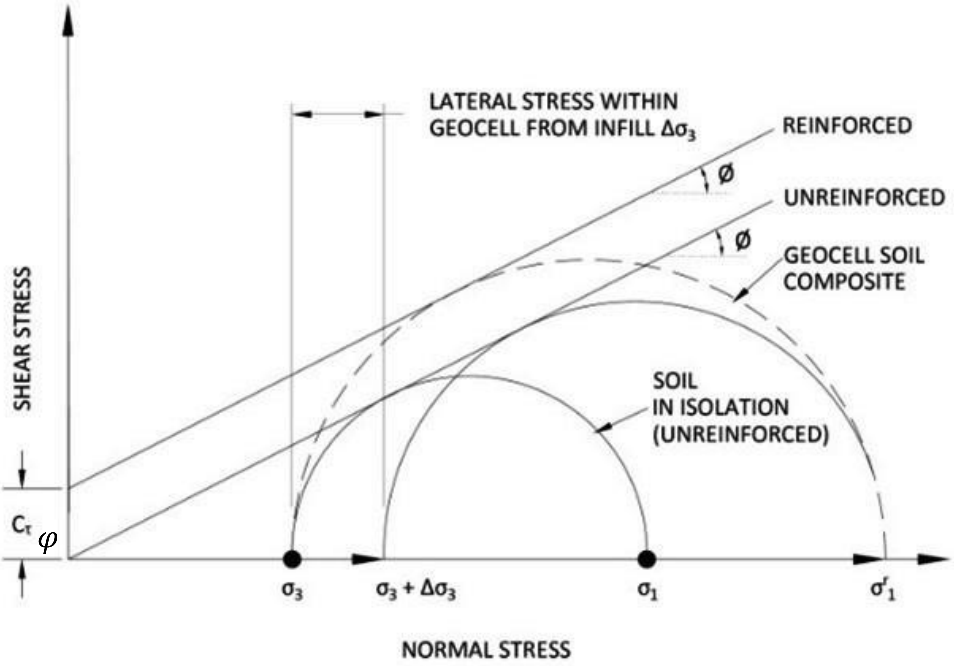


Fig. EM-6: Effect of cell confinement on non-plastic soil – "apparent cohesion" and friction angle

It can be derived from Fig. EM-6 that

$$c_\tau = \frac{\Delta\sigma_3}{2} * \tan\left(45° + \frac{\varphi}{2}\right) \tag{EM-4}$$

where

$\Delta\sigma_3$ is the lateral pressure under "at rest" condition acting on the geocell cell and is equal to $\sigma_1(1 - \sin\varphi)$;
σ_1 is the vertical stress on the infill within the cell;
φ is the angle of internal friction of the infilled (nonplastic) soil.

If *h* is the depth of the geocell, then over a unit length, the shear capacity of the geocell layer considering only "apparent cohesion is

$$\Psi_\tau = c_\tau . h \tag{EM-5}$$

Shear Resistance of HDPE Geocell Cell Walls

There is a need to assign a holistic value of shear resistance to the geocell layer. A significant component of shear resistance of a geocell layer is the shear resistance offered by the geocell

HDPE wall / strap. Hence the shear parameter of the geocell material needs to be evaluated for consideration.

Considering the slip circle in Fig. EM-5, the shear resistance offered by the straps of the geocell should be considered along with the "apparent cohesion" and angle of internal friction of the infilled soil. It is significant to note that "apparent cohesion" will be mobilised only as long as the confinement offered by the cells is intact. Hence the shear stresses in the HDPE straps must be lower in magnitude than the ultimate shear strength if the straps. This is invariably catered to when one considers the safety factors against global shear failure as greater than 1.0, of the order of 1.3 or 1.4 as mandated by standards. With these conditions, there will be no shear failure of the straps and the confinement essential to mobilise "apparent cohesion" will not be jeopardised.

The total shear resistance offered by the geocell reinforced soil layer can be considered as,

$$S_{Total} = S_{HDPE} + S_{Soil} \qquad \text{(EM-6)}$$

where,

S_{HDPE} is the shear resistance offered by the HDPE straps per unit embankment length

S_{Soil} is the shear strength of infilled soil i.e., angle of internal friction plus "apparent cohesion"

$$S_{Soil} = c_{\tau} + \sigma \tan \varphi_{\tau} \qquad \text{(EM-7)}$$

Yet another outlook is to combine the shear resistance offered by the HDPE straps and the "apparent cohesion" of the infill as a single entity as shown in Equation (EM-9). This is to facilitate design analyses using commonly available slope stability analysis software.

Literature survey and in-house laboratory tests indicate that the shear strength of virgin HDPE can be as high as 33Mpa, based on a crushing shear of cylindrical samples.

However, one requires to determine the direct shear strength of the drawn and textured virgin HDPE strap. The effect of perforations and their pattern are considered thereafter. Strata has developed and fabricated an equipment to determine the shear strength of the virgin HDPE geocell textured strap, described in the subsequent Section. The shear strength of HDPE material which is used for manufacturing geocells is of the order of 12Mpa.

The shear resistance contribution of HDPE material depends on following five significant aspects:

1. **Grade of HDPE material being used:**

   `This defines the basic shear characteristics of the material. Inferior quality of HDPE or hybrids cannot provide shear resistance as virgin high quality HDPE.

2. **Thickness of the straps:**

   The geocell strap thickness plays an important role to resist shear force. All other parameters being the same, a thicker strap will have the capacity to resist higher shear. For basal reinforcement applications for soil structures on weak soils, it is advisable to select a style of geocells with nominal thickness of the order of 1.6mm.

3. **Depth of geocell:**

   The depth of geocell panels *h* governs the length of the incipient failure surface through the geocell layer. Depth of the geocell will therefore influence the capacity of the geocell to mobilise shear resistance. Multiple geocell panels may be considered if the design so warrants.

4. **Perforations in the geocell strap:**

   Perforations are essential for the functioning of geocells, including facilitating relief of excess pore water pressures and to route water out of the system. The water may for instance, may seep out from prefabricated vertical drains (PVDs), in case PVDs are also adopted as part of the holistic design. However, perforations reduce shear resistance of geocell straps. While it is essential to limit perforations to 12% of the geocell wall area, it is required to reduce the shear resistance of each strap due to the perforations. A procedure to determine the shear resistance of the perforated strap is discussed below.

5. **Weld spacing of geocells:**

   This will govern the number of straps in the unit plan area of geocells in expanded condition. The closer the weld spacing, the greater will be the number of straps to contribute to shear resistance against shear failure. However, the cell aspect ratio should be as close as possible to 1.

Reduction of Shear Resistance of Geocell HDPE Strap due to Perforations

Fig. EM-7 shows a wall of a typical geocell of depth 150mm.

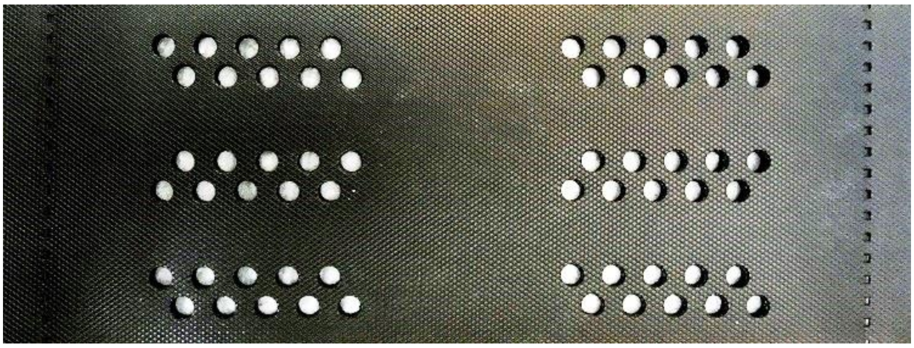


Fig.EM-7: Typical geocell strap of depth 150mm with its texturing, perforations, and seam welds at the ends

Perforations are the weak areas within the strap. The incipient shear failure path would be a curve, approximated to a line along which the shear resistance from the strap is the least. There are two facets to this:

1. The least length.
2. The maximum number of perforations along any line.

Both these need to be checked and the line that provides the least shear resistance shall be the governing shear resistance offered by a single strap for stability analysis.

Considering the typical 150mm strap shown in Fig. EM-7, the least vertical length is depicted by the yellow line as shown in Fig EM-8; vertical and passing through three perforations any which way in this case.

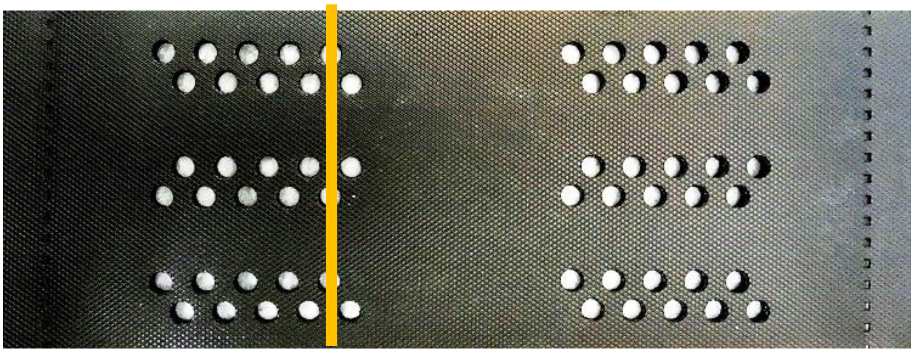


Fig.EM-8: The vertical path of least length along the example geocell strap

While considering the same strap, the other possible failure path could be along the yellow line as seen in Fig. EM-9, inclined but passing through the maximum possible number of perforations, in this case, six numbers. The path is longer than that in Fig. EM-8, but the number of perforations is double.

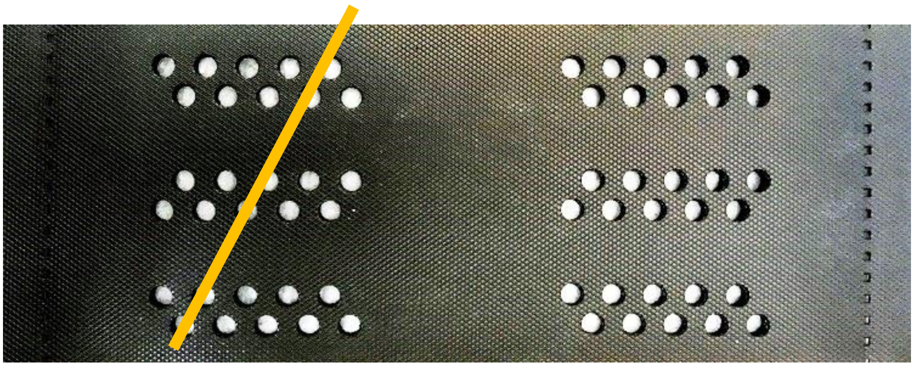


Fig. EM-9: The longer inclined path but traversing through maximum possible perforations

Computations need to be done for both cases and the least shear resistance obtained thus should be considered for calculating shear resistance of the HDPE geocell cell wall, T_{st}.

Computation of Shear Resistance of the Geocell HDPE strap

Based on all aspects relating to the HDPE strap and the perforations, shear resistance of the HDPE geocell cell wall, T_{st} , can be calculated in units of MN over unit length of the soil structure, but subject to the orientation of the geocell panels. If '*n*' number of straps are intercepted by an incipient shear failure surface being analysed per unit length of embankment, the shear resistance, S_{HDPER} of a single geocell layer will be

$$S_{HDPER} = n.T_{st} \tag{EM-8}$$

"Apparent cohesion" from Equation (EM-5) due to the infill is an add-on. Hence the total shear resistance of a single geocell layer is

$$\Omega_G = \Psi_\tau + S_{HDPER} \tag{EM-9}$$

Marginal soil has been used successfully as infill for geocells in road applications where the subgrade California Bearing Ratios (*CBR*) are low. The material may be locally available soils which are generally not used for pavement construction, or pond ash. When marginal soils are considered as infill for geocells as basal reinforcement, the value of "apparent cohesion" and angle of internal friction may not be significant enough to enhance the safety factors. However, including shear resistance of the geocell HDPE will show a significant improvement in safety factors. In passing it must be noted that while pond ash is often taken for granted as marginal material, it has a high consolidated, drained friction angle φ, as much as 38^o.

Factors Related to Shear Strength of Geocells

HDPE is a manufactured standardised product and no safety factor need to be applied to its shear strength, which may be considered from codal specifications or based on actual tests done on the straps. Analyses of general shear failure implies determining the factor of safety. Alternately factors may be applied to shear strength of HDPE straps, and to the "apparent cohesion" and angle of internal friction of infill, while applying the LRFD method. But in neither case, there will be failure or "collapse" of the confining cells, considering the safety factors also applied to the manufactured HDPE material under strict quality checks.

Strata Geosystems has developed a test equipment that determines the shear strength of a textured, unperforated geocell strap, besides determining Seam Peel Strength of strap welds. Initial tests on ten samples indicate average shear strength values of the order of 12Mpa. This value may be considered for design. For quality control purposes and to consider finer nuances such as texturing and tolerances of thickness, shear strength should be determined by tests with the strap shear test equipment. The straps may numerously be tested to arrive at a statistically correct value, taking into account the tolerances that may be allowed for the strap thickness.

Orientation of Geocell Straps

The significance of geocell strap direction vis a vis lateral stability has been discussed earlier and illustrated in Figs. EM-3(a) and (c). It was also noted that in the case of reinforced soil structures with steep, near vertical sides, lateral stability is not an issue.

Since the shear resistance of a geocell strap, (along with the "apparent cohesion" and angle of internal friction of the infill soil) is a contributor to shear resistance against activating forces, the strap orientation matters. The number of straps that are intercepted by the incipient shear failure surface per unit length (1m) is significant.

The shape of an individual cell of a geocell in plan is a curvilinear rhombus. Hence one diagonal is shorter than the other. It therefore stands to reason that if the longer diagonal, which is also the direction of the straps, is parallel to the soil structure cross section (and as a corollary, the shorter diagonal is along the longitudinal direction), there will be larger number of straps intercepted by the incipient slip surface within a unit longitudinal length of the embankment. Hence the direction of the straps parallel to the cross section of the soil structure is to advantage and should be adopted. The number of straps per unit length of the analysis model should be considered accordingly.

This is illustrated in Fig. EM-10 which diagrammatically shows the number of straps intercepted over a length along two orthogonal orientations of the geocell straps. Over a given length, there are seven straps intercepted across the straps, as against four straps intercepted along the strap direction.

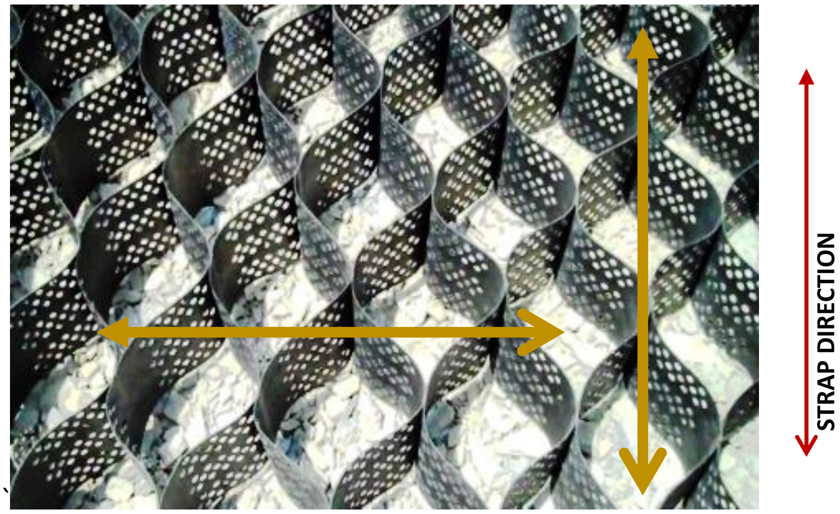


Fig. EM-10: Geocell straps that can be intercepted per unit length

The general shear analysis addresses shear resistance of the straps. Tension developed in the geocell system due to lateral deformations in case of a trapezoidal earth embankments may be considered separately and would not be additive to Eq (EM-8).

Shear strength of HDPE material used for manufacturing of geocells is obtained from laboratory tests as described in the following Section. From the test results, the average shear strength of HDPE used for manufacturing is 12MPa.

The style of geocells used is StrataWeb® 356-150 where the opening diagonal dimensions are 259mm x 224mm. Considering that the straps of the expanded panel are along the cross section of the embankment, approximately 8 straps are covered in 1m width in the direction perpendicular to the cross section of the embankment.

The thickness of each strap of geocell cell wall is 1.65mm.

The coverage ratio in plan per meter can be calculated as,

$$Coverage\ ratio\ of\ HDPE\ in\ plan = \frac{Strap\ Thickness \times No.of\ Straps}{Unit\ Width}$$

$$Coverage\ ratio\ of\ HDPE\ in\ plan = \frac{1.65 \times 8}{1000}$$

$$Coverage\ ratio\ of\ HDPE\ in\ plan = 0.0132$$

Hence,

$$Shear\ strength\ of\ HDPE\ per\ metre\ width\ (S_{HDPE}) = 12MPa\ \times 0.0132$$

$$S_{HDPE} = 0.1584\ MPa$$

i.e.

$$S_{HDPE} = 158.4\ kPa$$

Shear strength of confined soil considering "apparent cohesion" can be calculated as per Equation (EM-4) as,

$$c_r = \frac{\Delta\sigma_3}{2}\tan\left(\frac{\pi}{4} + \frac{\emptyset}{2}\right)$$

Considering an embankment of 4m height with soil properties:

Embankment **soil:**

Unit weight: 18 kN/m^3

Angle of internal friction = 36^o

Cohesion = 0kPa

Geocell infill soil:

Unit weight = 18kN/m^3

Angle of internal friction: 32^o

Cohesion = Apparent cohesion:

$$c_\tau = \frac{33.85}{2}\tan\left(\frac{\pi}{4} + \frac{32}{2}\right)$$

Hence

$$c_\tau = 30.5\ kPa$$

Hence, the geocell reinforced layer can be modelled as,

Geocell Reinforced Soil Layer –

Unit weight = 18kN/m^3

Cohesion = c_τ + S_{HDPE}

Cohesion = 30.5 + 158.4 = 188.9kPa

Angle of internal friction = 32^{o}

Properties of foundation weak soil:

Unit weight = 18kN/m3

Angle of internal friction = 0^{o}

Cohesion = 19.6kPa (corresponding to SPT N = 3)

Analysis of unreinforced and basal reinforced embankment on weak ground are carried out in a slope stability software, ReSSA (Reinforced Slope Stability Analysis). ReSSA is commonly used slope stability software, and any alternative software with Bishop's slip circle method can be used.

Analysis of embankment on the weak soils:

Geometry:

Embankment height = 4m

Side slopes = 1V:2H

Embankment top width = 14m

Live Load Surcharge = 20kPa

Factor of safety without any reinforcement in static condition is 1.14 as shown in Fig. EM-11.

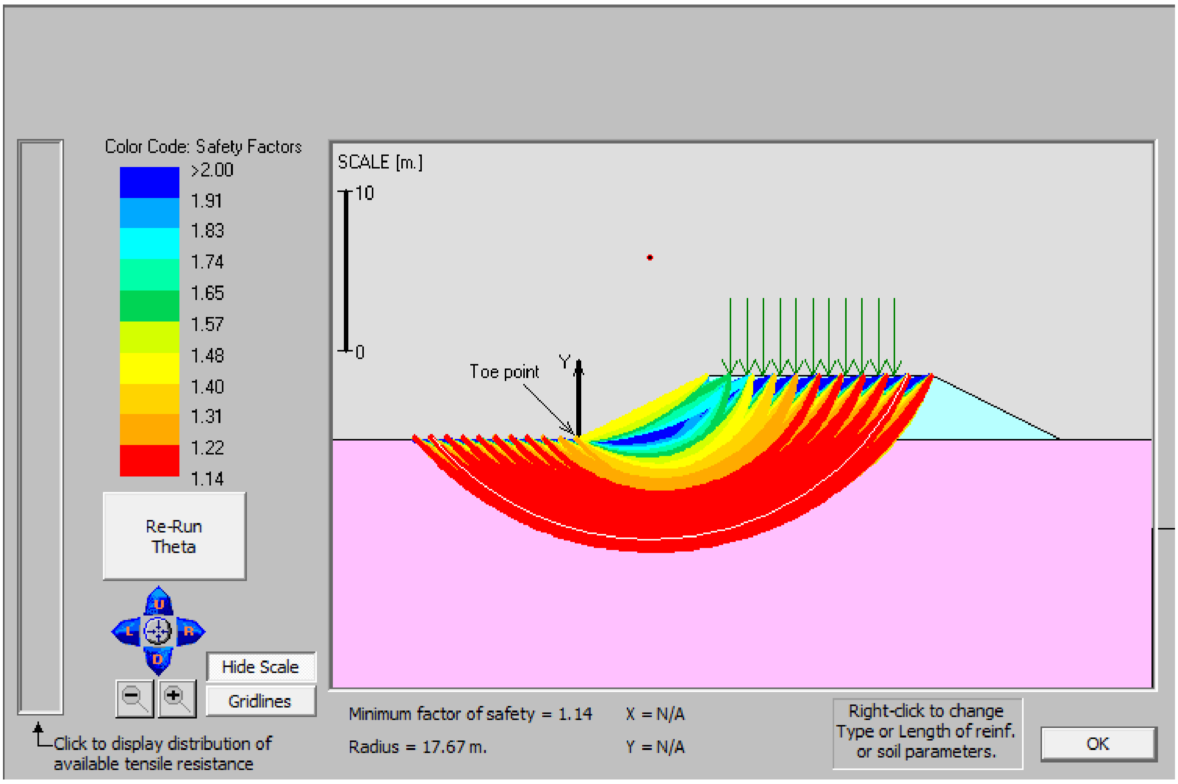


Fig. EM-11. Slope stability analysis of unreinforced section

Considering the shear strength parameters of geocell reinforced system, analysis has been carried out by providing one layer of 150mm depth of geocell (356-150) at the base of the embankment covering full base width of embankment. Two additional layers of approximately 1/3rd of the base width is added below the first layer at a location (middle 1/3rd) so that it captures all the slip circles with safety factors less than 1.4 (a safety factor recommended by IRC 113) to ensure a minimum factor of safety of 1.4.

These additional layers are provided to cover only 1/3rd of the entire base width for optimization in design and cost of the basal reinforcement system, since providing 3 layers over the entire base width would give no additional advantage for additional cost of material, excavation, and construction time. The curtailed layers of reinforcement are placed at the bottom to optimize the cost of excavation and dewatering, and ease of construction.

The slope stability analysis of reinforced section with critical slip circle factor of safety as 1.4 is shown in Fig. EM-12.

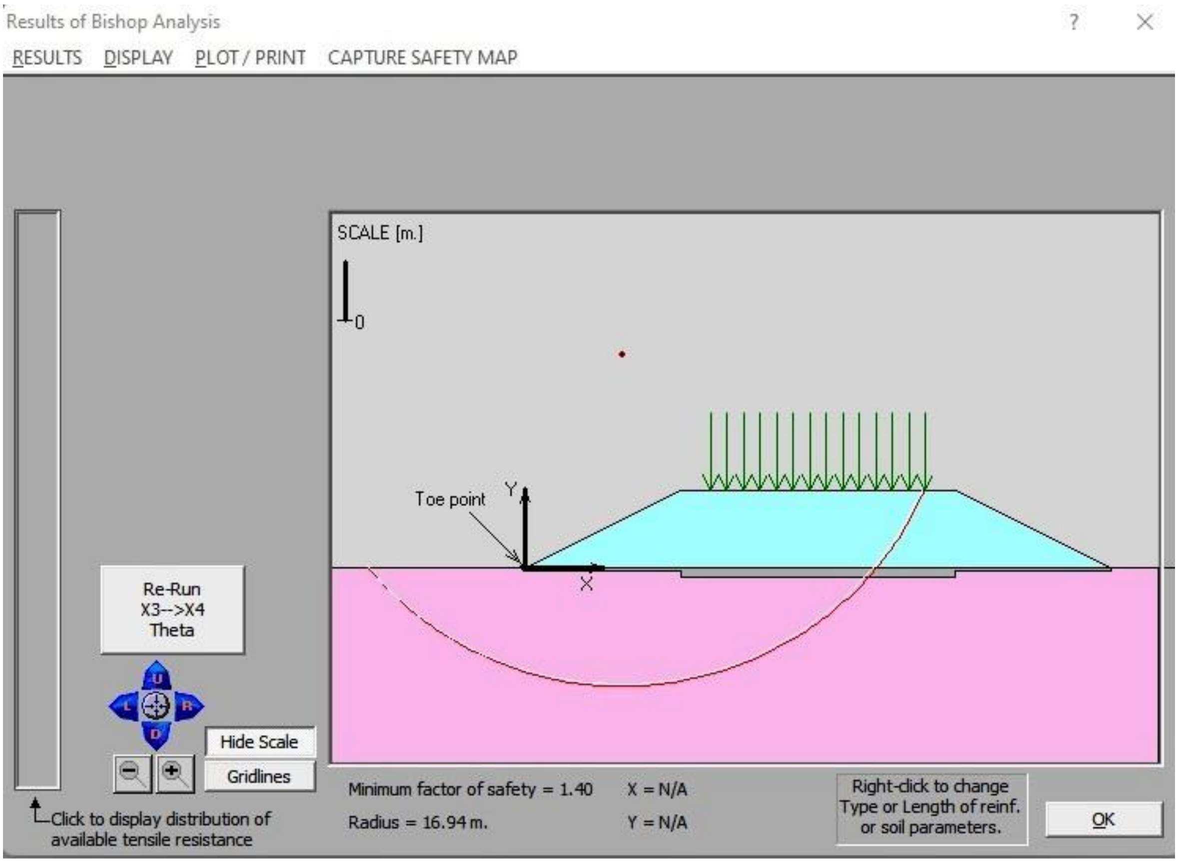


Fig. EM-12. Slope stability analysis of reinforced section - Critical slip circle with safety factor of 1.4

Fig. EM-13 shows a map of the slip circles with the corresponding safety factors for the reinforced section analyzed.

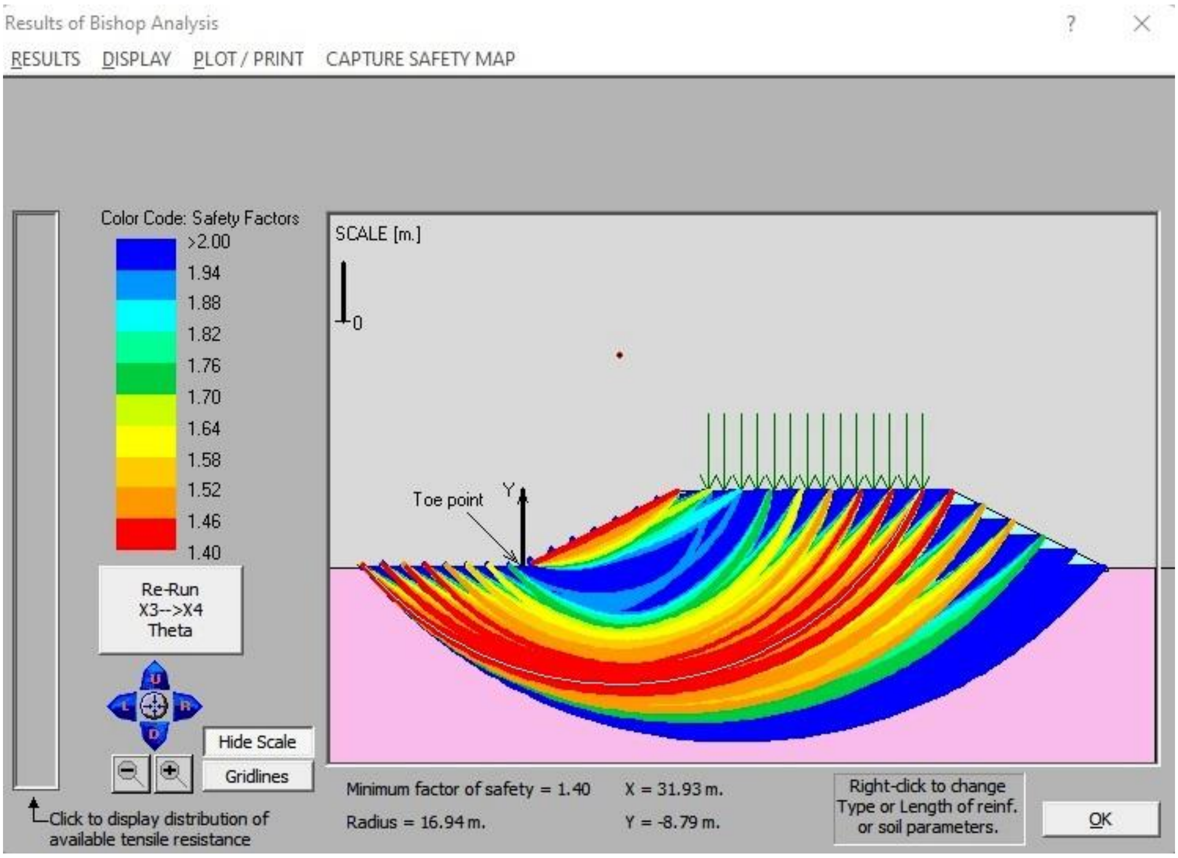


Fig. EM-13. Safety circle map with the geocell reinforcement layers

Fig. EM-14(a) shows the arrangement of geocell layers in cross section. Fig. 14 (b) shows the geocell arrangement in detail.

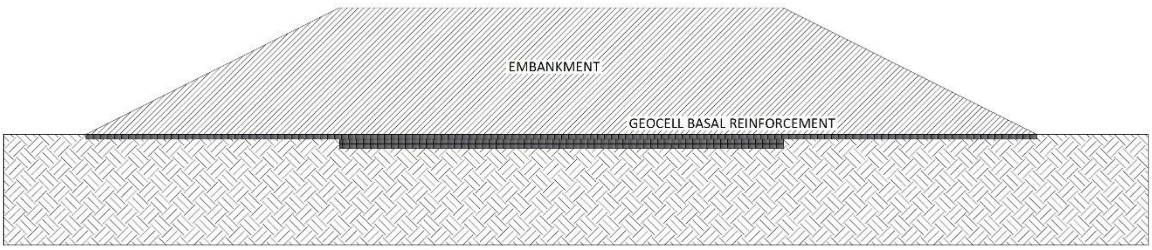


Fig. EM-14(a): Arrangement of geocells

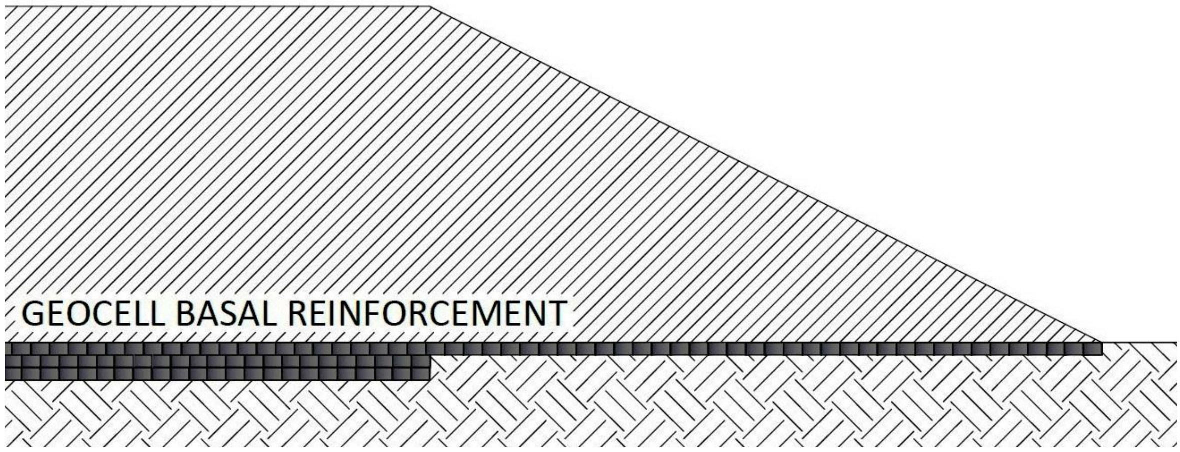


Fig. EM-14(b): Detail of arrangement of geocells

Fig. EM-14: Arrangement of geocells (1 layer of 356-150 over entire base and 2 curtailed layers of 356-150)

CONCLUSIONS

Geogrids and geocells are used as basal reinforcement for lateral stability and stability against global failure.

If differential settlements along the longitudinal length of the embankment / reinforced soil system are not significant, uniaxial geogrids may be used with the machine direction of the geogrids along the transverse direction of the embankment / reinforced soil system. If differential settlements in the longitudinal direction are significant, appropriate biaxial geogrids are recommended. Alternately, two layers of uniaxial geogrids may be considered. In this case, the upper layer of geogrid should be placed with the machine direction along the transverse direction of the earth structure. The lower layer of the geogrid should be placed with the machine direction along the longitudinal direction of the earth structure.

Geocell panels, particularly designed and manufactured as load bearing systems, have been successfully used as basal reinforcement for embankments and reinforced soil structures founded on weak soils. A simple method that evaluates the shear resistance offered by geocell layer(s) has been recommended, to be considered during general / global shear failure analyses of soil structures.

The take-aways of this Section include the following:

1. Conventional trapezoidal earth embankments need to be studied for lateral stability with appropriate geosynthetic reinforcement.
2. There is a well-defined method to consider shear resistance offered by geogrids and load bearing geocell layers.
3. Geocells are three-dimensional cellular geosynthetic systems infilled with non-plastic materials to include gravels, sand, non-plastic silts, and pond ash. Hence one can consider

both the “apparent cohesion” and angle of internal friction of the infill as well as the shear strength of the HDPE that the geocell is fabricated from.

4. When geocells as basal reinforcement are required to be infilled with marginal soils, the “apparent cohesion” will not be adequate enough to significant.
5. Depending on the requirements brought forth by lateral stability analysis and the global stability analyses, there may be a need to provide multiple layers. These layers should be appropriately located and dimensioned such that the slip circles with safety factors lower than the mandated value are taken care of, and the safety factor of the critical circle is higher than the mandated value.

SECTION V – B: APPARATUS TO DETERMINE SHEAR STRENGTH OF GEOCELL STRAP (PATENT PENDING)

(Code: AP)

Currently a major indeterminate in the design and application of geocells for load bearing as basal reinforcement is the limit of shear resistance of a geocell mat. The previous Section has highlighted how slip circle analysis can be carried out for checking global stability. There are two components of shear resistance of a geocell panel:

1. Shear parameters of the confined infill non-plastic soil, angle of internal friction φ and "apparent cohesion" c_r.
2. The shear strength of the cell walls, S_{HDPE} within the unit length of analysis.

The previous Section has explained how infill parameters are determined. However, the shear strength of HDPE is a material parameter which can be determined only through tests. One method is to determine the unconfined compression strength of a HDPE cylinder. However, the phenomenon of shear stresses on geocell straps when geocell panels are used as basal reinforcement is quite different from shearing of a cylinder. Tests should also account for texturing of the strap, while perforations can be considered separately as highlighted in the previous Section. Test evaluation of shear strength is also a quality control parameter at the manufacturing unit.

To obtain realistic values of shear strength of a textured HDPE strap used for fabricating geocell panels, an apparatus has been devised by the Strata Team along with the Author. The device is fabricated such that it can be used to determine the tensile strength of geosynthetics as well as determine Seam Peel Strengths by the four ISO Methods. The test for shear strength requires special grips, differing from the conventional. Fig. AP-1(a) shows the apparatus in shear test mode and Fig. AP-1(b) highlights grip details.

Fig. AP-1(a): The apparatus in shear test mode

Fig. AP-1(b): The grips

The geocell textured and unperforated strap is held vertically upright as seen gripped in Fig, AP -1(b). The right grip is fixed whereas the left grip is moveable vertically, linked with the geared motorised system. The two jaws of both the grips are lined with textured HDPE to ensure that there is no slippage between the HDPE and the jaws during the test. The HDPE strap is placed perfectly vertical, and the jaws are bolted tight. There is a gap between the moveable grip and the fixed grip, not exceeding 1mm.

The moveable grip is pulled upwards at a rate of 20mm per minute, the rate specified by ISO -13426 -1 for seam peel strength tests.

Fig AP-2 shows a typical failed strap sample.

Fig. AP-2: Typical shear failure of strap

Fig. AP-3 shows the print-out of the first series of tests conducted on ten samples of a single lot and is also a typical print out.

Fig AP3 also shows load – deformation curves, highlighting the Peak Strength of the elastic stage, the plastic stage, followed by failure. The peak load (stress) of the elastic stage defines the upper stress limit for geocells in basal reinforcement to ensure that the confinement of the infill is intact and “apparent cohesion can be mobilised.

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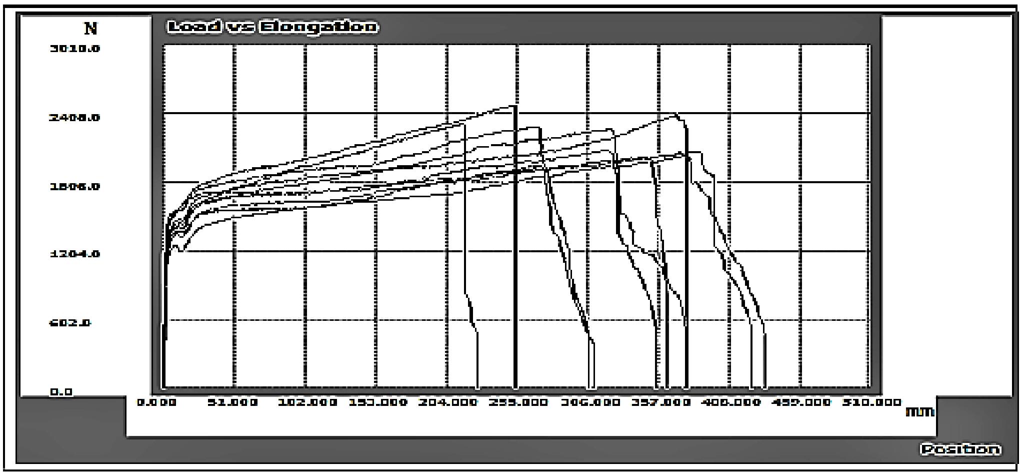
Pardi, Valsad, Gujarat

Report No.	210629/01D	Sample ID	SHEAR TEST SAMPLE
Sample Name	Shear Test	Thickness (average)	1.65mm
Lot No.	21S552	Test speed	20mm/min.
Width	100.00 mm	Mass per unit Area	1052gsm

Sr.No.	Peak Load (N)	Peak Elongation (mm)	Break Elongation (mm)
1	2020.00	343.80	363.40
2	2271.00	321.80	355.80
3	2387.00	369.60	377.40
4	2085.00	316.60	376.40
5	1982.00	266.40	309.80
6	2071.00	381.00	423.60
7	2036.00	375.80	434.20
8	2321.00	215.20	226.00
9	2287.00	264.20	307.40
10	2477.00	251.20	251.20
MIN	1982.00	215.20	226.00
MAX	2477.00	381.00	434.20
AVG	2193.70	310.56	342.52
STDEV	174.87	58.39	68.46
COEFVA	0.08	0.19	0.20



Tested by: NITESH

Checked by: SANDIP

Fig. AP3: Typical print-out of readings

The average elastic peak shear strength over several sets of samples works out to approximately 12Mpa. Unconfined compression strength tests on pipe grade HDPE rods with the HDPE of similar grade as that for the geocells, display an average shear strength of the order of 33Mpa. The magnitude of the difference justifies using the Strata apparatus to determine the shear strength of the geocell textured HDPE straps.

SECTION V – C: THE PAVEMENT COMPONENT OF ROADS:

(Code: PV)

PREAMBLE

The pavement is a structural entity which safely transfers cyclic stresses from various types of vehicles from the surface to the subgrade over its designated life, with vertical and horizontal deformations in its components within defined limits.

The pavement may be flexible or rigid; this Lecture Paper discusses the commonly adopted flexible pavement only.

The pavement surface is subject to vertical and horizontal stresses and these stresses develop strains in the various components of the pavement, which, when exceeded beyond certain magnitudes, can cause rutting, and surface fatigue cracks (Fig. PV-1). Horizontal stresses and corresponding strains are also generated by vehicle traction and other factors including temperature variations. The quality of the riding surface is a key factor to give comfort to driver and passengers. The pavement is designed to cater to these stresses and minimise strains over the designated life – Bagli S (2017).

Fig. PV-1: Rutting and fatigue cracks – Internet

While this Lecture Paper addresses both (flexible) pavements and embankments bearing the pavements, (earlier Section), the flexible pavement is also discussed separately. At the final stage of design, pavement and embankment may be holistically deliberated. However, it is quite often that pavements are supported directly on natural but dressed subgrade rather than an engineered embankment.

CHALLENGES IN DESIGNING AND CONSTRUCTING A PAVEMENT

The concept of sequential components that form flexible pavements has been followed since the days of Thomas Telford and John McAdam almost two centuries ago, with few modifications down the line. However, ` in the current-day scenario, sound natural material is scarce, costly and availability is much constrained owing to indisputably and justifiably strict environmental norms.

The flexible pavement often necessitates to be constructed from marginal materials. Besides this, considering the high cost of ground treatment, pavements directly supported on weak soils with low California Bearing Ratio (CBR) would require further engineering beyond the established and standardised norms. There are cases highlighted herein where marginal material is required to be used even where the CBR value is very low. These conditions require a paradigm change in the design of the pavement section using geosynthetics.

CONVENTIONAL PAVEMENT SECTION

The conventional concept of the pavement is shown in Fig. PV-2 – Zornberg (2012).

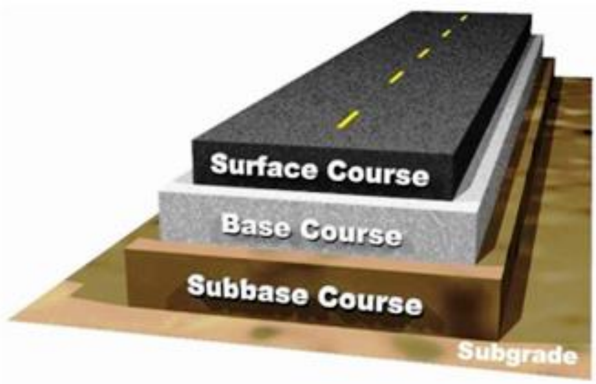


Fig. PV-2: Conceptual sketch of a conventional pavement (after Zornberg)

There are two common philosophies of design:

1. The “mechanistic-empirical” method, developed by the National Cooperative Highway Research Program (NCHRP), and also advocated by the Indian Roads Congress (IRC), IRC:SP:59-2019 “Guidelines for use of geosynthetics in road pavements and associated works”;
2. The American Association of State Highway and Transportation Officials (AASHTO) flexible pavement structural design.

The AASHTO method of design is based on pavement performance incorporating road-user’s definition of pavement failure, as well as the effect of structural parameters (such as component material properties and thicknesses), and the magnitude and frequency of the axle loads. The method uses empirical equations developed from road tests.

The mechanistic-empirical method considers the resilient modulus M_R of each component in the analysis, which is based on structural stress-strain concepts. The method uses mechanistic principles and detailed input data to minimise reliance on empirical observations and correlations. It improves design reliability, allows flexibility to reduce life cycle costs and provides economic solutions.

Determining the stresses and strains within the pavement components by the Finite Element Method (FEM) is trending. The advantage is that the modelling can consider boundaries between pavement components as elements and corresponding stresses and strains are compared with limits set for fatigue cracking and rutting. However, owing to high cost of required software, this method is not accessible for common usage as compared to the software commonly used for pavement analysis and design in India, viz. IITPAVE.

PAVEMENT SECTION INCORPORATING GEOSYNTHETICS

The essence of pavement design incorporating geosynthetics is to ensure that:

1. horizontal strain at the junction of the bituminous layers and the underlying granular component (Point A in Fig. Pv-3) is within defined limits.
2. vertical strain at the top of the subgrade (Point B in Fig. PV-3) is also within defined limits.

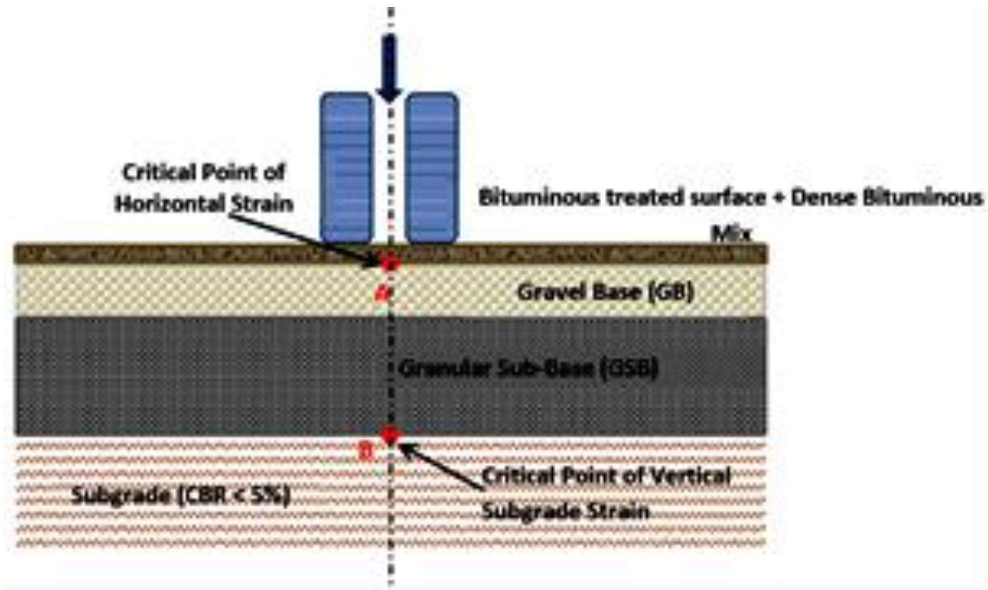


Fig. PV-3: Critical strain points in a pavement section – Bagli, S (2017)

On incorporating geosynthetics, the following one, or combinations of the following can be achieved:

1. Reduce the thickness of costlier pavement section component(s) particularly where the *CBR* is as low as 1%.

2. Use marginal materials within the pavement section without compromising on the performance or its life in terms of traffic.
3. Extend the life of the pavement in terms of number of million standard axles (msa) / equivalent single axle loads (ESAL), and also increase the time between maintenances.

Geosynthetics often considered for pavements are HDPE geocells and polyester or polypropylene biaxial geogrids, flexible or rigid.

Geosynthetics are generally placed nearest to the level of imposed load. This is often at the interface between the bituminous layers and the granular layers. Where poor CBR value of the subgrade is the prime concern, the geosynthetic is laid above the dressed and compacted subgrade. Figs. PV-4(a) and PV-4(b) show HDPE geocells and geogrids / geogrid composites in the pavement section below the bituminous layer.

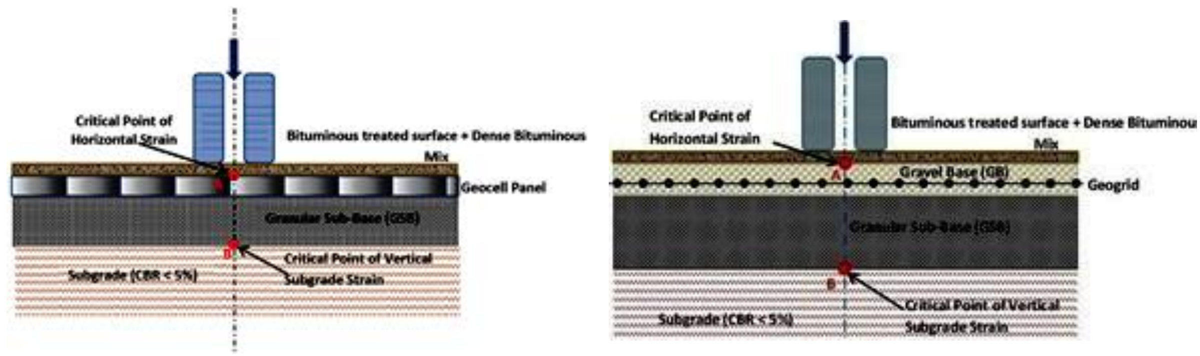


Fig. PV-4(a): Geocells in pavement section Fig. PV-4(b): Geogrids in pavement section

Fig. PV-4: Geosynthetics within the pavement section

The philosophies of design outlined above differ when designing for geocells and for geogrids. As a practice, pavements incorporating geocells are designed by the mechanistic-empirical method. Till recently in India, designs of pavements incorporating geogrids followed the AASHTO method, but are rapidly adopting the mechanistic-empirical method, a preferred method gaining popularity.

Components of the pavement section are modified if economies are to be affected. Thickness of the most expensive section is reduced first. Analysis is carried out with the geosynthetic in position and modulus M_R of the relevant pavement component, appropriately and judiciously modified considering the influence of the geosynthetic over the relevant portion. Horizontal tensile strains at Point A and vertical strains at Point B are thus evaluated and compared with the corresponding values at A and B in the conventional section, or the prescribed limits. The levels of strains in the modified section reinforced with the geosynthetic must be less than or just equal to those for the conventional section, or the prescribed limits, for the new section to be acceptable. The entire process would be by trial-and-error analysis.

If the life of the pavement is to be extended and the time for the next maintenance is to be prolonged, thicknesses of the various components are retained and the new life in terms of increased standard / equivalent axles is evaluated.

MECHANISTIC-EMPIRICAL METHOD

The Method evaluates the relevant strains at the two critical Points A and B in Fig. PV-3 and Fig. PV-4, by basic first principles. The new resilient modulus of that component of the pavement, within which the geosynthetic is integrated, must be evaluated, and judiciously and appropriately used in the analysis.

THE DAHELI TESTS PROPOSAL

A field parametric study has been proposed by the Author and Strata at the Strata Daheli Plant to determine the moduli variations considering geogrids and geocells within the layers. The subgrade would be a standard synthetic material expanded polystyrene) with a consistent value of a low *CBR*. This material will be of appropriate thickness, which will take due consideration of one set of plate load tests conducted directly on the material: plate size 300mm diameter.

The variables would include:

1. Styles of geocells and geogrids,
2. Type of layer and infill material, to include material which grades as wet mix Macadam (WMM), silty sand, gravelly sand, sandy gravel, and local residual soil.
3. For each geosynthetic type, varying thicknesses of the above geotechnical materials in order to assign modulus to each material and each layer thickness and determine a relationship between modulus for a base thickness and increasing thicknesses. This would also establish the postulate of Basu et al (2013) that the effect of geocells (the "zone of influence") extends beyond the thickness of geocell. The tests would relevantly study the variations in M_R of geogrids with various material thicknesses.

Unfortunately, constraints owing to the Chinese Virus pandemic have put a hold on this Study.

MECHANISTIC-EMPIRICAL METHOD – GEOCELLS

Adequate information is available on the improvement of the resilient modulus M_R of the pavement component with the inclusion of geocells in that component. The ratio of the improved resilient modulus as against the original (unreinforced) resilient modulus is known as the Modulus Improvement Factor (MIF). While it is necessary to conduct field and / or laboratory tests to determine the MIF for a particular style of geocell and the classification of layer and infill, extensive research has shown MIF to range anywhere between 2 and 3 and higher, depending essentially on the type of infill. According to Rajagopal *et al* (2011), the value may be of the order of 2.75 and Basu et al (2013) have stipulated *MIF* to be of the order of 2.5.

Regarding the "zone of influence" of the geocell (i.e., the thickness of the layer beyond the depth of the geocell) over which the MIF is effective, Basu *et al* (2013) have highlighted that owing to the confining effect of the geocells, there is further interlocking of the granular material above and below the geocell. According to Basu, the MIF is considered to be effective 25mm above the geocell and 20mm below the geocell within the granular material layer. The inclusion of these additional thicknesses above and below the geocell layer is left to the discretion of the designer.

As discussed earlier for load bearing geocells, Rajagopal *et al* (2011) have also stated, based on tests that the aspect ratio, i.e., depth to average diameter ratio of the geocell, should be as close as possible to 1.

As seen in the earlier Section IV, "The Macrolevel Mechanics of Load Bearing Geocells – Elastic Proposal", geocells invoke load spread. Hence if load tests are carried out on geocells in a laboratory using the standard 300mm diameter plate, the testbed must be adequately sized to allow that much load spread, to obtain factual results.

MECHANISTIC-EMPIRICAL METHOD – GEOGRIDS

In the case of geogrids, the MIF may similarly be determined by field and laboratory tests. The "zone of influence" of the geogrid reinforcement depends on several factors outlined below.

Tests with granular base course with geogrids demonstrated reinforcing effects typically approximately 30mm in thickness on either side of the geogrid – Schuettpelz et al (2003). The designer should therefore apply the MIF value to a layer 30mm above and 30mm below the geogrid, though judiciously, to the M_R of that pavement component.

Geogrids may be placed in more than one location within the pavement in the base and / or sub-base courses. There is also an opinion (also expressed through IRC:SP:59-2019) that the modified M_R would apply to the *entire* pavement component within which the geogrid is placed, a debatable point according to the Author, unless multiple layers of geogrid are judiciously placed in position, or tests are carried out on the relevant reinforced thicknesses to establish the corresponding M_R, as brought out earlier in this Section.

Due consideration needs to be given to parameters *viz.* geogrid flexibility / stiffness and aperture size, and particle size of the material being reinforced while considering the effective reinforced layer thickness.

It is advisable to conduct field load tests with the geogrid at the centre of the material layer. The tests should be repeated on various thicknesses of the material reinforced thus, to determine the *MIF* for respective thicknesses. Needless to state, baseline tests are essential for each thickness to determine the respective M_R *without* reinforcement, to determine the *MIF* for corresponding layer thickness with reinforcement. This was one set of tests during the proposed Daheli Tests.

Analysis to determine the critical strains at Points A and B is carried out by the KENPAVE software, or IITPAVE which is preferably used in India.

MECHANISTIC-EMPIRICAL METHOD CONSIDERING GEOSYNTHETIC LAYER COEFFICIENT RATIO

Conventionally, pavement design in India has been carried out by the mechanistic-empirical method, the procedure recommended by Indian Roads Congress (IRC), IRC:S[:59-2019 "Guidelines for use of geosynthetics in road pavements and associated works". Besides the CBR of the subgrade, two of the various inputs are M_R for the pavement components and MIF for the layer with the geosynthetic, where the zone of influence is judiciously considered.

A lot of work has been done in India to determine MIF for geocells infilled with various types of material. However, MIF is not commonly stated for geogrids in pavement design. Though on its way out, globally LCR for the respective style and branding of the geogrids are considered. Use of LCR is appropriate for the AASHTO method of pavement design but cannot be used in the mechanistic empirical method of analysis and design.

The AASHTO method considers the pavement section as a multi-layered elastic system. It introduces an empirical parameter, "Structure Number" SN which throws light on the total pavement thickness and its resilience to repeated traffic loads. SN is defined as:

$$SN = (a_1 d_1) + (a_2 d_2 m_2) + (a_3 d_3 m_3) \qquad \text{(PV-1)}$$

where

a_1, a_2, a_3 are structural layer coefficients reflecting resilient moduli of the bitumen layer, base course, and sub-base course respectively; determined empirically or from nomographs from AASHTO Guide for Design of Pavement Structures.

d_1, d_2, d_3 are thicknesses of the bitumen layer, base course, and sub-base course respectively.

m_2, m_3 are moisture modifiers for base course and sub-base course respectively and reflect the quality of drainage and the percentage of time that the pavement is exposed to near-saturation moisture levels.

SN is used in Equation (PV-2) below, a standard empirical relationship which determines the anticipated cumulative 18kip Equivalent Single Axle Loads (ESALs):

$$\boldsymbol{logW_{18} = Z_R * S_O + 9.36 * \log(SN+1) - 0.2 + \frac{log\frac{\Delta_{PSI}}{2.7}}{0.4 + \frac{1094}{(SN+1)^{5.19}}} + 2.32logM_R - 8.07} \qquad \text{(PV-2)}$$

where

Z_R is the standard normal deviate reliability level;

S_O is the overall standard deviation;

Δ_{PSI} is the allowable loss in serviceability;

M_R is the resilient modulus of the underlying subgrade.

SN is determined from Equation (PV-1); and the thicknesses of individual layers are determined through several iterations with due consideration to the relative costs of each layer and the respective minimum specified thickness, particularly of the surface bituminous course.

For designing with geogrids, a unique empirical number called the Layer Coefficient Ratio LCR is defined for a specific geogrid by the manufacturer. This number is a multiplier in Equation (PV-1) and is applicable for the pavement component within which the geogrid is placed. Hence Equation (PV-1) would be modified as:

$$SN_{GS} = (a_1 d_1) + (LCR_2 a_2 d_2 m_2) + (LCR_3 a_3 d_3 m_3) \quad \text{(PV-3)}$$

Where

SN_{GS} is the new Structural Number for the pavement incorporating the geogrid;

LCR_2 is the Layer Coefficient Ratio for the specific geogrid located in the base course, invariably greater than 1;

LCR_3 is the Layer Coefficient Ratio for the specific geogrid located in the sub-base course, invariably greater than 1.

SN_{GS} thus obtained is substituted for SN in Equation (PV-2).

LCR values are provided by the geosynthetic manufacturer based on exhaustive tests carried out through accredited laboratories.

LCR AND THE MECHANISTIC EMPIRICAL METHOD

Where it is essential to carry out analysis of a pavement section reinforced with geogrids, but by the mechanistic-empirical method, the Author recommends the following procedure for designing with geogrids whose LCR is specified – Bagli, S. (2017):

1. Resilient moduli M_{R2} and M_{R3} are evaluated for the base and sub-base respectively. The units are converted from kPa to psi (now to be considered as E_{BS} and E_{SB} for use in the AASHTO process as per the empirical Equation (PV-4) and Equation (PV-5) below to determine the structural layer coefficients a_2 and a_3.
2. The tensile horizontal and vertical strains are evaluated for the conventional section at the critical Points A and B for the given subgrade CBR and traffic / pavement life in terms of msa as per IRC:37-2018 "Guidelines for design of flexible pavements".
3. Structural layer coefficients are derived from the resilient moduli (in psi) using the following AASHTO equations:

$$a_2 = 0.249(log_{10} E_{BS}) - 0.977$$
(PV–4)

$$a_3 = 0.227(log_{10} E_{SB}) - 0.839$$
PV-5)

4. Consider the layer within which the geogrid is placed, base or sub-base, or both. Accordingly, the corresponding structural layer coefficient(s) is / are modified by multiplying by the corresponding Layer Coefficient Ratios. Hence:

   $LCR_2 a_2 = 0.249(log_{10} E_{BSGS}) - 0.977$

   or

   $$E_{BSGS} = 10^{\frac{(LCR_2 a_2)+0.977}{0.249}} \qquad \text{(PV-6)}$$

   $LCR_3 a_3 = 0.227(log_{10} E_{SBGS}) - 0.839$

   or

   $$E_{SBGS} = 10^{\frac{(LCR_3 a_3)+0.839}{0.227}} \qquad \text{(PV-7)}$$

5. From Equation (PV-6) and / or Equation (PV-7), E_{BSGS} and / or E_{SBGS} are evaluated, and the values are converted from psi units to kPa units to arrive at M_{R2GS} and / or M_{R3GS}.
6. M_{R2GS} and / or M_{R3GS} are then used to determine the reduced thicknesses of the pavement components, or the msa with the geogrids within the pavement component(s) by the mechanistic-empirical method. As in the case of geocells, the increased modulus is applicable only to the reinforced zone of influence of the geogrid.
7. Till tests prove otherwise, the improved M_R due to incorporating geogrids should be confined to 30mm above and 30mm below the reinforcement as discussed earlier. As per the Author, the effect of M_{R2GS} and / or M_{R3GS} should therefore be only to the extent of about 60mm.

MARGINAL SOIL ZONES ALONG WITH LOW CBR SUBGRADES

There can be conditions when roads are required to be constructed through regions where the borrow soils available are marginal, and the subgrade is weak. Roads may have to be constructed on an emergency basis through such areas. Pavements on such roads are often required to carry heavy vehicular traffic, for example along the country's West and North borders.

Three case studies with marginal soils are presented where geocells have been successfully used. In all these three cases, the pavements in unpaved conditions carried heavy vehicular equipment. Two of these cases were rigorous field trials to check the efficacy of geocells.

CASE STUDY 1: DESERT SILTY FINE SANDS

Trials conducted in the deserts of Rajasthan in silty fine sand, using HDPE perforated and textured geocells of depth 150mm and weld spacing of 356mm. Trials were carried out to check out a rapidly constructed pathway for tracked heavy vehicles. The is silty fine sand is non-plastic and considered as marginal for the application of an unpaved road. The CBR value is low, less than 5%. The material also tends to raise dust due to traction of vehicles which is not desirable in the border areas.

For the trials, the surface soil was roughly dressed and HDPE geocells were laid out manually as seen in Fig. PV-5(a). Sand from the surrounding area was filled into the expanded geocell panels with a front loader, Fig. PV-5(b). Considering the need for rapid deployment, compaction was done by the bucket of the front loader and the movement of the loader itself. The completed track was rigorously tested with heavy vehicles as seen in Fig. PV-5(c). Speed could also be achieved over the unpaved surface which otherwise was not possible. A notable feature was that despite speed, owing to material confinement, scarcely any dust was raised with tracked vehicle movement.

Fig. PV-5(a): Laying the geocell panel

Fig. PV-5(b): Infilling geocell panels with local sand

Fig. PV-5©: Unpaved track through several trial runs

Fig. PV-5: Trial in the Rajasthan Desert

Case Study 2: Silty Fine Sands with High Ground Water Level

Where there is silty fine sand subgrade and the ground water table is high, the subgrade has a low CBR value, often <3%, rendering it difficult to ply light, let alone heavy vehicles. In this case study, in the owner's parlance, the soil was termed "boggy". Pictures in Fig. PV-6 show how tracks of heavy equipment rutted deep into the unpaved subgrade, virtually demonstrating quicksand condition.

Fig. PV-6: Heavy tracked vehicle rutting deep into the unpaved subgrade; the pictures highlight depth of ruts

As in the previous case, pathways for heavy vehicles through marshy areas are required impromptu, using locally available material. Trials were proposed by the owner considering HDPE geocells. Owing to quicksand conditions and since such pathways are to be unpaved to prevent aerial and electronic detection, geocells of depth 300mm and weld spacing of 356mm were proposed. The geocells were perforated and textured. The perforations, very essential here, ensured rapid dissipation of pore water pressures from within the geocells. The unpaved pathway was thus created and as seen in Figs PV-7(a), PV-7(b) and PV-7(c) proved that geocells can be reliably deployed where subgrade conditions are difficult, even for heavy traffic and high axle loads and for rapid deployment.

Fig. PV-7(a): Unpaved geocell reinforced path with tracked 50T vehicle after several passes

Fig. PV-7(b): Unpaved geocell reinforced path with wheeled loaded vehicle after several passes

Fig. PV-7©: Comparison between the unreinforced, unpaved pathway (lower portion of the picture) and reinforced, unpaved pathway (upper portion of the picture)

Fig. PV-7: Case in marshy subgrade

Case Study 3: Highly Plastic Clay in Subgrade with Low CBR - Bagli, (2017

The Indian National Highway NH-44 near Churaibari on the Assam side of the Assam-Tripura border traverses forested and undulating terrain. The subgrade of the highway is highly plastic, weak clayey soil of low permeability. This region experienced exceptionally heavy rainfall since March 2016 right into September 2016, which completely damaged about 500m of the stretch and reduced it to a swampy mass as seen in Fig. PV-8. Conventional repairs were unsuccessful and traffic disruption created a social and political crisis with commodities getting scarce and expensive in Tripura with every passing day.

Fig. PV-8: NH-44 at Churaibari in July 2016 before rehabilitation

The issues, particularly weak subgrade of very low permeability was addressed while designing a pavement section with geocells within the proposed pavement section. The $\boldsymbol{CBR}$ value was as low as 0.5%. Traffic intensity of 20msa was considered. A section was designed and adopted as in Fig. PV-9. Geocells of depth 150mm and weld spacing 356mm were recommended. A $\boldsymbol{MIF}$ of 2 was conservatively considered. Granular material was available within reasonable lead and used as infill which ensured drainage of ingressed water.

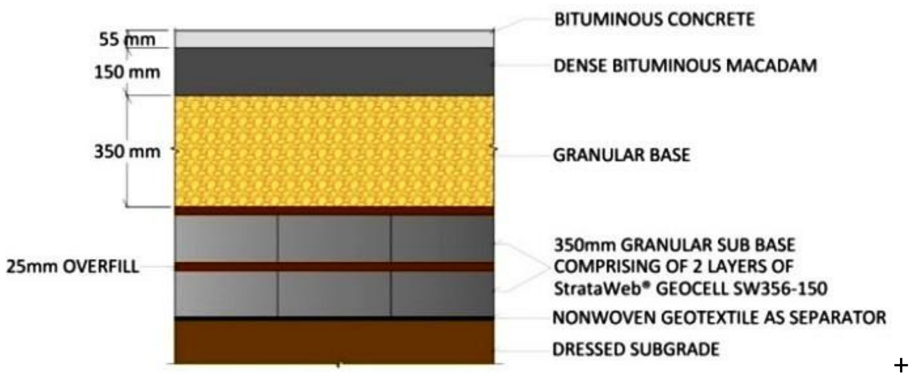


Fig. PV-9: The section

The pavement section was completed up to the 350mm granular base component by August 15, 2016, in a record time of 15 days from start of work notwithstanding wet weather conditions. Traffic was allowed to ply immediately on laying the compacted 350mm granular base component for several reasons, the major being:

1. The crisis did not merit any further delay of normal traffic flow.
2. It was not prudent to lay the top two bitumen components during the continuing rain and wet conditions.
3. Since the work was done under trying conditions, the behaviour of the road left unpaved was to be monitored and reviewed for a period under adverse weather conditions.

The procedure was successful, and Fig. PV-10 illustrates heavy vehicular traffic plying the unpaved pavement without any distress, notwithstanding wet conditions.

Fig. PV-10: Heavy trailers being hauled over the unpaved road in extreme weather conditions

This stretch of NH-44, after being paved has further withstood the heavy brunt of the 2017 monsoons also as can been seen in Fig. PV11. The performance of the pavement to date is excellent.

Fig. PV11: The paved pavement in 2017, after the second spell of heavy monsoons

This case study has highlighted that a pavement can be rapidly reconstructed in an emergency situation, and yet can be considered for long term application. The case study highlights how the pavement can be used for all classes of traffic even up to the granular base level only without distress. The bituminous toppings can be constructed after weather conditions are more conducive and the surface is dry enough to receive the bitumen courses, without interruption to regular traffic.

Besides the above, geocells in pavements have several other advantages. These include the following:

1. Geocells have been proven successful when incorporated in pavements traversing expansive soils in the subgrade.
2. Pavement sections incorporating geocells are leaner in section and have proved to incur economies on several projects. The economics also lie in reduced life cycle costs owing to less maintenance and longer periods in between maintenances.
3. With the confinement effect of the geocells, the effective shear strength of the infill is improved. Hence locally available non-plastic material can be used.
4. Economies in design are also affected with the reduced consumption of raw material and reduction in project time owing to less material to be handled and worked upon.
5. Geocells can be stored and transported in flat conditions occupying little space, as compared to the volume being laid out with expanded geocell panels. Hence there is economy in transportation and storage.

SECTION V – D: CONCLUSIONS – GEOSYNTHETICS FOR ROADS

(Code: CNRD)

Geosynthetics are an important facet of highway engineering. This has been prompted by several inevitable factors that include:

1.. Rapidly depleting natural resources,

2.. Rigid environmental laws.

3.. There is a dire need to use materials, which at one point of time were considered as waste. A significant example is pond ash, a waste from coal fired power plants which is very useful for construction of embankments and earth structures with steep slopes along with geosynthetics as reinforcement. Pavements can be designed using waste materials such as blast furnace slag and pond ash with appropriate geosynthetic reinforcement and by observing certain construction precautions.

4.. Construction with geosynthetics does not require any skilled labour and generally does not require any specialised construction equipment.

5.. Geosynthetics and in particular, geocells can be utilised for rapid construction of emergency pavements as well as roads / pavements over difficult subgrades and terrain. The CBR value of the subgrade may be as low as 1% if not even lower, according to the case studies presented.

6.. Conventional ground improvement techniques are proving to be time consuming and costly. Application of appropriate geosynthetics into the system may not only reduce time of construction but also prove to be cost effective.

7.. Design of both components of the road, earth structures and pavements which have been designed and constructed with geosynthetics can ensure long life with minimum maintenance. Geosynthetics *per se* can have a design life of over 100 years.

8.. Analysis and design of flexible pavements with geogrids can also be carried out by the mechanistic-empirical method. However, the effective zone of the geogrid within the pavement component must be considered with great discretion.

9.. On considering maintenance and life cycle costs, earth structures and pavements with geosynthetic reinforcement score well over conventional designs.

10.. The design and economics of a road system with geosynthetics will need a holistic approach, considering the support structure, whether it is an earth embankment or a reinforced soil structure. Both design and economics need to consider short-term and long-term performance and requirements. In the choice of geosynthetic material, the relative effective performance is the prime consideration. As can be seen from the NH-44 case study, cost was a secondary criterion, whereas getting the highway functional at the earliest was the need of the hour.

SECTION VI: GEOSYNTHETICS AS LOAD BEARING SYSTEMS FOR RAILWAYS

(Code: RW)

PREAMBLE

Engineering of the railway permanent way is geotechnical oriented. The scope for geosynthetics is vast for railway applications. Geosynthetics go a long way, some of the applications including:

1.. Geocells: For slope erosion prevention.

2.. Geogrids and geocells, individually or jointly for embankment foundation stabilisation – as basal reinforcement.

3.. Geogrids and geocells, individually or jointly for load bearing applications, essentially for spreading the load over a wider area to reduce the bearing pressure on the underlying formation and thereby reducing deformations of the track surface.

4.. Save scarce natural resources.

5.. Enhancing safety of railway operations in general.

MINISTRY OF RAILWAYS / RDSO

The Ministry of Railways / RDSO have incorporated geosynthetics in "*Guidelines and Specifications for Design of Formation for Heavy Axle Load*", Report No. RDSO/2007/GE : 0014 dated November 2009. This document mentions the application of geocells and geogrids as basal reinforcement for embankments on weak soils.

Thereafter, the Ministry of Railways / RDSO have published "*Comprehensive Guidelines and Specifications for Railway Formation*" (Specification No. RDSO/2020/GE: IRS-0004 dated September 2020), which advocates the use of geosynthetics appropriately. While outlining the basic functions in general, the document highlights the for various applications for a railroad formation that include:

1.. Ground improvement with vertical band drains / prefabricated vertical drains (PVDs) and geotextile encapsulated stone columns,

2.. Drainage with geocomposites,

3.. Basal reinforcement for embankments on weak soils with geocells,

4.. Soil reinforcement with geogrids to reduce the thickness of the blanket layer and strengthening a weak formation during retrofitting.

On the basis of success at a bridge, the Ministry / RDSO have recently published *"Transition System on Approaches of Bridges*", Report No. GE: R-50 (Revision 1) dated July 2021, generally based on the Case Study presented in this Section.

The Ministry of Railways is developing more guidelines and specifications for more extensive applications of geosynthetics in railway formation designs.

US DEPARTMENT OF TRANSPORT, FEDERAL RAILROAD ADMINISTRATION

To study geocell based subgrade stabilising systems, the US Department of Transport, Federal Railroad Administration. The study was as part of a subgrade renewal project and included a study of track geometry degradation trending on *Amtrak's* NEC Main Line. The project specifically focused on resolving subgrade instability beneath two tracks in the area of the Oakington Road bridge, located in Havre de Grace, MD. The study was reported in the document *"Field Demonstration of Geocell Substructure Support System Under High-Speed Passenger Railroad Operations",* published by the US Department of Transport, Federal Railroad Administration.

The expectations of the proposed geocell system (Fig. Rw-1) were to:

1.. reduce the degradation of track geometry,

2.. reduce the ballast - subgrade interface pressures,

3.. increase surfacing cycle.

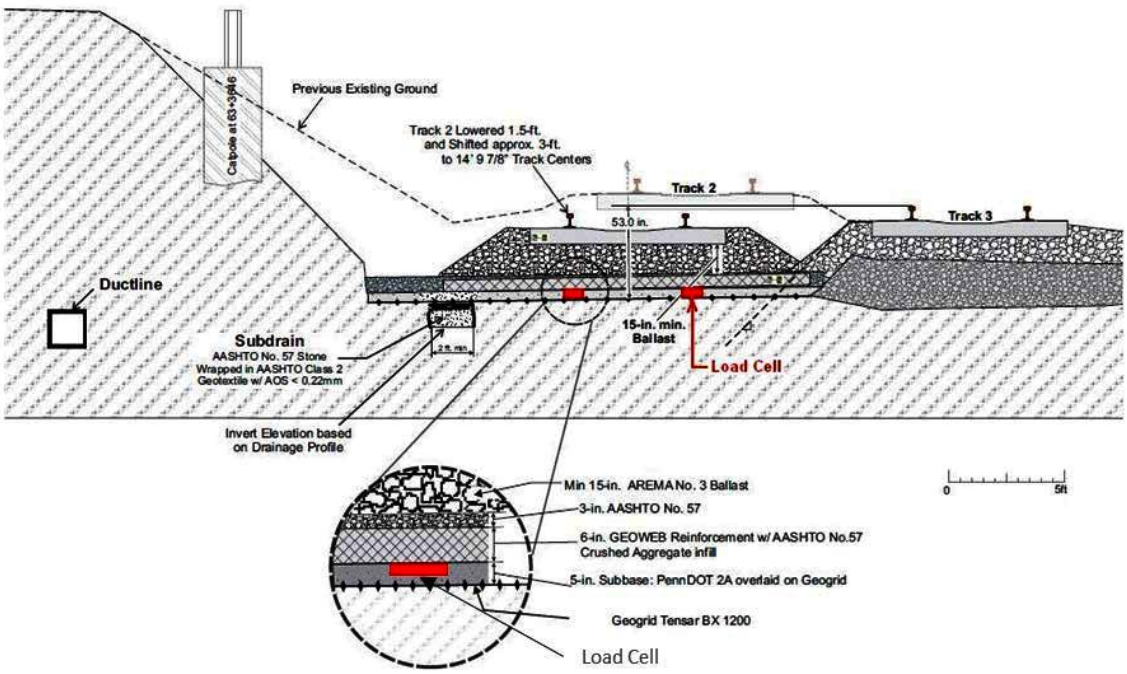


Fig. RW-1: Final design for geocell installation (after US Federal Railroad Administration)

A testing plan was developed to monitor and evaluate the geocell system:

1.. to determine if this system controls track geometry degradation:

2.. whether it is cost effective for the results achieved.

Pressure transducers were installed below the left and right rails at the ballast - sub-ballast interface both within (Fig. RW-1) and beyond the geocell reinforced section of the track. Pressure readings were monitored at defined intervals over a seven-month period after completion of re-installing the track / maintenance. In addition to pressure data, monthly track geometry data was also monitored and supplied by Amtrak and analysed for changes in the rate of track geometry degradation.

Analysis of the pressure and track geometry data showed that:

1.. The geocell system was very effective in minimising and controlling track geometry degradation. Analysis of the track condition, as defined by the Track Quality Index (TQI), showed that the geocell reinforced zone had a measurably reduced rate of degradation. This, in turn, translates into a corresponding extension of maintenance (surfacing) cycles.

2.. After installation, interface pressures within the geocell reinforced stretch were roughly half of the measurements taken beyond the geocell reinforced zone.

The study concluded that by utilising a geocell-based subgrade stabilising system on a high-speed track, showing significant historic geometry degradation, an increase in surfacing cycle by a factor of 6.7 can be realised. A cost benefit analysis was done to determine the return on investment (ROI) of installing the geocell system. For the current location, an ROI of nearly 113% was realised by installing the geocell system.

WESTERN RAILWAY CASE STUDY

PREAMBLE

The favourable results of the study carried out by the US Federal Railroad Administration at Oakington was, as a precedent, one of the factors that encouraged the Western Railway engineers to change the paradigm and consider a system of geocells to replace the conventional concrete approach slab at the transition from earth embankment to a rigid bridge structure.

For the reader passionate over railways and trains, this Case Study reflects how divisions of the Indian Railways function as a well-oiled synchronous machine with the resources at hand.

THE PROBLEM

Differential settlements between the earth embankment bridge approach just beyond the abutment and the bridge structure, often supported on piled foundations, are one of the major issues that limit the intended speed. This issue required a solution with ambitious plans on the anvil to increase the speed limits and hike up axle load to 32.5T from 25T. Differential settlements at the structure – embankment transitions manifest as rough running at bridge approaches. This also causes increased wear and tear of track, and stock wheels and bearings.

To ensure a smooth ride, an appropriate transition system is needed which provides a gradual change in the “spring” or deflection characteristics of the track bed from the earth embankment to the more rigid bridge structure. The proposed system should bridge over the differential settlements between the two structures. It would also help if the proposed system spread the track load over a wider area below the ballast and reduce the deformation of the embankment fill at entry to and exit from the bridge.

While devising a solution, it was assumed that the subgrade of the embankment had achieved its 90% consolidation and deflections below the subgrade were negligible. This assumption is quite valid for embankments that have been constructed several decades ago and well maintained according to Railway routine guidelines and true to the Indian Railway traditions.

THE SOLUTION

The Western Railway proposed using a system essentially comprising layers of geocells to considerably reduce deformations of the track surface / formation and maintain the track geometry at the bridge approaches. Before adopting the proposed system on a major scale, it was decided to carry out monitored trials on the busy and nigh-speed Bombay-Baroda route, near Surat.

It is significant to note that the proposal does away with the conventional concrete slab.

OBJECTIVES OF THE TRIAL

As per Ministry of Railways / RDSO Guidelines, approach slabs are to be provided on both bridge entry and exit approaches of the unballasted bridge deck with spans of 12.2m or more. One end of the approach slab is supported on the bridge abutment and the other end is resting on the formation. The approach slab should be minimum 4m long and of reinforced concrete.

However, the conventional style for approaches has the following limitations:

1.. Nonuniform settlements of the slab with movement of the track that it supports. The track movement Is due to several reasons including climatic variations, temperature variations over 24 hours and traction forces due to rolling friction, acceleration, and braking.

2.. Quite often, the approach slab is damaged and is not replaced in time owing to extent of work involved, adherence to maintenance schedules, etc.

The Western Railway desired to strengthen the bridge approaches to cater to an increased speed of 160kmph on the prime Mumbai-Delhi trunk route. The Railway engineers desired to consider geocells to overcome the limitations of the conventional approach slab and enhance the performance of the approaches.

LOCATION OF THE TRIALS

To save on downtime, abridge without the concrete approach slabs was desired. The bridge designated Major Br No. 417 across Mindhola River near Lajpore Village; Surat was selected. The bridge is located within the Sachin- NRL block section of VR- ST of the Railway. The bridge has thirteen plate girder spans, each 18.3m. The concrete approach slabs of this bridge were not yet replaced, pending proposed major maintenance. Hence the approaches of this bridge were deemed ideal for trials with geocells layers in lieu of concrete approaches. Fig. RW-2 shows the satellite imagery of the bridge location. Fig. RW--3 gives a close-up view of the imagery.

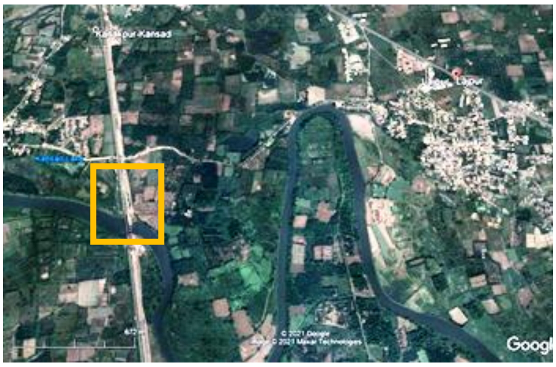


Fig. RW-2: Satellite imagery of Bridge 417(courtesy Google)

Fig. RW-3: Magnified image of the location within the rectangle

ORGANISING THE TRIALS

Two trial stretches were taken up at Bridge 417, on March 02, 2021, and March 16, 2021, respectively. The first trial stretch was taken up on the Up line (for South bound traffic towards Mumbai), at the North end. The second trial section was taken up on the same track, at the South end of the Bridge.

TRANSITION SYSTEM AT THE APPROACHES

The trials were carried out with three layers of the Indian make **StrataWeb®** geocells, depth 150mm and weld spacing 356mm, according to the design recommend by IIT Madras. The infill material is that of the blanketing as per G-14 specifications. A layer of nonwoven geotextile was placed below the three-layer geocell system, layer of geotextile to prevent loss of aggregates from the geocell pockets and to provide separation and good drainage.

The transition system with geocells, shown in Fig. RW-4 was approved by IIT Madras.

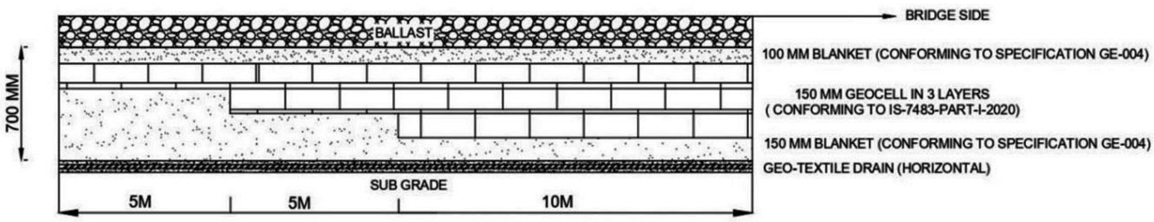


Fig. RW-4: Section for trial stretch recommended by IIT Madras (Prof K Rajagopal)

TRIAL PRE-REQUISITES AND PRE-BLOCK ACTIVITIES

Traffic on the Southbound Up Line was required to be blocked for adequate time to carry out the work safely and to quality parameters. Block on the track could only be secured over a limited time owing to heavy traffic on one of the country's busiest trunk routes. A block period of 4 hours 30 minutes was allocated to carry out the entire set of activities for the installation of the system at each trial stretch respectively. Considering the strict and limited time allocated for the work during the block, and the limited work-space constraints, the execution of activities during the block was required to be well-planned, coordinated and concerted. Hence prior to the commencement of the block, preparations were made to keep in readiness to commence work sequentially on the set of activities during the block for timely and quality installation under the space constraints.

Activities prior to the respective blocks included the following with traffic plying with a speed limit of 20kmph:

1. Removal of guard rail on approach.
2. Loosening fittings.
3. Making cuts in rails for easy removal.
4. Removing additional ballast from shoulder and filling into bags.
5. Locating and shifting signal and transmission cables at the approach by S&T Department.
6. Marking the existing rail levels on the OHE Mast.
7. Keeping in readiness all materials required including geocells, nonwoven geotextile, blanket material, adequate ballast, equipment, water, and manpower.

The work on the first stretch commenced on March 02, 2021, as scheduled. To ensure that all activities were completed within the allocated traffic and power block time (4hours 30 minutes), timings were allocated to each activity. This was also a Time Study exercise of sorts as basis for time allocation for similar works on other bridge approaches.

The activities with the respective allocated time durations are listed in Table: RW-1.

Table: RW-1: Time allocation for each activity for installing the geocell system

Sr. No.	Activity	Duration
1	Removal of Rails + Sleepers + Ballast over the 20m (approx.) stretch	45 minutes
2	Excavation to required depth	60 minutes
3	Installation of `geotextile and geocells including infilling and compaction	75 minutes
4	Placing of Ballast + Sleepers + Rail over the 20m (approx.) stretch to the original position	60 minutes
5	Spacing of sleepers and boxing with required compaction by the Dynamic Tamping Express Machine	30 minutes

CONSTRUCTION SEQUENCE

The construction sequence during the block was as follows:

1. The tracks were dismantled, and the rails were removed along with the sleepers and fittings (Figs. RW5, RW-,5 RW-6) and RW-7.

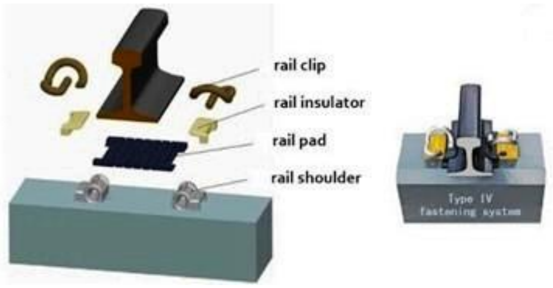


Fig. RW-5: Removal of rail fastening system (Courtesy Net)

Fig. RW-6 Removal of rails

Fig. RW-7: Removal of sleepers

2. The formation including ballast was excavated up to 1400mm below the rail level (Figs. RW-8 and RW-9).

Fig. RW-8: Removal of ballast

Fig. RW-9: Removal of sub-ballast / blanket material

3. Soil was excavated to reach 700mm depth of formation over the 20m length, and 5m width. The excavated area was levelled and compacted prior to laying nonwoven geotextile (Fig. RW-10).

Fig. RW-10: Checking the excavation level

Fig. RW-11: Laying the nonwoven geotextile

4. Prior to infilling back, the blanket material, nonwoven geotextile was placed over the entire excavated stretch measuring 20m x 5m (Fig. RW-4). According to RDSO guidelines. overlaps of the geotextile were minimum 300mm. Laying the nonwoven is shown in Fig. RW-11.

5. Blanket material was placed on top of the nonwoven geotextile. The blanket material was watered and hand-tamped to 100mm thickness (Fig. RW-4). The procedure is shown in Figs RW-12, RW-13, RW13, RW-14 and RW-15. A timber template was used to ensure that compacted thickness of 100mm was achieved.

Fig. RW-12: Infilling of 100mm thick blanket layer

Fig. RW-13: Watering the blanket material for compaction

Fig. RW-14: Compaction with manual hand tamper

Fig. RW-15: Compaction by manual hand tamper

6. The bottom-most layer of geocell StrataWeb® SW 356-150 (weld spacing 356mm and 150mm depth) was placed. The dimensions of this layer were 10m length x 5m width. The layer was placed on top of the 100mm thick compacted blanket.

   The geocell layer was held in expanded position by temporary steel hooked stakes. The stakes maintained the expanded panels before infilling and kept them in position. The stakes also ensured that the cells were not distorted and maintained their dimensional integrity.

   Blanket material was infilled within the geocells with a backhoe and spread manually with shovels. The infilled material was watered and compacted with a manual hand tamper to achieve the required compaction. The infilling of the geocell was done 25mm above the brim of the geocell as a cushioning layer to avoid damage to the brim of the geocells. The final compacted thickness of this layer was maintained as 175mm.

   This procedure is seen in Figs. RW-16, RW-17, and RW-18.

Fig. RW-16: Bottom-most geocell layer

Fig. RW-17: Infilling geocells

Fig. RW-18: Watering and compaction of bottom-most geocell layer

7. The middle layer of geocell, also StrataWeb® SW 356-150, was placed, with expanded overall dimensions **15m length x 5m width.** This layer was placed on top of the bottom-most layer of infilled geocells overfilled by 25mm). Placing of infill, spreading, and compaction were carried out similar to the procedure for the bottom-most geocell layer. This activity is illustrated in Fig. RW-19.

ig. RW-19: Laying and infilling of middle geocell layer

Fig. RW-20: Laying and infilling the topmost layer of geocells

8. The topmost geocell layer (also StrataWeb® SW 356-150) was similarly placed. The expanded dimensions of this layer are 20m length x 5m width. The Infill material was placed, spread, and compacted after watering, similar to the lower two geocell layers. The placing of this layer is illustrated in Fig. RW-20.

9. The 100mm thick compacted blanket layer was provided over the uppermost geocell layer. Ballast was laid over the blanket as seen in Fig. RW-21.

Fig. RW-21: Laying ballast over the blanket layer

Fig. RW-22: Placing sleepers over ballast

10. Sleepers were placed back into position after laying and spreading the required thickness of ballast as seen in Fig. RW22.

11. Rails were placed back in position on the sleepers (Fig. RW-23) manually by sliding on rollers.

Fig. RW-23: Replacing rails on sleepers

Fig. RW-24: Placing rail rubber pads between rails and concrete sleepers

12. Rail rubber pads were placed between the steel rails and the concrete sleepers (Fig. RW-24).
    Rail pads are elastic polyurethane mats, provided essentially to protect sleepers. The pads cushion the effect of high-frequency impact of the steel rails due to high-speed locomotive and rolling stock. The pads also provide electrical insulation to information systems.

13. After placing rail rubber pads, adjacent rails were joined together by fish plates (rail joint bars or splice bars), as seen in Fig. RW-25.
    Function of a fish plates is to join rail ends to maintain line and level of the top table and the gauge face of the rail ends. Fish plates also resist impact. Deflections at the joints are also minimised by closer spacing of sleepers. Other functions of fishplates include:

    i. *Transfer of wheel loads from one rail to the other.*
    ii. *Provide stiffness to rail and rail joint (Fig. RW-25, left picture).*
    iii. *Allowing expansion and contraction of rail ends due to temperature variations.*

Fig. RW-25: Placing rail fish plates; left picture shows stiffening fishplate

14. Rail clips were clamped to the sleepers (Fig. RW-26) to ensure proper fastening between sleeper and steel rail, and maintain the alignment of the rail which can go out of alignment due to any movement, bending, warping etc.

Fig. RW-26: Typical clip; Clamping between rail and concrete sleeper with rail clips

15. Once the fastening systems were in place, rail alignment was checked using a Track Gauge and superelevation measuring device as seen in Fig. RW-27.

16. After the alignments were checked to satisfaction, the sides of the sleepers were packed with ballast as may be seen in Fig. RW-28.

Fig. RW-27: Alignment check using Track gauge measurement device

Fig. RW-28: Sides of sleeper packed with ballast

17. Compaction of the ballast was further carried out by the “Dynamic Tamping Express “machine (Fig. RW-29).

    The Dynamic Tamping Express machine (Fig. RW29(a)) imparts vibration and compacts ballast after correction of the track geometry, for safe movement of high-speed trains. The machine also helps in eliminating manual measurement of track quality after maintenance.

Fig. RW-29(a): Dynamic Tamping Express machine

Fig. RW-29(b): Compaction of ballast by Dynamic Tamping Express machine

18. After the required compaction by Dynamic Tamping Express machine, the track was opened for movement of passenger express trains (Fig. RW-30) and cargo freighters on March 02, 2021, itself. Initially, the speeds were restricted, limited to 20kmph. Fig. RW-30 shows the first passenger train on the geocell reinforced approach.

Fig. RW-30: The first passenger train on the trial geocell reinforced approach stretch

Fig. RW-31: Finished track surface at the bridge - approach transition zone

19. The completed and operational track surface provided smooth movement of train traffic over the bridge - approach transition (Fig. RW-31).

20. Two weeks after opening to traffic, on March 16, 2021, the track was monitored visually. It was noted that there was no deflection nor change in track geometry / alignment.

21. The second trial stretch was taken up on the South approach of the bridge of the same Up track on March 16, 2021.

22. Both the trial approaches were completed as scheduled within the stipulated time periods.

POST STRENGTHENING TRACK PARAMETERS - PERFORMANCE MONITORING AND EVALUATION

The Western Railway has monitored the performance of the trial approaches as part of routine inspection and maintenance. The monitoring is generally done as follows:

1. Periodically monitor evaluate Track Geometry Index (TGI), using track recording machine (usually done thrice a year).
2. Periodical level / deflection measurement with progressive increase in speed of trains every month, starting with the initial speed limit of 20kmph and progressively increasing by 20kmph to the maximum speed of 160kmph.

On an immediate basis, following successful completion of work, the speed limit caution of 20Kmph was relaxed to normal after 3 rounds of tamping on April 3 and 28, 2021. Thereafter, track parameters were recorded once weekly. Tables RW-2 and RW-3 provides the details of performance parameters.

Table RW-2: North Approach – Track parameter review after geocell reinforcement

Date	Apr 03, 2021				Apr 10, 2021				Apr 16,2021				Apt 23, 2021				30.04.2021			
Station	G	XL	UE		G	XL	UE		G	XL	UE		G	XL	UE		G	XL	UE	
			L	R			L	R			L	R			L	R			L	R
0	-4	3W			-3	4W			-4	3W			-2	4W			-2	3W		
1	-3	4W	1	1	-2	3W	1	1	-2	3W	0	0	-2	3W	1	1	-2	3W	0	1
2	-4	4W	0	1	-3	4W	1	1	-4	4W	1	0	-3	4W	1	1	-3	4W	1	1
3	-3	5W	1	1	-2	2W	2	1	-3	4W	0	1	-3	3W	1	1	-2	3W	1	1
4	-4	2W	2	0	-2	4W	2	1	-3	3W	1	1	-2	4W	1	0	-3	4W	1	1
5	-2	3W	1	1	-4	2W	2	0	-3	2W	1	0	-4	2W	2	0	-4	3W	1	0
6	-4	2W	1	1	-3	4W	2	1	-3	4W	2	1	-3	3W	1	0	-2	4W	1	0
7	-3	3W	1	2	-3	5W	1	1	-4	3W	1	1	-3	3W	0	1	-3	3W	1	0
8	-3	2W	1	2	-4	3W	2	1	-3	4W	1	0	-4	4W	1	1	-4	3W	0	0

Table RW-3: South Approach – Track parameter review after geocell reinforcement

Date	April 28, 2021				April 29, 2021				May 01, 2021				03.05.2021				May 05, 2021			
Station	G	XL	UE		G	XL	UE		G	XL	UE		G	XL	UE		G	XL	UE	
			L	R			L	R			L	R			L	R			L	R
0	-3	3W			-4	3W			-3	2W			-2	2W			-3	3W		
1	-4	3W	0	0	-3	3W	0	0	-2	3W	1	0	-3	2W	0	1	-2	3W	0	0
2	-3	4W	1	1	-2	3W	0	0	-3	2W	1	0	-2	3W	1	1	-4	2W	1	0
3	-2	3W	1	1	-2	5W	2	1	-3	3W	1	1	-4	2W	1	1	-2	4W	2	1
4	-4	2W	1	1	-3	3W	2	2	-3	3W	0	1	-5	4W	2	0	-3	3W	1	1
5	-3	3W	1	1	-4	4W	1	1	-2	2W	1	1	-4	2W	2	0	-4	4W	1	0
6	-4	2W	1	0	-3	4W	0	1	-3	3W	1	0	-5	3W	1	0	-3	2W	2	0
7	-3	3W	1	1	-4	3W	1	0	-5	3W	0	0	-2	3W	0	0	-3	4W	2	1
8	-2	3W	0	1	-2	3W	0	0	-4	4W	1	1	-3	4W	1	0	-2	3W	1	1

Key to Symbols of Tables RW-2 and RW-3

All dimensions are in mm:

G: Gauge

XL: Cross Level

2W, 3W, 4W etc.: W stands for West. Hence 2mm, 3mm, 4mm etc. deformations to the West

UE L & UE R: Unevenness, Left and Right Rail respectively.

Considering the magnitude of quality parameter readings, the Western Railway concluded that there was no deterioration in track, even after a period of more than 1.5 months of traffic. In addition to recording the track parameters, frequent inspections were carried out. The running on the Bridge approaches was found to be very good, according to these inspections as highlighted in Table RW-4.

Table RW-4: Inspection observations

Type of Inspection	Date	Inspecting Authority	Observation
Last Vehicle	May 05, 2021	DEN C	Running was smooth
Last Vehicle	April 06, 2021 May 05, 2021	ADEN ST	Running was smooth
Foot Plate	April 03, 2021 April 21, 2021	ADEN ST	Running was smooth
Trolley Inspection	April 07, 2021	ADEN ST	Running was smooth
Last Vehicle	April 06, 2021 May 05, 2021	SSE PWAY NVS	Running was smooth
Footplate	April 09, 2021 April 25, 2021	SSE PWAY NVS	Running was smooth
Trolley Inspection	April 07, 2021 April 15, 2021	SSE PWAY NVS	Running was smooth

Moreover, two OMS runs were carried out since rehabilitation of the approaches and no peak has been observed on the approaches. It was therefore concluded that the trials of strengthening the approaches were successful. On this basis, RDSO published *"Transition System on Approaches of Bridges*", Report No. GE: R-50 (Revision 1) in July 2021.

SECTION VII: GEOSYNTHETICS AS LOAD BEARING SYSTEMS FOR CONTAINER YARDS

(Code: CY)

PREAMBLE

With increase in maritime container cargo traffic, there is a need for increased capacity for container transit storages at existing ports. With ports (other than for inland ports) invariably required in marine or riverine environments, container terminals are located on land reclaimed from mudflats of sea, creeks, rivers, and estuaries. Owing to geographical and environmental constraints, suitable material is not available for such reclamations. Besides, the natural deposits are invariably weak. Weak founding sub-strata pose a challenge to developing container yards alongside ports. Several ground improvement techniques have been used in the past which change the mechanical characteristics of the subsoil strata.

Yet another method to support heavy loads over weak soils is to distribute the imposed pressures to an extent that these pressures can be borne by the subsurface safely and with tolerable deformations that could cause distress to the systems being supported. This can be achieved by judicious use of geosynthetics.

CASE STUDY - ALLCARGO LOGISTICS LTD.

The All-Cargo Logistics Limited case study relates to a container yard near Kidderpore Docks in Kolkata, India, on the banks of the Hooghly River. The surface stratum comprises of ±3m thick fill which includes construction debris in clay matrix, with significant voids. This is underlain by soft marine (riverine) clay.

Standard loaded containers will be placed maximum four-high in close clusters. To effectively reduce imposed pressures on the fill and at subsoil levels, two layers of HDPE geocells, two layers of rigid biaxial PP geogrids and nonwoven geotextiles have been judiciously designed and placed, along with concrete paver blocks at the surface.

By avoiding the conventional layer of dense lean concrete, there is considerable savings in capital costs and construction time. The proposed cross section of the paving system also minimises differential settlement besides reducing imposed bearing pressures. With this system, the maintenance cycle is extended which not only reduces downtime, but also life cycle costs.

AN OVERVIEW

All Cargo Logistics Ltd. has set up a logistic park in the Kolkata Port area, comprising essentially of a container yard, besides an administration office and security gate house, with provision for minor storages. Fig. CY-1 shows the site location. Besides the minor light structures, the container yard area provides for container stacks four high, arranged in blocks over the paved system over the subgrade. The system also supports stackers operating adjacent to the container stacks.

The area is low lying and in a congested locality as seen in Fig. CY-1(a), near Kidderpore Dock. The subsoil includes 7 to 8m of riverine deposits of very soft to soft clay, topped by about 3m of heterogeneous fill, the surface nature of which is highlighted in Fig. CY 2.

Fig. CY-1(a): Site location (marked red) at Kidderpore Dock

Fig. CY-1(b): The Site

Fig. CY-2: Original Ground Condition (heterogeneous fill)

THE ISSUES

The primary concern was adequacy of bearing capacity for the heavy container loads. It was essential to either improve the safe bearing capacity or ensure that the imposed bearing pressures are low enough to be sustained by the subsoil strata. Under container loads, non-uniform settlements were yet another concern to be addressed.

The conventional solution considered by the Client and Consultant was to install prefabricated vertical drains, or PVDs. The process of ground consolidation with PVDs requires surcharging, which requires large quantities of earthwork to achieve the final load. In order to develop the safe bearing capacity, this surcharging would have to be carried out stage-wise to ensure that the bearing capacity progressively developed adequately to take the next load of surcharge. This is a slow process requiring, close monitoring of settlements, as well as sub-soil pore water pressures. However, the Client did proceed with PVDs for the lightly loaded single and two storied structures.

In the container stack-yard, the loads are heavier considering various loading combinations of sustained container stacks and intermittent stackers. While PVDs with appropriate surcharge would have improved the safe bearing capacities, for the stacking area, PVDs required wasteful surcharging equivalent to the anticipated container stack loads. For 90% consolidation, the time required with PVDs was estimated at five months only. However, for the stack yard, the magnitude of surcharge, transporting and handling of the surcharge material within a congested urban site, time involved for stage-wise retaining of surcharge to develop the required bearing capacities, cost of earthworks and its redundancy after surcharging, was a daunting thought and not practical.

THE SOLUTION AND ITS ANALYSIS

In view of the negatives outweighing the pros of the PVD option, an innovative solution was worked out. A detailed analysis was carried out considering worst case scenarios for loading, along with deliberations on settlement. The solution incorporated geocells using the guidelines of the INTERPAVE Manual (1).

It was thought prudent to judiciously place the geocells within the paving to reduce the bearing pressures onto the subgrade by effectively spreading the imposed loads over a larger area. This provided a leaner design for the pavement section. The attribute of load-spread provided yet another advantage. Considering that the consolidating stratum below is of limited thickness, settlements under sustained limited area loads will not only be reduced but also occur over a wider area. As a consequence, differential settlements would be within acceptable limits.

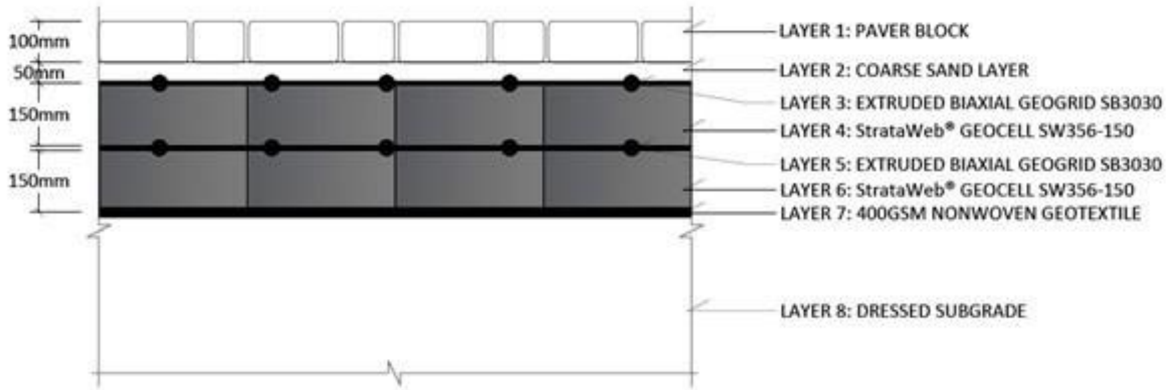


Fig. CY-3: Section adopted

The section illustrated in Fig. CY-3. is designed to consider the worst-case scenarios of four-stack high containers and stacker. Top down, the system comprises:

I Concrete paver blocks of thickness 100mm.

II Sand 50mm thick.

III Rigid biaxial geogrid placed at the base of the sand layer and atop infilled geocell.

IV Geocells infilled with 10mm down gravel, of depth 150mm.

V Rigid biaxial geogrid placed between the two geocells within gravel.

VI Geocells infilled with 10mm down gravel, depth 150mm.

VII 400GSM PET nonwoven geotextile.

Analyses showed that imposed pressures with the section proposed in Fig. CY-3, using StrataWeb® SW356-150mm are well within the safe bearing capacity. Two layers of geocells were found necessary for adequate load spread. Vertical deformations for sections with geocells are also less than those for a conventional section.

The soft consistency of the top fill required laying a nonwoven geotextile as a separation layer before placing the lowest layer of geocells. Considering the paucity of material qualifying as "Wet Mix Macadam" (WMM) in the region, 10mm down-graded gravel was used as infill, as well as topping for the geocells. Rigid geogrids were provided below the paver blocks as well as between the geocell layers to cater to traction forces and forces due to turning of the stacker equipment as well as the container tractor-trailers.

Construction sequence of the geocells is shown in Fig. CY-4. Visual inspection has been carried out after one year to evaluate the performance of the geocells. Figure 5 shows no distress indicating overall good performance of the section.

Fig. CY-4(a): Dressed fill surface

Fig. CY-4(b): First layer of geocells laid over nonwoven

Fig. CY-4(c): Infilling of geocells

Fig. CY-4(d): Completed paving with geocells

Fig. CY4: Construction sequence

Fig. CY-5: Completed paving with geocells after one year

CONCLUSIONS

For the container yard in this case study, a section incorporating two layers of geocells with weld spacing of 356mm and depth 150mm was considered. The geocell layers were in-filled with non-plastic granular material to reduce stresses on the foundation subgrade to within the limits of safe bearing capacity of the soil.

Apart from the direct cost savings, the long-term benefits and indirect cost savings include:

Savings in project construction time.

Reduction in differential settlements leading to improved operational efficiency and reduced downtime.

Reduction in life cycle maintenance cost owing to the flexible nature of the pavement and reduced differential settlements.

SECTION VIII: GEOCELLS FOR GRAVITY WALLS

(Code: WL)

PREAMBLE

While most geosynthetics have a planar profile, geocells have a three-dimensional, rhomboidal structure. While the three-dimensional structure imparts flexural strength, the cellular configuration of the geocell provides confinement to the infill material. Geocells with appropriate infill material can be used as building blocks for structures such as gravity retaining structures. Cases have proved geocells to be cost effective as compared with conventional concrete structures. The infill materials are local soils. Apart from cost and time savings, geocell structures help fostering greenery and give a pleasing aesthetic finish.

Retaining walls have traditionally been used to retain earth for terraces and grade separators. The material of construction for these walls has been stone masonry and reinforced concrete. Masonry structures are designed essentially as gravity structures where the self-weight of the wall and backfill are stabilising agents against the activating sliding and overturning forces. The wall is structurally designed such that it is completely in compression and there is no tension within the body of the wall. The entire system is also checked for global stability. The reinforced concrete wall system is similarly designed. However, the vertical stem and footing are subject to bending moments and shear and are designed accordingly.

Masonry structures are constructed of stone rubble in cement mortar. Construction requires specialist masons, a fast-disappearing breed, and is very slow and laborious. The material of construction is stone, a natural resource whose quarrying is mostly banned in several regions in India from environmental considerations. In today's milieu, masonry retaining walls are generally not an acceptable option. Reinforced concrete retaining structures can also be cumbersome to construct particularly when the site is remote and ready-mix concrete is difficult to procure. Site production of quality concrete would require appropriate machinery and equipment. Reinforcement steel is an essential ingredient which requires elaborate cutting, bending, placement and binding as per the drawings. Adequate space and skilled workmen would be required for these works. A major factor for reinforced concrete retaining walls is the high cost of construction.

Geocells have commonly been used for prevention of erosion of natural and man-made soil slopes as well as for spreading of loads over larger areas where the subgrades / subsoils are weak. Such geocells have also been considered as building panels for gravity structures and as modular units of fascia for reinforced soil structures.

GRAVITY STRUCTURES

Gravity retaining structures can be constructed from panels of geocells. These walls can be fabricated rapidly. Construction of the gravity retaining structure using geocells is simple;

the geocell panels are stacked atop each other. Soil or aggregate is infilled within the geocells as the expanded panels are stacked. The panels are placed such that there is a progressive backward batter. The mechanics of the system is the same as a conventional gravity wall and the infilled geocell panel layers resist activating sliding forces and overturning moments by virtue of their weight.

Geocell gravity retaining walls have a reasonable degree of flexibility that allows them to be constructed over foundation soils which are non-uniform along the length with possibilities of large differential settlements. The soil ledges created along the front face by the backward batter can be vegetated for aesthetics.

Considering that one of the design criteria is sliding between two layers of geocells, it is necessary that the infill is non-plastic soil, since the coefficient of friction μ is a function of the drained angle of internal friction $\emptyset$. The cell walls are perforated which facilitates passage of water through the walls of the geocells and circumvents the need for specific weepholes. This further vindicates the need for high permeability non-plastic soil as infill. Often for aesthetic, the outermost geocell strap is unperforated. In such cases, perforation holes are drilled through straps for cells at specific horizontal and vertical intervals

MECHANICS OF GEOCELLS AS GRAVITY WALLS

The typical gravity wall constructed of geocells is shown in Fig. WL-1. The wall basically comprises of HDPE geocells placed atop each other. The orientation of the geocells is such that the straps are parallel to the alignment of the wall. The number of cells along the cross section of the wall is based on design requirements, essentially overturning, and sliding at each cell base.

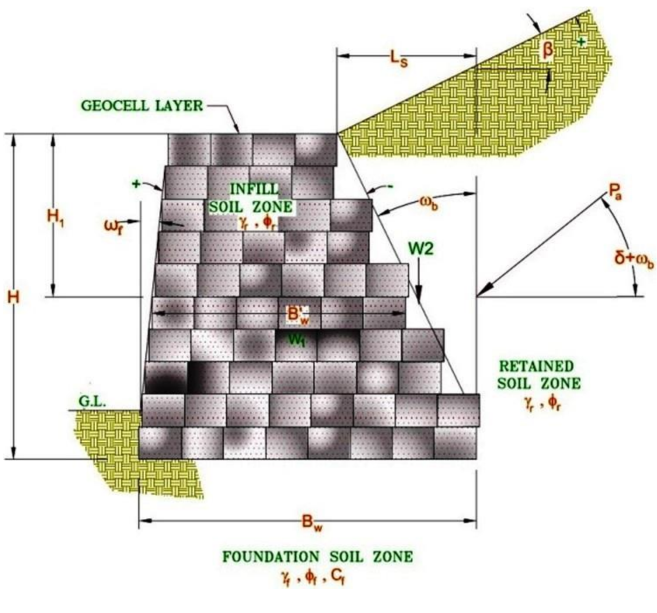


Fig. WL-1: Typical gravity retaining wall of geocells

STATIC ANALYSIS

Analysis of the wall is similar to the conventional gravity wall. Activating sliding and overturning forces are resisted by the self-weight of the structure and the contributing backfill above the wall internal batter. There is no mortar connection between the geocell layers; hence it is prudent to check that the factors of safety against sliding and overturning at *each* geocell layer junction, ensuring that the resultant force lies within the middle third of the section width. The minimum allowable factors of safety recommended are 1.5 for sliding and 2.0 for overturning.

Coulomb theory is generally considered to evaluate earth pressures against the geocell wall, and the active earth pressure coefficient is

$$k_a = \frac{\cos^2(\emptyset+\omega_b)}{\cos^2\omega_b \cos(\omega_b-\delta)\left[1+\sqrt{\frac{\sin(\emptyset+\delta)\sin(\emptyset-\beta)}{\cos(\omega_b-\delta)\cos(\omega_b+\delta)}}\right]^2} \qquad \text{(WL-1]}$$

+where the various symbols are explained in Fig. WL-1.

SEISMIC ANALYSIS

During a seismic event, the retained fill will exert an additional dynamic horizontal thrust along with the static thrust. Additionally, the geocell wall is also subjected to horizontal inertia force. All seismic parameters are to be considered as per the Bureau of Indian Standards IS 1893 (Part 1): 2016 “Criteria for Earthquake Resistant Design of Structures – Part 1 General provisions and Buildings”.

Seismic analysis has been done for the Aamby Valley Project discussed below. The site, in Lonavala, Maharashtra is located in Zone III as per IS 1893. At the time when the Project was designed, the 2002 Edition was followed. Accordingly:

Zone Factor $Z = 0.16$ for Zone III

Importance Factor I is considered as 1, since the structure is neither important service, nor community structure.

Stability of the structure is checked against sliding and overturning under seismic conditions. The safety factors should be 0.75 times those under static conditions. Hence under seismic conditions, the factor of safety against sliding under seismic conditions is the minimum 1.1 and 1.125 against overturning. These conditions were satisfied in the case of Aamby Valley.

CASE STUDIES

Two case studies concerning geocell gravity retaining walls are discussed here. One is within a private property at Aamby Valley and the other is in public space in Bengaluru. In both cases, gravity retaining structures were to be constructed on an urgent basis. A gravity retaining wall constructed of geocells was deemed to be fastest.

In Aamby Valley, rainfall is heavy, and the structure was required to be pervious to relieve pore water pressures. The region lies within Zone III on the seismic map, not too high, but nonetheless, the structure designed on the basis of static loads, was required to be checked for seismicity.

In the Bengaluru case, the rainfall is not intense, and the surface was rendered impervious with an asphaltic pavement over the surface. However, since the outer cells of the geocell wall were required to be filled with concrete, the structure was designed for full hydrostatic pressure. Bengaluru lies in seismic Zone II and did not warrant a seismic check for a 2m high earth structure.

AAMBY VALLEY

Aamby Valley is a holiday cluster located about 17km from the hill station of Lonavala in Maharashtra, India. The area is located on the Western Ghats along the West periphery of the Deccan Plateau. The basic rock is igneous basalt, and the surface soil is lateritic, derived from this rock. Being located on the Western Ghats, the terrain is hilly and highly undulating. Lonavala encounters one of the intense rainfalls of the country and Aamby Valley faces the full fury of the monsoons.

The courtyard in front of the Sundesha Properties Bungalow was subjected to heavy erosion due to surface run-off since the monsoons of 2013. Erosion was critical at places to an extent that caused soil collapse in wedges which aggravated slope failures at some locations. The problem became critical with the possibility of a slope failure undermining the plinth of the bungalow and endangering the shallow footings of the structure. Fig. WL-2 highlights the condition of the courtyard after the 2013 monsoons. A permanent engineering solution was sought, essentially to be implemented before the onset of the 2014 monsoons.

Fig. WL-2: The initial condition

A retaining wall structure was proposed to contain the ground supporting the building foundations and prevent further erosion. However, time was inadequate prior to the onset of the 2014 monsoons to construct a conventional structure such as a masonry gravity wall or a reinforced concrete retaining wall. A gravity wall constructed of HDPE geocells was proposed. The geocells are 300mm deep with weld spacing of 356mm. The structure stretches over 70m and is 3m high. Considering the scale of construction, the geocell gravity wall was most rapid to construct as well as cost effective.

The geocell gravity wall comprises of infilled geocell panels placed one on top of the other. Design of the gravity structure was carried out as outlined above. Appropriate surcharge loads were also considered during the design of the system. The system was checked panel by panel for sliding and overturning. Individual cells are perforated, which allows subsurface water that may have seeped into the courtyard to flow out without any build-up of pore water pressures. Seismic analysis was also carried out on the section as highlighted above. The wall section is as shown in Fig.WL-3.

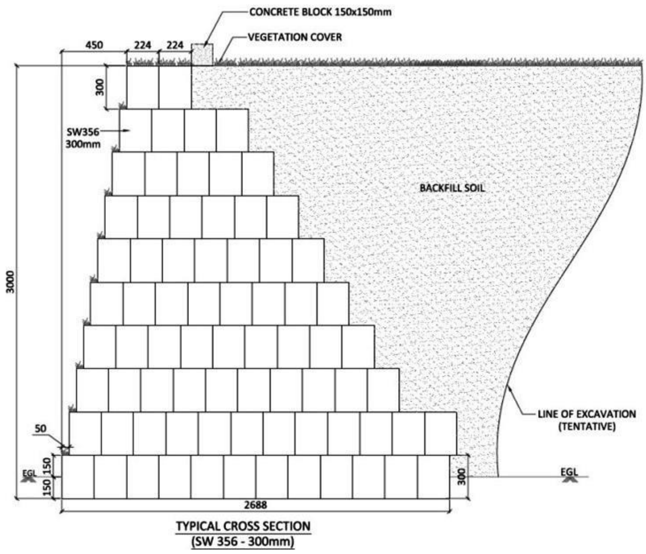


Fig. WL-3: Geocell wall cross section at Aamby Valley

Construction was simple and rapid. Initially, the bottom panels were spread out with the help of steel spikes. These panels were manually infilled with local lateritic soil. The full depth of 300mm of the cells were filled and topped off with additional 25mm to 40mm of soil. A vibratory plate tamper was used to compact the infilling. After trimming off the excess soil, the next row of panels was placed in position with the help of spikes. The panels were placed with an inward batter of 50mm to line. Earth backfill behind the geocell panels was placed simultaneously to support the overhanging offset of the panels behind the wall due to the batter. The batter of each panel also created a ledge of soil to form a base to sustain vegetation.

The sequence of construction is illustrated in Fig. WL-4. Construction was completed by May 2014, about one week from start of work.

Fig.WL-4(a): Bottom-most row of geocells being laid out

Fig. WL-4(b): Progressive construction of geocell wall

Fig. WL-4(c): Wall batter detail

Fig. WL-4(d): Completed wall with mesh in front for green climbers

Fig. WL-4: Sequence of construction

Bruhat Bengaluru Mahanagara Palike Bridge Approach

The Bruhat Bengaluru Mahanagara Palike (BBMP) had awarded the work of a bridge construction over a stormwater drain near Gali Anjaneya Temple on Mysore Road in Bengaluru, to URC Construction, Erode. STUP Consultants, Bengaluru were the Consultants for this Project.

Construction of the approach to the bridge was required within a short timeframe and in the midst of traffic congestion. A conventional concrete retaining structure would have required space for construction, besides suffering time over-runs. In view of these constraints, it was decided to construct retaining structures of the approach ramps using geocells for economy, speed and ease and convenience of construction in the congested environment. Considering that the wall is located in a public area, it was thought prudent to shotcrete the fascia to prevent vandalism and minimise damage in case of vehicle collision.

The height of the wall varies from 1m to 2m. Area of the wall fascia is 1,137sq.m. The design cross section is shown in Fig. WL-5.

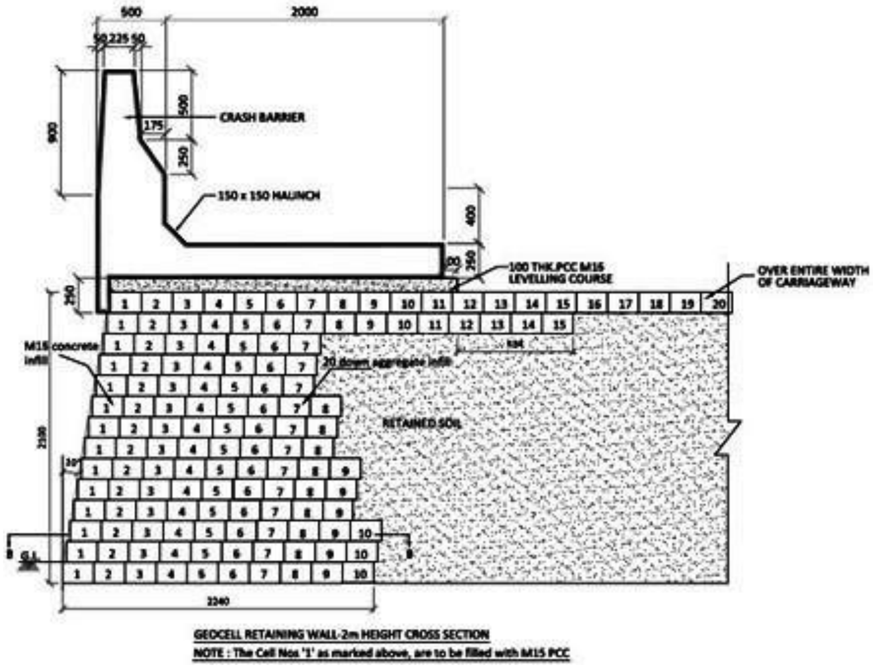


Fig. WL-5: The engineered section

Work on the wall commenced during the last week of January 2013 and was completed in 10 days' time. HDPE geocells, weld spacing 356mm and depth 150mm were used. Each panel measured 2.6m X 5.6m. The size of each cell was 224mm X 259mm. The width required at the base was spread over the dressed ground and held in position with steel spikes as may be seen in Fig. WL6.

Cells along the fascia were filled with M15 grade concrete. The cells beyond were infilled with 20mm-down aggregates with a slight over-fill (Fig. WL-7). The in-filled panels were compacted with a 10-tonne vibratory road compactor. The over-fill ensured that the vibratory compactor did not damage the geocells.

After compaction, the over-fill was scraped off and the next layer of geocell panels was laid on top. All layers were completed likewise, and the retaining wall was completed in ten days' time.

Fig. WL-6: The bottom-most geocell layer

Fig. WL-7: Infilling the bottom-most layer

Thereafter, the concrete friction slabs for the crash barriers were cast on the surface along the edges and the crash barriers were cast on top as seen in Fig. WL8. The facade was shotcreted (Fig. WL9).

Fig. WL8: Friction slab (left) and crash barrier construction (right

Fig. WL9: The completed wall

Conclusions

Geocells have been successfully used to construct gravity retaining structures.

The major advantage over masonry gravity retaining structures and reinforced concrete retaining walls is that construction is rapid. The material is light and can be manually handled. Neither mechanised equipment nor skilled manpower is required. Construction can proceed even in bad weather conditions. The system can be used for constructing ramps and approaches during emergency conditions. Geocell gravity retaining structures also prove to be more cost effective particularly where the magnitude of the work is small. Local soil can be used as infill, which reduces material haulage time and costs.

Since the geocells used are perforated and the perforations are 10mm diameter, hydrostatic pressures do not develop behind the wall as long as the infill material is adequately pervious. Hence no separate weep holes are required. The perforations cover less than 12% of the surface area of the cell walls and do not compromise the tensile strength of the material.

The material is flexible and does not crack unlike concrete and masonry structures. Owing to the flexibility of the material one can render smooth corners and curves.

Geocells provide a green solution. While in the Aamby Valley case, a mesh was erected in front of the wall to foster climbers, the soil within the batter can also nurture vegetation.

The Indian manufactured HDPE geocells used for these Projects are fabricated with HDPE with 2% carbon black. The material is UV resistant and does not deteriorate with time.

Last but not least, the system is truly environment friendly. It leaves a very small carbon footprint owing to minimal resource requirements and minimised transportation.

SECTION IX: GEOCELLS AS FASCIA FOR REINFORCED SOIL STRUCTURES

(Code: FS)

PREAMBLE

Geosynthetic reinforced systems have replaced reinforced concrete retaining walls by virtue of their simplicity of construction, speed of erection, significant cost economics and essentially use of less natural resources. Reinforced soil retaining structures, being flexible in nature, are extensively used not only for roads in plain terrain but also along hill roads. While concrete fascia (precast blocks or panels) is commonly used, as a step further towards use of geosynthetics, the Author holds a brief for perforated HDPE geocells as fascia.

Researchers and Practitioners have highlighted various aspects and relative advantages of facing elements. Bathurst (1992) presented numerous concept design approaches for reinforced soil wall and gravity walls using geocells which widened the practical use of such flexible structures. Studies conducted by Mehdipour et.al. (2013) carried out numerical analysis on geocell reinforced soil structure using Fast Lagrangian Analysis of Continua (FLAC) 2D programme cited benefits of geocells in reinforced soil structures. Ling et.al. (2009) and Latha (2016) conducted and simulated laboratory shaking table test to analyse the performance of geocell retaining wall and concluded its benefits. However, very few field case studies were presented for such type of applications.

This Section brings out a simple design approach with connection between geogrid reinforcement and geocell fascia based on friction and hinge height, along with construction methodology. A reference is made to a case study in hill terrain susceptible to high seismic events.

TYPES OF FASCIA ELEMENTS

Facing elements may be “hard” or “soft”. The elements are provided to retain fill material, prevent local slumping and erosion of steeply sloping faces, and to lend to aesthetics.

Hard facing may consist of mostly concrete, occasionally steel wire mesh, and rarely steel sheets, and timber. In general, in India, the following two types are widely used:

1. Reinforced concrete panels.
2. Modular precast concrete blocks.

These typical fascia types are schematically shown Fig. FS-1 and Fig. FS-2.

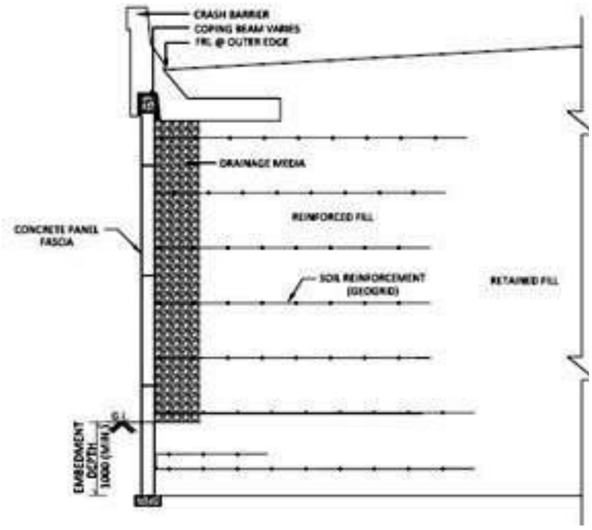


Fig. FS-1: Reinforced soil wall with concrete panel fascia

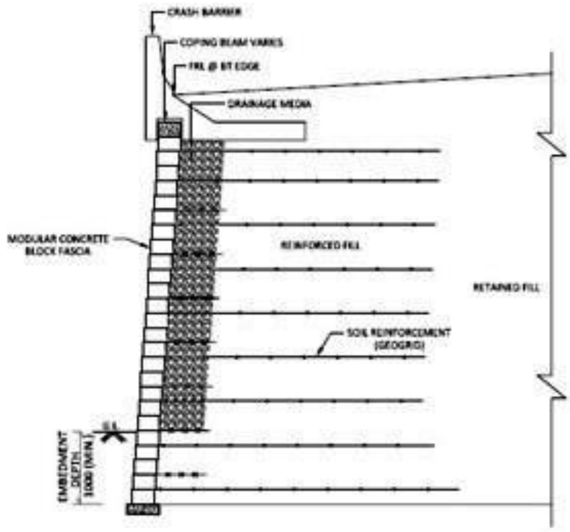


Fig. FS-2: Reinforced soil wall with concrete modular block fascia

Soft fascia generally requires external temporary formwork to support facing elements. However, construction can be expedited using these facing elements with minimal labour as compared with concrete facing elements. Soft facing requires less storage space with minimal handling. Broadly, two major categories of soft fascia element include following facing types:

1. Wrap around facing.
2. Geocell fascia.

In case of wrap around facing elements, geogrids are wrapped around at the face and returned into soil directly below the next upper layer. This process can impede the work. In the case of geocells as fascia, the infilled soil within the cells holds the geogrid reinforcement by soil-reinforcement interaction. Geocell fascia are flexible and robust. Thus, the speed of construction is relatively faster as compared with reinforced soil wall with wrap around fascia. Typical schematic cross-sections of soft facing elements are shown in Fig. TS-3 and Fig. FS-4.

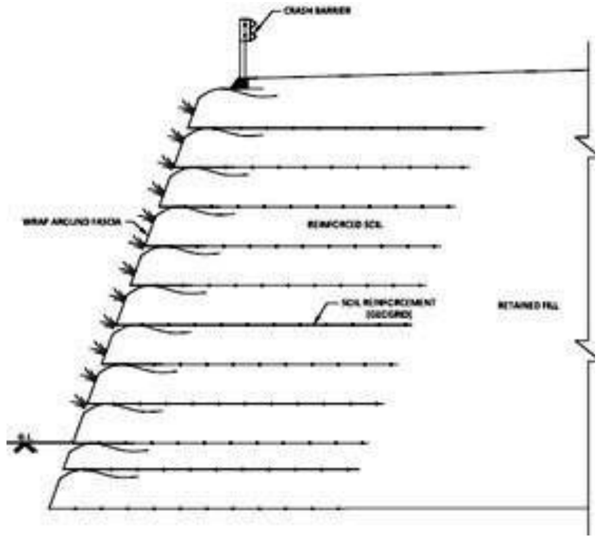


Fig. RS-3: Reinforced soil wall with wrap around fascia

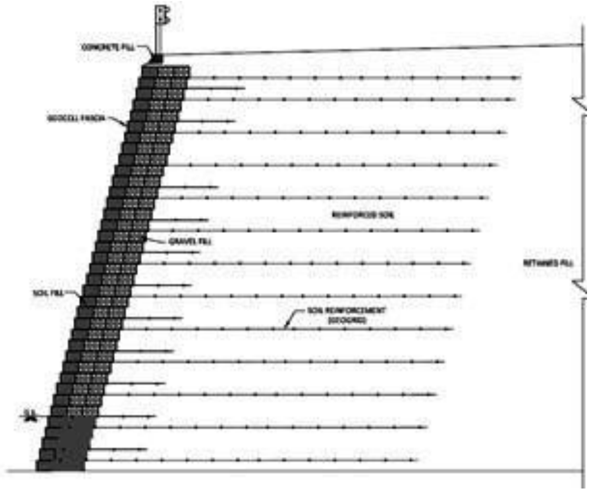


Fig. FS-4: Reinforced soil wall with geocell fascia

ADVANTAGES OF GEOCELL FASCIA

To mobilise connection strength, geocells are essentially filled with non-plastic soil. Geocell fascia are particularly beneficial in hilly terrain where there is little space adequate enough to cast concrete blocks or panels.

The essential advantage of concrete blocks and panels lies in economies of scale. In hilly terrain, the required fascia area may not be large enough to warrant setting up of casting yards from economic considerations. Besides, level land for such yards may not be available.

Geocells are transported and stored in collapsed form and are expanded into panels at site. Hence logistics to site would not be a major constraint. Being light and flexible, and filled in situ, geocells can be manually handled. On the contrary, concrete blocks weigh as much as 40kg and panels as much as 1000kg. each.

Flexibility of geocells permits smoother and sharper curves and corners.

WIDENING THE APPROACH ROAD TO ARMY OFFICERS' MESS IN SIKKIM

THE PROJECT

This case study addresses use of geocells as fascia for a reinforced soil system to widen the approach road to an Army Officers' Mess. The location is a in hilly terrain in Sikkim. A satellite imagery of the location is shown in Fig. FS-5. The road is 2m to 6m above the Mess grounds. The road required widening by 3m for commute and a parking lot. Length of the road is about 100m.

The region is within Seismic Zone IV and is also subject to heavy rainfall, annually 2,000mm to 5,000mm.

With such conditions, a reinforced soil structure with geocell fascia was considered appropriate and was proposed.

Fig. FS-5(a): Satellite imagery of the project site

Fig. FS-5(b): Proposed RS wall location

ANALYSIS

Analyses of the reinforced soil structure were carried out using the MSEW (Mechanically Stabilized Earth Wall) software recommended by FHWA (Federal Highway Administration). Input parameters considered for the analyses are listed in Table FS-1.

Table FS-1: Input parameters for analyses

Material Properties	Value
Reinforced Soil	γ_1 = 18.5 kN/m^3, ϕ = 30°, c = 0kPa
Retained Fill	γ_2 = 18.5kN/m^3, ϕ = 28°, c = 0kPa
Foundation Soil	γ_3 = 18 kN/m^3, ϕ = 30°, c = 0kPa
Seismic Zone & Factor (IS 1893 (Part 1): 2002)	Zone IV (0.24)
Geogrid (StrataGrid™ Systems)	SGi 40; Short Term Ultimate Tensile Strength: 40 kN/m) SGi 60; Short Term Ultimate Tensile Strength: 60 kN/m SGi 80; Short Term Ultimate Tensile Strength: 80kN/m) SGi 100; Short Term Ultimate tensile strength: 100 kN/m)
Geocell (StrataWeb® Systems)	SW445-200: Weld spacing = 445mm, Depth = 200mm

where

γ is the unit weight of soil,

ϕ is internal friction angle of soil,

c is cohesion of the soil.

Geocell style SW445-200 is textured and perforated. The perforations ensure that pore water pressures are relieved, and the reinforced soil structure need not be designed for hydrostatic pressures within the structure. The geocell depth of 200mm was selected to ensure adequate compaction within the geocell. The depth of 200mm also ensures that each soil; layer of the reinforced backfill does not exceed 200mm and serves as a quality control tool.

Static Analysis

As per FHWA-NHI-10-024 (2009) guidelines, analysis was carried out using MSEW Software and exported into ReSSA (Reinforced Slope Stability Analysis) for evaluation of global stability. For evaluation of local stability of geocell fascia, connection strength between geocell and geogrid was evaluated using frictional resistance of geocell infill soil and geogrid. Hinge height was evaluated corresponding to a batter of 20° with the vertical. Connection Strength versus Normal Load relationship is shown in Fig. FS-6.

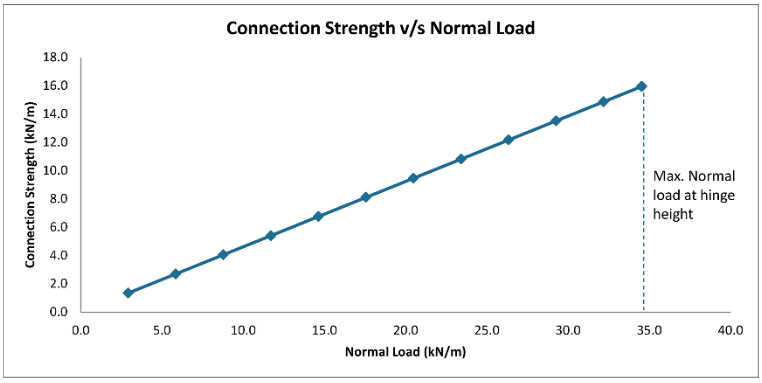


Fig. FS-6: Connection Strength v/s Normal Load

Safety of reinforced soil structure was evaluated against sliding, overturning, geogrid pull-out failure, geogrid tensile failure considering the determined long term design tensile strength (LTDS), and bearing capacity, by using MSEW. The system was proven to be safe for both internal and external stability.

Further global stability analysis through ReSSA was carried out after MSEW, to indicate a safety factor against global failure of 1.75 as against the required value of 1.3 as shown in Fig. FS-7.

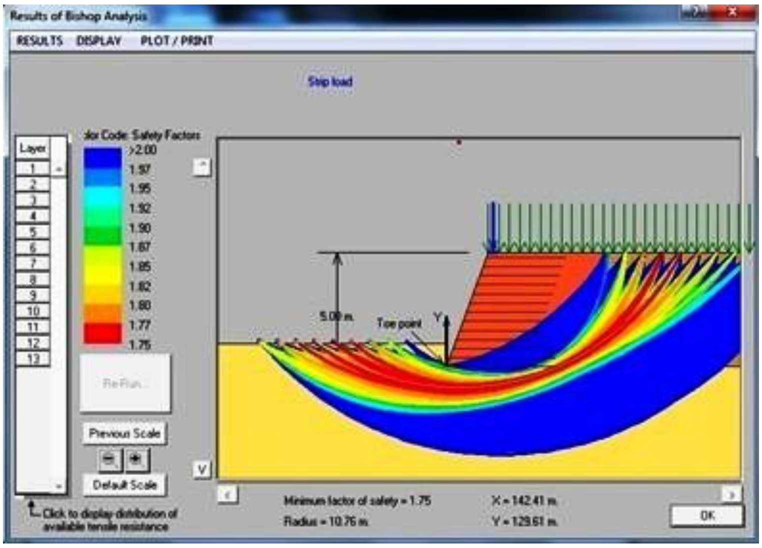


Fig. FS-7: Global stability analyses

Seismic Analysis

During a seismic event, the retained fill would exert additional dynamic horizontal thrust along with the static thrust. The seismic analysis is carried out as for any reinforced soil structure with reference to relevant guidelines.

The reinforced soil structure was thus analysed for seismic stability considering the total seismic horizontal force, which adds to the static horizontal forces, increasing the risk of sliding failure. Here, the concept of “Capacity: Demand Ratio” (CDR) is applied, wherein CDR must exceed 1. This ratio defines the capacity of the structure as against the factored load onto / within the structure. In this case, the seismic analysis output showed that the structure is safe against sliding failure with a $CDR_{sliding}$ = 1.40 (in excess of 1) for external stability. Likewise, the structure is found safe against sliding in internal stability at all geogrid reinforcement levels. Fig. FS-8 illustrates the designed cross section of the reinforced soil structure with geocell fascia for a height of 5m.

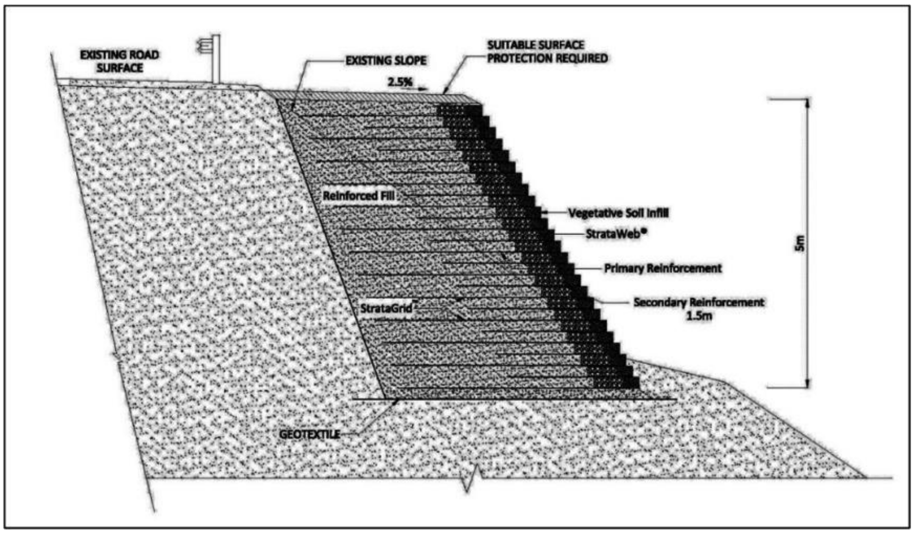


Fig. FS-8: Cross section of 5m high reinforced soil structure with geocell fascia

CONSTRUCTION SEQUENCE

SITE PREPARATION

Stone, debris, rank material, dead wood etc. was removed from the site. The subgrade was compacted by a vibratory roller. Compaction was specified to better 95% modified Proctor density.

LAYING GEOCELLS

The levelled ground was marked to locate and align geocell fascia. The first layer of geogrid was laid as per design. The geocell panels were expanded and placed uniformly using the temporary alignment stakes. Fig. FS-9 shows typical placement of geogrid and geocell.

Fig. FS-9: Laying of geogrids and geocells

The geocell fascia Is of three-cell width. The outermost strap of the geocell fascia was coloured green as per Client requirements.

INFILLING AND COMPACTION

Backfill for reinforced fill was placed and compacted in layers of 200mm. Compaction control was achieved with soil layers restricted to 200mm (Fig. FS1-0). Non-plastic soil was infilled within geocells and compacted with a handheld mechanised vibratory compactor. After achieving required compaction, excess infill material was scrapped out to facilitate placement of the next geogrid and subsequent geocell layer.

Fig. FS-10: Infilling and compaction for reinforced soil

AESTHETIC APPEARANCE

For a pleasing aesthetic look, the geocell panels were manufactured with the outer strip given a green coloration admixture which also rendered the strap UV resistant (Fig. FS-11). An offset of 70mm to 75mm was maintained between successive layers of geocells for vegetation growth. This batter also ensured a general slope of 20° with the vertical. The sequence of laying geogrids, placing, and aligning geocells, placing and compaction of soil was done till completion of the reinforced soil structure to its required height.

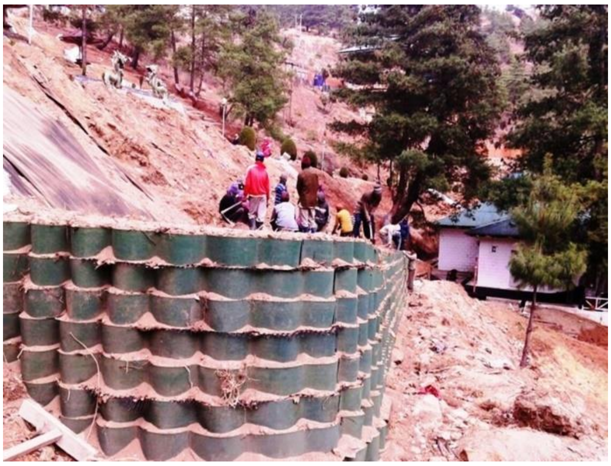


Fig. FS-11: Green façade of reinforced soil wall

CONCLUSIONS

The work was completed in September 2015 and has witnessed heavy rainfalls since. Sikkim has also experienced at least 10 seismic events ever since with magnitudes as high as 4.3.

This case study highlights that geocells can be effective as flexible fascia elements for reinforced soil structures in hilly terrain with challenging contours. Unlike hard fascia, which is vulnerable to cracking during major seismic events, flexible geocell facia perform better.

SECTION X: GEOCELLS FOR CONSTRUCTION OF CHECK DAM

(Code: CD)

PREAMBLE

The concept of geocells lends humongous opportunities to innovate. This Section presents an innovative approach to construct a small check dam, for the first time in South Asia at least. An acknowledgement to the Client which gave us a free hand to innovate all along when other proposals did not succeed.

Check dams are small dams constructed across a stream or minor waterway to counteract erosion by reducing the water flow velocity. Check dams have been used ever since ancient times in India to replenish ground water and wells. Conventionally such check dams are constructed as earth or masonry (rubble or concrete) gravity retaining structures.

The check dam of this Case Study is near Satpara, Madhya Pradesh in India as seen in Fig. CD-1(a). Prior to the proposal of installing the check dam, rainwater was allowed to flow through the stream without any attempts at ground water recharge. Fig. CD-1(b) shows the arid landscape, highlighting the paucity of water.

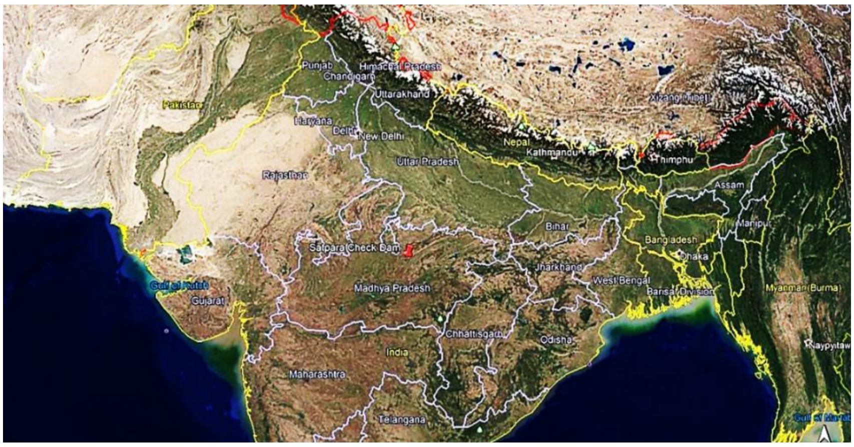


Fig. CD-1(a): Location of the site (Satpara Check Dam in Madhya Pradesh)

Fig. CD-1(b): Satellite imagery of the proposed check dam site (circled) before construction

Fig. CD-1: Proposed site imageries

The dam was constructed as a "Corporate Social Responsibility (CSR)" by a corporate entity to enhance ground water recharge for the nearby farmlands.

The site is remote and almost inaccessible for construction equipment and vehicles. It was therefore proposed to construct the check dam with HDPE geocells which are easily carried manually. The geocells were infilled with locally available non-plastic granular material. The outermost cells on the upstream side were infilled with concrete to avoid erosion of material from the outer cells and to restrict ingress of water into the body of the check dam. To relieve any possible build-up of pore water pressures within the body, suitable weep hole pipes have been provided thorough the downstream side. With the use of geocells, the work was carried out with minimum manpower and equipment. Construction commenced in mid-May 2018 and was completed by end-June 2018, well in time before the onset of the South-West monsoons.

In view of submergence of adjoining properties, the check dam was restricted to a height 3.8m from stream bed, with the spillway invert at 2.8m above the stream bed. The spillway comprised of a series of heavy-duty concrete pipes, 250mm nominal diameter were located below the crest with their invert at 2.8m above the stream bed. These pipes were supplemented by a trapezoidal channel at one end. The longitudinal length of the check dam is about an average 20m from abutment to abutment along the stream cross section profile.

The subsoil comprises essentially of plastic silt.

THE CONCEPT

The conventional method of construction of the check dam (earth fill, masonry) had the following constraints:

1. Either uneconomical due to limited budget, or not viable.
2. Constraints of construction time, only 2 to 3 months for the onset of the monsoons.
3. Remoteness of the site.
4. Poor accessibility for motorized construction equipment due to private property and cultivated farmlands.

In view of these constraints, the Client sought a proposal with a geosynthetic solution and a check dam of geocells as building blocks was recommended. It was proposed to infill the geocells with locally available non-plastic material, except the outer cells on upstream and downstream sides, which are filled with concrete to confine the material within the cells and limit the ingress of water into the body pf the check dam.

ANALYSIS

The geocell check dam is designed similar to conventional gravity structures. The check dam is basically constructed by placing interconnected geocell panels on top of each other. The orientation of the geocells is such that the geocell straps are perpendicular to the direction flow of water. The geocells proposed have weld spacing of 660mm and depth of 200mm. The number of cells in each layer along the cross section of the wall are based on the design computations by evaluating internal and external stability of the structure.

Analysis is carried out for static and seismic conditions. Minimum safety factors considered in the analysis of the dam for sliding and overturning are 1.5 and 2.0 respectively. Seismic analysis of the structure has been carried out for horizontal acceleration coefficient corresponding to Zone III. The cross-sections through the width of the check dam and its longitudinal cross-section profile are shown in Fig. CD-2 and Fig. CD3 respectively.

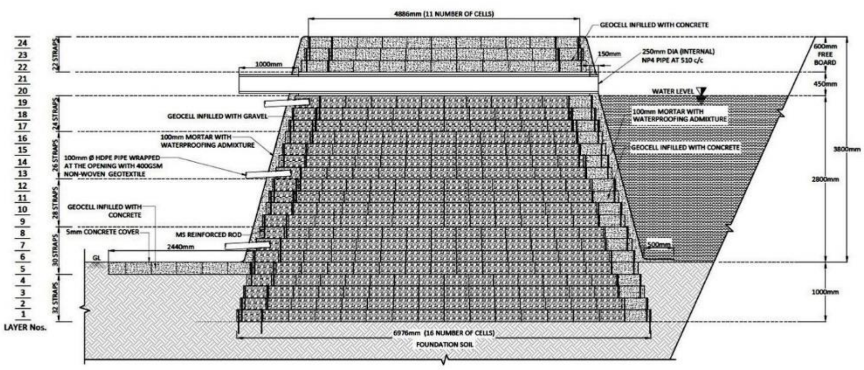


Fig. CD-2: Cross-section through the width of the check dam

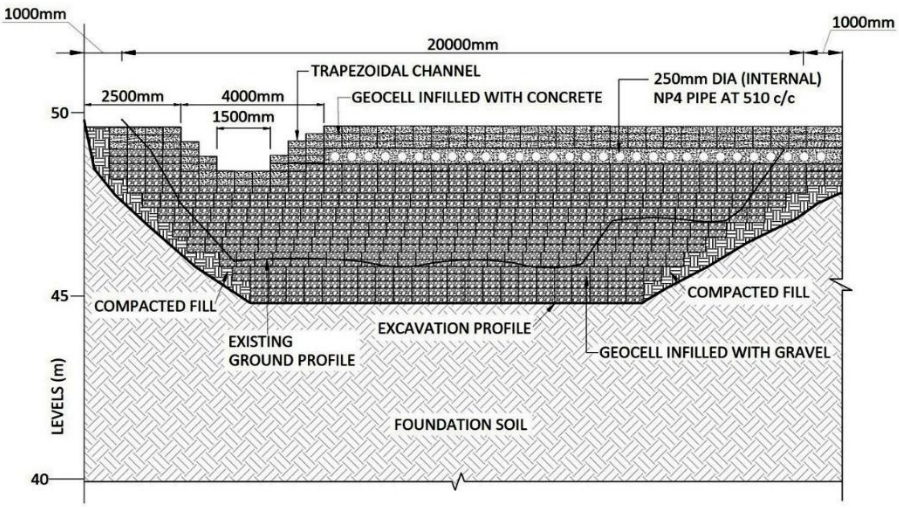


Fig. CD-3: Longitudinal cross-section of the check dam from the upstream side

CONSTRUCTION METHODOLOGY

Fig CD-4 Shows the terrain at the stream section at the selected check dam location, which required dressing prior to construction.

Fig. CD-4: Terrain at the stream at the location selected for the check dam

The construction sequence was as follows:

1. The stream section was dressed to the required profile for the construction of the check dam.
2. The check dam is embedded 1m into the dressed bed and 1m on either side of the slopes to maintain the overall stability of the dam.
3. The cross section of check dam is trapezoidal. To form a trapezoidal cross section, an offset (batter) of 50mm was provided on both the faces of the dam between each lower and upper layer of geocell panels. This was achieved by relaxing the expansion of the panels as required.
4. The bottom width of the check dam is 6.9m and the top width of the dam is 4.9m. These dimensions were based on the requirements of stability.

5. 8mm diameter, 400mm long MS rods were placed in the outer cells of each layer on the upstream and downstream sides so as to hold the geocell panels in position during compaction of infill material.
6. The outermost cells on the upstream side were filled with concrete to prevent ingress of water within the dam and remaining cells were infilled with locally available non-plastic material.
7. In order to create a smooth profile of the upstream and downstream surfaces of the check dam, additional veneer concreting was done on both the faces.
8. 100mm diameter PVC pipes as weep holes were provided at 1.5m vertically and horizontally on the downstream side of the check dam to release pore water pressure (Fig CD-7).
9. spillways were constructed with heavy-duty pipes of internal diameter 250mm, placed at a center-to-center distance of 510mm, along with a trapezoidal channel with a bottom width of 1500mm is provided at one end of the check dam. The invert level of the heavy-duty pipe spillway and the bottom of trapezoidal channel is 1,000mm from the top of the check dam.
1. The top three layers of geocells, i.e., layers above the heavy-duty pipe spillway were infilled with concrete to prevent ingress of water into the body of the check dam.

Fig. CD-5 and Fig, CD-6 show details of construction.

Fig. CD-5: Infilling of geocells with non-plastic soil

Fig. CD-6: Construction of check dam up to spillway level

Fig. CD-7: Provision of weep holes on downstream side

Photographs of the completed geocell check dam after its first monsoon (6 months post construction) and second monsoon season ($1\frac{1}{2}$ year post construction) are shown in Fig. CD-8 and Fig. CD9 respectively. Construction of the check dam using geocells has helped in water storage capacity and charging the ground water, as may be observed from the satellite imagery of the site during non-monsoon season (November-2018) in Fig. CD-10.

Fig. CD-8(a): Upstream

Fig. CD-8(b): Reservoir behind the check dam

Fig CD-8: Check dam, after the first monsoon – 2018 (6 months post construction

Fig. CD-9(a): Upstream

Fig. CD-9(b): Reservoir behind the check dam

Fig CD-9: Check dam, after the second monsoon after construction (2019)

Fig CD-10: Satellite imagery of the Site after the construction of check dam (November 2018))

CONCLUSIONS

Considering the success of the check dam at Satpara, it is concluded that HDPE geocells facilitate rapid construction of gravity structures including check dams.

Geocells being light-weight material, handling is easy and neither mechanised equipment nor skilled workers are required for construction.

The check dam has been provided with adequate pore water relief systems in the form of perforated pipes embedded adequately in the body of the dam.

Heavy duty concrete pipes and the trapezoidal channel at the side have proved adequate as spillways to prevent overtopping of the structure.

Use of geocells infilled with locally available soil has drastically reduced the consumption of concrete to a great extent.

SECTION XI: SLOPE EROSION PROTECTION

(Code: SL)

PREAMBLE

Geocells with low aspect ratio are recommended to prevent erosion of slopes of soil embankments. Erosion along an earth embankment slope may be because of several factors, essentially rainwater run-off, wind, and water seepage from with the soil mass, etc. In order to prevent erosion of soil along the slope, geocells are installed along the slope surface. The concept of protection with geocells is simple and logical.

The challenge is the retention of geocells along a slope under various conditions and constraints. This aspect is partially covered in Section I on landfills, and more is brought out here. Retention of geocells along slopes can best be explained through case studies typical of which are highlighted here. Cases of greening of slopes shrouded with geomembrane along a slope is brought out in Section I, where penetration by supporting anchor stakes is not permitted.

The case studies discussed here include the DLF Golf-Course at Gurgaon in the National Capital Region (NCR) of Delhi, and road embankment slope erosion protection against rainwater runoff at Bogibeel in Assam. With steep gradients, slope protection with geocells proved to be effective as compared to the conventional methods, not only in terms of savings in time and money but also providing environmentally friendly solutions.

EROSION CONTROL

Embankment side slope erosion of the soil along highways is a major issue particularly in areas of heavy rainfall. Stone pitching has been commonly used and Fig. SL-1 is a common enough sight along roads.

Fig. SL-1: Typical damaged stone pitching

Stone pitching has the following disadvantages:

1. It is difficult to lay stone pitching on a slope steeper than 1V: 2H.
2. There are restrictions on quarries and pitching proves to be expensive specially when leads are long.
3. Skilled craftsmen are hard to come by.
4. Laying stone pitching is slow.
5. As can be seen in Fig. SL-1, regular maintenance is required.

MECHANICS OF GEOCELL STABILITY ON SLOPE

The mechanics of support of geocells on slopes is highlighted in Fig. SL2.

The infilled geocell is shown with the typical activating forces down the slope and resisting forces by virtue of friction along the slope which is a function of the friction between the slope soil and the infill, and the downward force normal to the slope surface. Generally, one targets for a safety factor of 1.5 against sliding. However, this can be difficult to achieve and invariably steel, timber or PVC stakes are provided for safety for achieving the required safety factor.

Sometimes an anchor trench is provided at the top of the embankment for anchoring the geocells to mobilise adequate resistance against sliding, providing the net balancing resistance to achieve the desired safety factor against sliding forces.

If the slope is shrouded with a geomembrane that cannot be pierced as in the case of the outer slope of a landfill containment, the geocells are supported by uniaxial geogrids draped down the slope and anchored at the crest. The geocells are suitably connected to the geogrid drape. This system is described in detail in Section I.

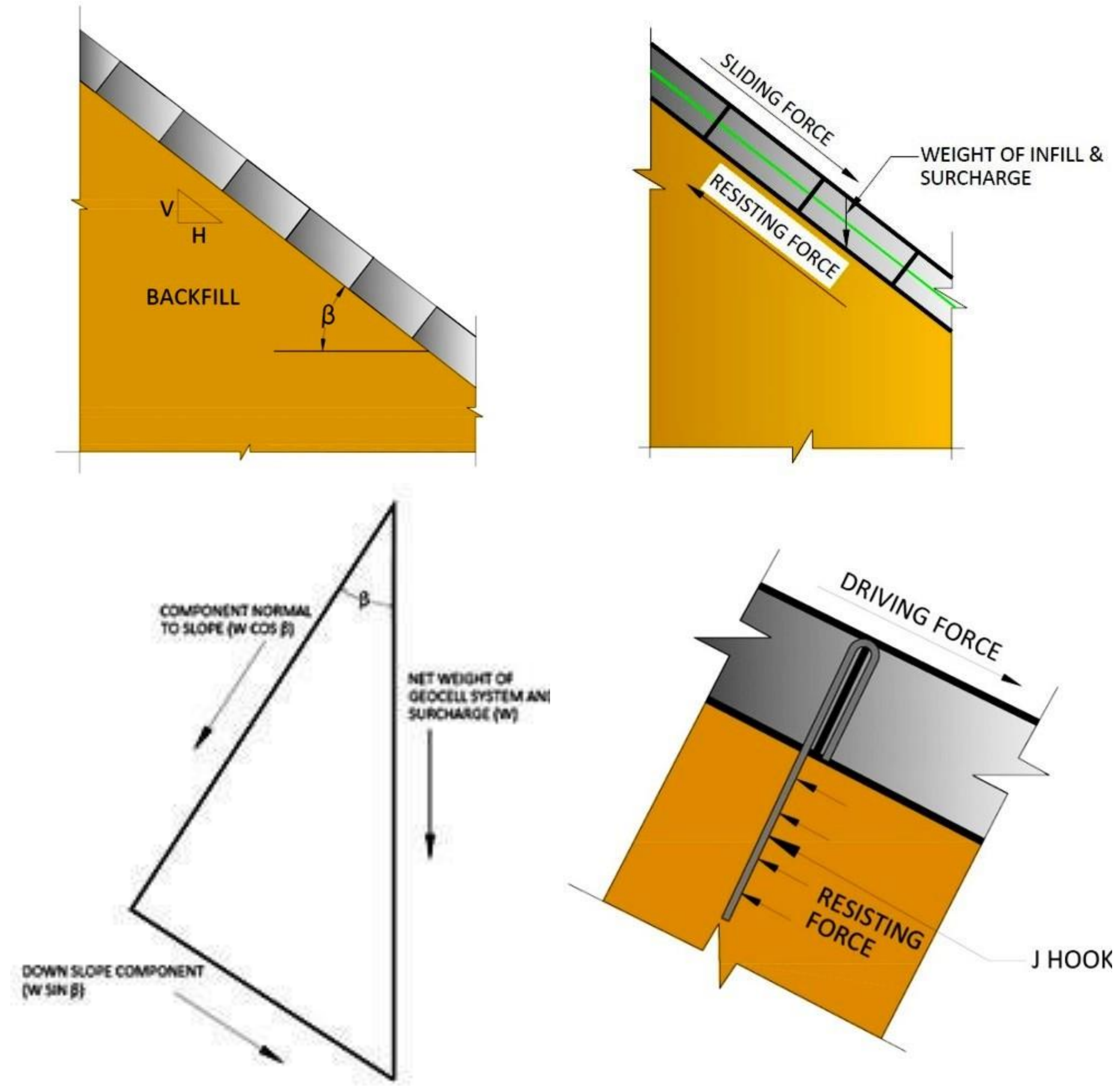


Fig. SL -2: The mechanics of stability of an infilled geocell panel along a slope

THE MECHANICS OF ANCHOR STAKES

The cells of the geocell are in-filled with vegetative soil, gravel, or lean concrete. The vegetative soil infill within the geocell prevents soil erosion and promotes vegetation which gives an aesthetically pleasing look. Because of the slope, a net sliding force act on the entire geocell system. In order to hold the geocells on the slope against net sliding force, the geocells are held in position by anchor stakes / pins spiked into the slope surface. The net sliding force is distributed among the stakes and the anchor trench at the top (if at all provided). Tendons are used to maintain the geometry of the geocells while installing the geocells on the slope surface. The anchor stakes are normally 8mm diameter mild steel rods having an inverted U-shape at the top to hold on to the geocell brim. Quite often, anchor stakes are designed for pull-out capacity, but the net sliding force acts laterally on the stakes. Therefore, stakes should be designed for lateral loads rather than for pull-out. The Lecture Paper recommends design of anchor stakes using the analogy of laterally loaded piles. A typical cross section of slope erosion protection using geocells is shown in Fig. SL-3.

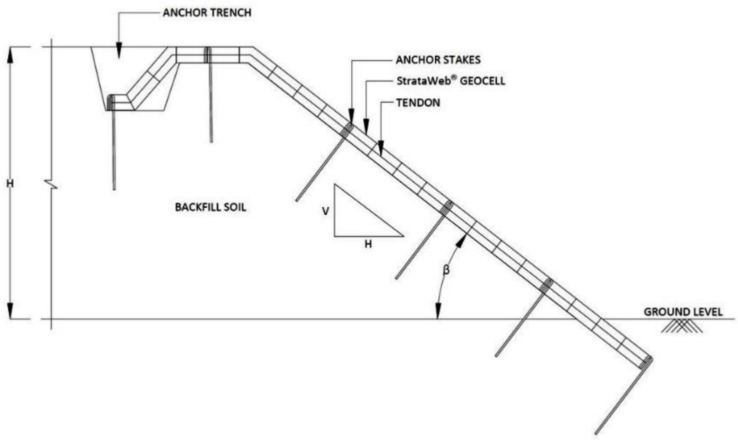


Fig. SL3: Typical cross section of embankment slope with geocell for slope erosion protection

DESIGN

Design of anchor stakes considering pull-out capacity of the stake is inappropriate since the net sliding force acting on the geocell acts perpendicular to the anchor stakes, similar as in laterally loaded piles. Hence anchor stakes are designed analogous to laterally loaded piles and their capacity is determined accordingly. The net sliding force acting laterally on the anchor stakes is shown in Fig. SC-4.

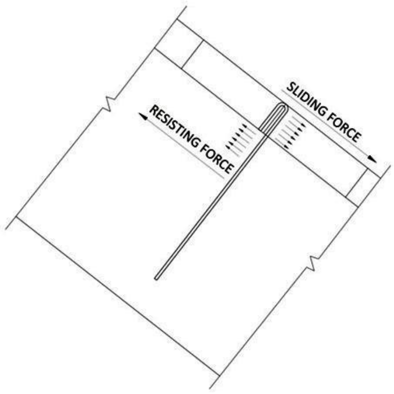


Fig. SL-4: Net sliding force acting laterally on the anchor stake

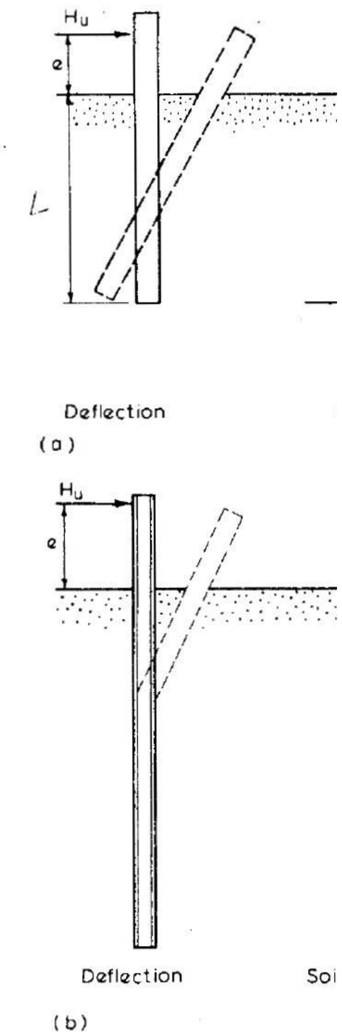
Anchor stakes resist lateral loads perpendicular to the stake by mobilising lateral resistance of soil around it as shown in the Fig. SLP4. Considering the pile analogy, there are various methods to calculate lateral capacity of piles. The governing parameters include type of pile, geometry of pile and soil properties. Ultimate analysis is carried out by either Broms' method or Meyerhof's method. Elastic solution according to Matlock and Reese (1960) is generally used for determining lateral load capacity of vertical piles which are embedded in granular soils.

The Author finds the several assumptions considered in Broms' method closest to the conditions that are applicable to laterally loaded anchor stakes. Hence, Broms' method has been recommended for the analysis to determine load carrying capacities of laterally loaded anchor stakes driven and embedded on an embankment slope.

BROMS' METHOD

Broms (1965) provides a simplified solution based on assumptions of shear failure in soil in case of short piles and bending in the pile governed by plastic yield resistance of the pile section in case of long piles. A major advantage of Broms' method is that it determines the lateral capacity of short and long piles (for both free head and fixed head in non-plastic as well as plastic soils.

Piles can be divided into short and long piles. Short piles are rigid such that they move in the direction of the lateral load by rotation or translation. Long piles are flexible where the top will rotate or translate without movement of the bottom of the pile. The behaviours of short and long piles with free head and restrained head are schematically shown in Fig. SL-5 and Fig. SL-6.



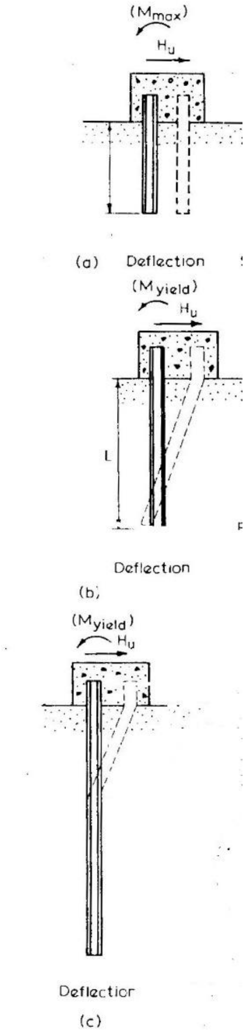


Fig. SL-5: Free head short and long piles   Fig. SL-6: Fixed head short and long piles

Highway embankments are generally constructed from soil with a fair degree of plasticity. Hence analyses of anchor stakes along such embankment slopes would be analogous to short or long piles in what Broms classifies as “c soils”. Broms has evaluated solutions to determine ultimate lateral resistance of laterally loaded short and long piles in “c soils” as seen in Fig. SL-7 and Fig. SL-8 respectively.

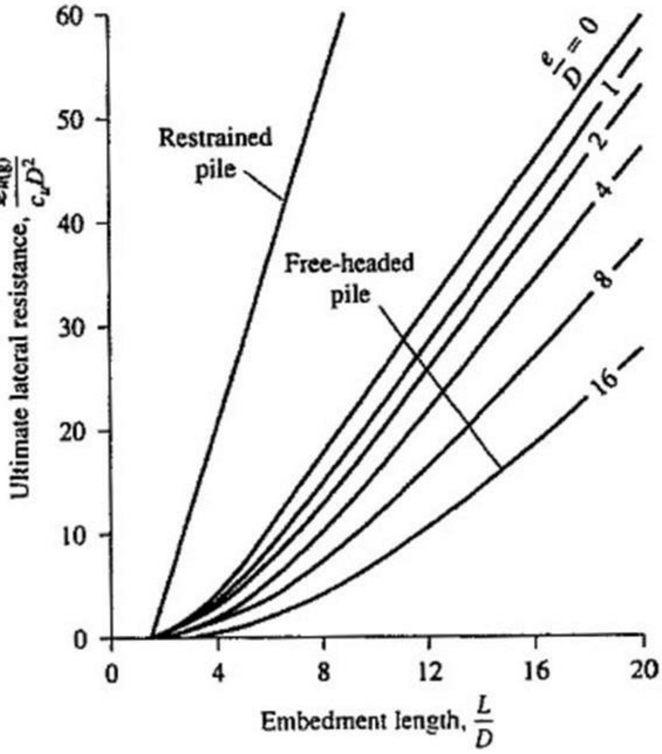


Fig. SL-7: Broms' solution for ultimate lateral resistance of short piles; *e* is the distance between the point of load application and the soil surface

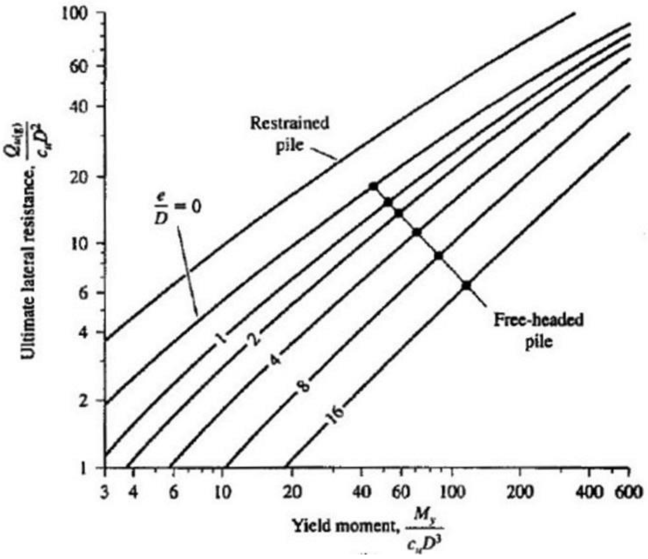


Fig. SL-8: Broms' solution for ultimate lateral resistance of long piles

For erosion solutions along slopes, depth of geocells is commonly 75mm, occasionally 100mm and only in one-off cases, 150mm. For this analysis, the Author has considered the commonly used depth of geocell as 75mm. The total length (L) of the anchor stake considered is 450mm and the diameter (D) is 8mm. The anchor stakes are hooked onto the brim of the geocell and furthermore, the geocell cells are infilled with soil. Hence the anchor stakes are considered as restrained. Considering the chart for Broms' solution for ultimate lateral resistance of restrained long piles in plastic soils i.e., c- soils, lateral load capacity of anchor stakes is calculated as follows:

Assuming that the embankment has a minimum undrained cohesion (c_u) value of 45 kPa.

Depth of geocells is commonly 75mm for erosion protection, hence for the purpose of analysis, depth of geocell considered is 75mm. The total length (L) of the anchor stake considered is 450mm and the diameter (D) is 8mm. By virtue of the soil infill within the cells of the geocells, stakes are assumed as restrained. Considering the chart for Broms' solution for ultimate lateral resistance of long piles in cohesive soils, lateral loading capacity of anchor stakes is calculated as follows:

Assuming that the embankment has a minimum undrained cohesion (c_u) value of 45 kPa.

As anchor stakes' top portion is embedded in geocell, considering the analysis for restrained piles,

$$Yield\ moment\ fctor = \frac{M_y}{c_u D^3} \qquad \text{(SL-1)}$$

where,

$M_y = SF_y$

S = section modulus of the pile (stake) section

$\mathrm{S} = \frac{\pi}{32}\mathrm{D}^3$ and $\mathrm{S} = 5.03 \times 10^{-8}\ \mathrm{m}^3$

F_y = yield stress of the pile material

$F_y = 250\ \mathrm{MPa}$

Hence,

$\mathrm{M_y} = 0.0125\ \mathrm{kNm}$

$Yield\ moment\ factor = \frac{0.0125}{45 \times 0.008^3} = 542.53$

From +Fig. SL-6,

Ultimate lateral resistance is given as,

$$\frac{\mathrm{Q_{u(g)}}}{\mathrm{c_u D^2}} = 100$$

$\mathrm{Q_{u(g)}} = 100 \times 45 \times 0.008^2$

$\mathrm{Q_{u(g)}} = 0.288\ \mathrm{kN}$

Similar analysis has been carried out with combinations of various undrained cohesion values for the embankment soil and diameter of the anchor stakes. The load carrying capacities are captured in Table SL-1 for 8mm, 10mm and 12mm diameter steel bars as anchor stakes in embankment soils with various undrained cohesion values.

Table SL-1: Lateral load carrying capacities of anchor stakes, 8mm, 10mm and 12mm diameter

Embankment Undrained Cohesion (kPa)	Lateral Load Carrying Capacity of Anchor Stakes (kN)		
	Diameters		
	8mm	10mm	12mm
20	0.13	0.20	0.28
25	0.16	0.25	0.36
30	0.19	0.30	0.43
35	0.23	0.35	0.5
40	0.26	0.40	0.57
45	0.29	0.45	0.64
50	0.32	0.50	0.72

Fig. SL-7 illustrates how lateral load-carrying capacities of anchor stakes vary with the undrained cohesion values of embankment soils, for 8mm, 10mm and 12mm diameter anchor stakes. It may be noted that the lateral load carrying capacity of a stake significantly increases with the increase in diameter of the stake as well as with increase in the undrained cohesion of the embankment material.

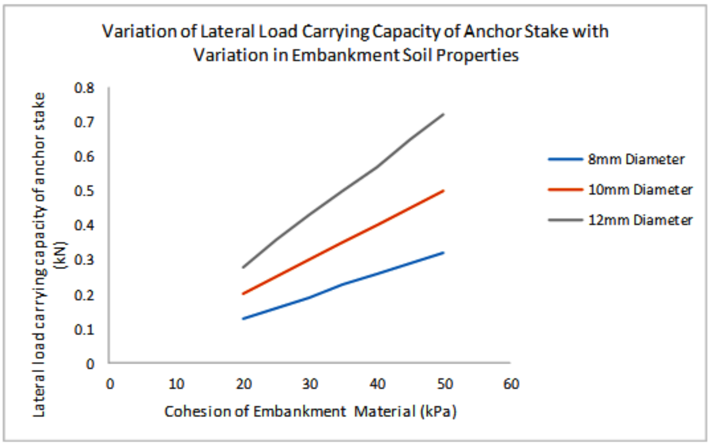


Fig. SL-7: Lateral load carrying capacity of anchor stakes and cohesion of embankment soil

CASE STUDIES

DLF GOLF COURSE, GURGAON

DLF Home Developers Limited was converting an 18-hole golf link into a world class 27-hole facility in Gurugram in 2014. The redesign created undulating terrain to heights as much as 25m to 30m. In some cases, the slope is 2H: 1V and a few as steep as 1H: 1V. Such heights and steep slopes caused heavy soil erosion along the slopes during the rains. Initially, hessian and mulch were tried but proved inadequate during heavy rainfall. Fig. SC-9 2 highlights the initial site conditions with rills and gullies.

Fig. SL9: Initial site conditions

Geocells were proposed over the slopes to arrest soil erosion. Analysis for slope protection using geocells includes several site characteristics as inputs. The length, height and the slope angle and the angle of internal friction of the slope material are important factors in determining the appropriate cell depths and anchoring design.

Prior to selecting an anchoring method, the net sliding force (NSF), i.e., the force that would have to be overcome to keep the system intact from sliding down the slope is calculated. If the NSF is negative, then the friction force between the geocells and the slope is sufficient to hold the system in place. The net sliding force is

$$NSF = [\{H \times L \times \gamma\} \times \{(FOS \times sin\beta) - (cos\beta \times tan\delta)\}] \quad \text{(SL-2)}$$

where

H and L are the height and the length of the slope respectively

γ is the unit weight of the infill soil

β is slope gradient

δ is the lowest value of angle of internal friction of infill soil and base soil

FOS is factor of safety for sliding

A suitable anchor trench was designed to counter the sliding forces. Additional support was provided with tendons and anchor stakes to keep system intact. Size of the trench is determined by Equation (SL-3):

$$L_T \times H_T = (Net\ Sliding\ Force \times Factor\ of\ Safety)/\ (Unit\ Weight\ of\ soil\ \times\ tan\delta) \qquad \text{(SL-3)}$$

where

L_T and H_T are the length and the height of the trench respectively

A typical cross section of the slope is shown in Fig. SL-10. Locally available soil was used as an infill for geocells.

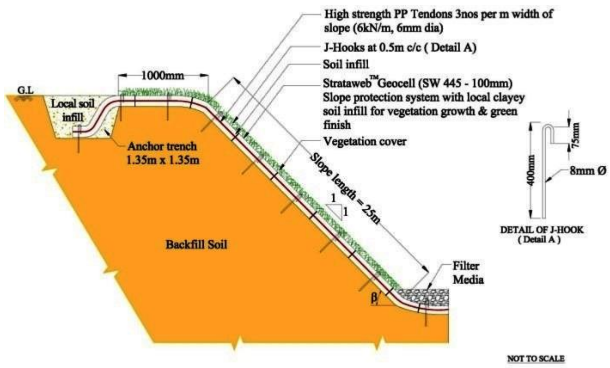


Fig SL-10" Typical cross section details of slope

After analysis, it was decided to use two styles of geocells, weld spacing 660mm and -100mm depth for 2H: 1V slopes, and weld spacing 445mm and 100mm depth for 1H: 1V slopes. Much of the work had already progressed by the time the geocells were introduced. With deadlines faced by the Client, 27,000m^2 geocells were successfully installed within three months, starting end-January 2015. The geocell panels were installed in difficult conditions with the slope was 25m-30m high and the gradient was as steep as 1H: 1V.

The topsoil of the slopes was scrapped off and the surface was dressed and levelled as required and watered and compacted manually. Geocells were laid along the slope by staking within anchor trenches. The expanded geocells were secured together with tendons and fixed against the slope by anchor stakes at appropriate interval. Infilling was done with a backhoe. Seeding and watering were carried out thereafter. Fig. SL-11 shows the construction sequence.

Fig. SC-11(a): Site grading

Fig. SC-11(b): Laying geocells

Fig. SL-11(c): Infilling with soil along steep slope

Fig. SL-11(d): Infilling with soil along slope with trees

Fig. SL-11(e): Vegetated slopes by geocells

Fig. SL-11: Sequence of slope vegetation with geocells

Post construction, the slopes have lush vegetation growth and are intact (Fig. SL-12).

Fig. SL-12: Lush vegetation growth along the slopes

On this Project, application of geocells for slope protection and for fostering vegetation proved to be time-saving and effective. The geocell system worked out to be more economical than the mulch and hessian cloth option. In fact, it was difficult, if not impossible to place mulch and hessian cloth over slopes of 1H: 1V. Vegetation was an essential requirement of landscaping at a golf course.

BRAHMAPUTRA BRIDGE APPROACH OF NH-52B AT BOGIBEEL, ASSAM

The road cum rail bridge across the Brahmaputra on the NH-52B is located at Bogibeel, about 17km downstream of Dibrugarh in Assam. The location is an area of high intensity rainfall. During heavy rains, the surface runoff along the embankment slopes erodes the surface soil, forming rills and gullies down the slope, which widen with more runoff. Fig. SL-13 illustrates the situation in October 2016.

Fig. SL1-3: Embankment slopes in October 2016, eroded by runoff during rainfall

The issue was essentially that of drainage. Several measures for drainage of water from the highway and cattle barriers were designed (and now implemented) by the Assam Public Works Building and NH Department. Besides these measures, the PWD decided to protect the side slopes further with geocells. The geocells would be infilled with compacted local soil and turfed with local "Uloo" grass. Strata was approached for design of the geocell system and supply of StrataWeb® SW 445 100, i.e., weld spacing 445mm and 100mm depth. The design is illustrated in Fig. SL-14.

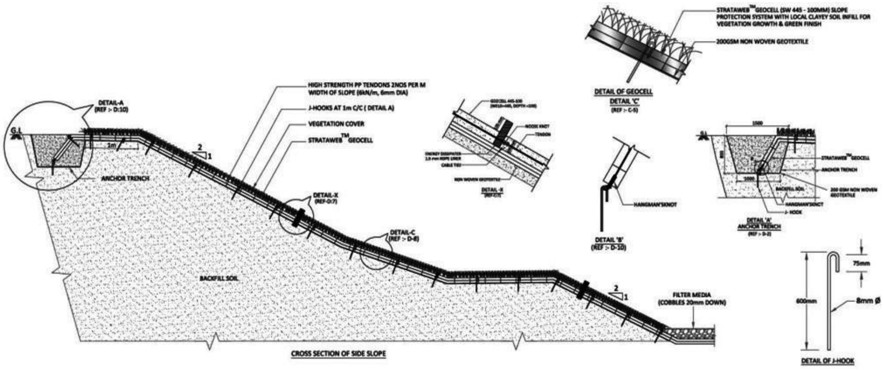


Fig. SL-14: Design and detailing of slope protection with geocells

The geocell efficacy was demonstrated over a stretch of 20m where rain cuts and side slope erosion were observed to be the highest in the flood inundated area, at the South bank of the Brahmaputra. The geocells were laid as per the Drawing (Fig. SL-14). After the infilling was done, Uloo grass was planted on the infilled geocells.

It is reported that the site, which was continuously devastated, has shown no signs of degradation notwithstanding heavy rains and floods. Conditions in June 2016, during the heavy rains and floods are highlighted in Fig. SL-15.

Fig. SL-15: The 20m geocell protected stretch (circled) and a profile of the turfed slope

CONCLUSIONS

It has been proposed to apply the laterally loaded pile analogy to determine the capacity or adequacy of anchor stakes to hold the erosion protection geocell system along an embankment slope. Computations with pile analogy are more realistic than those based on pull-out capacity of an anchor stake, since the net sliding force from the geocell is parallel to the slope, and perpendicular to the anchor stake. The method takes into account the diameter of the stake as well as the undrained shear strength (cohesion) of the soil within which it is embedded. The proposed method will provide site-specific and safe solutions which also facilitate optimisation from cost considerations.

The two cases, one of them on premium NCR land, and the other at a bridge approach on a highway of strategic importance, highlight that retaining soil cover on steep slopes, even under inundation, is possible. By containing soil within the cells, geocells help prevent erosion and foster vegetation even in difficult conditions. Roots of vegetation sustained by geocells help sustain the soil of the slope even better.

Installation of the geocell system is quite rapid. This saves time and money when compared with conventional methods.

SECTION XII: RESERVOIR LININGS

(Code: RE)

PREAMBLE

Reservoirs and canals need to be lined for several reasons. As in the case of protection of slopes against erosion, geocells have been successfully used as base material to retain the lining material, both along the side slopes and bed.

FUNCTIONS OF LININGS

The lining is required for the following reasons:

1. Prevent scour and erosion of side slopes and bed.
2. Provide a smooth surface to improve the flow characteristics in canals.
3. Reduce water leakage by a considerable extent,
4. Protect the barrier geosynthetic, normally HDPE geomembrane from damage and uplift (HDPE is lighter than water).
5. Prevent of vegetation growth along the side slopes and the bed, which can hamper flow of water and affect the quality of the water being transmitted or stored.

GEOCELLS AS LINING SUPPORT

Concrete linings are ideal to serve the above functions. However, concrete used directly as liners have the following drawbacks:

1. Concrete needs to be cast in situ, which may not be easy to lay along a side slope.
1. To prevent thermal and other forms of cracks, the cast in situ panels need to have nominal steel reinforcement.
2. To prevent cracking doe to thermal stresses and differential settlements, the concrete lining needs to be cast in panels. The joints may become the root cause of leakage. With the geomembrane below, the seeped water between the panels and the geomembrane can cause deformation of the panels during sudden drawdowns.
3. Panel joints filled with mastic would require regular maintenance.

These issues have been successfully resolved by laying geocells along the canal surface and infilled with concrete.

In water containment structures with geomembrane barrier linings, anchor stakes cannot be used. Hence as in the case of landfill containment slopes, the geocells are attached to uniaxial geogrids (with machine direction along the dip of the slope). The geogrids are anchored at the top. The geocells used are generally with large weld spacing.

CONCRETE INFILLING WITHIN GEOCELLS – CRACK WIDTH

When concrete is used for infilling, supporting infilled geocells with geogrids is not advisable since green concrete with pH in excess of 9 will damage the PET geogrid. One ignores the protection provided by the coating. The geocell will require anchoring at the crest.

One concern of infilling geocells with concrete is the crack width between the geocell wall and the set concrete. It is to be noted that the walls of the geocell are textured and perforated and the chances of crack formation are remote. However, assuming that the geocell walls are smooth and unperforated, the following computation proves that the crack width is negligible.

The commonly used geocell fir the purpose of reservoir. canal lining is that with weld spacing of 712mm and depth of 100mm.

When expanded, the diagonals of a single cell measure to a standard 475mm x 508mm (Fig. RE-1). Hence, ignoring the curvature of cell walls, the sides measure 336mm x 360mm.

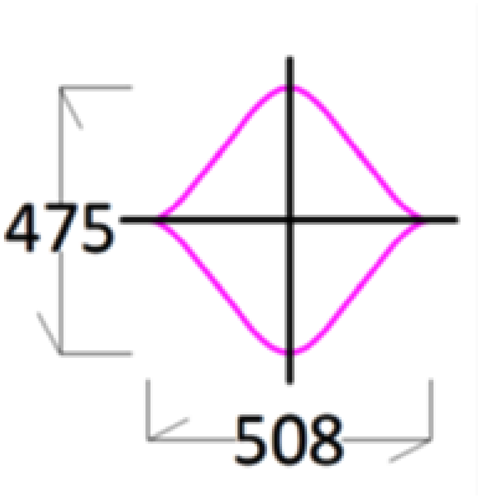


Fig. RE-1: Cell diagonal dimensions for weld spacing of 712mm

Thermal shrinkage of concrete within the cell of the geocell is determined by the basic thermal equation:

$\Delta L = L * \alpha * \Delta T$ (RE-1)

Where

ΔL is the thermal shrinkage due to temperature variation ΔT

L is the maximum dimension of the side of the concrete, i.e. 360mm

α is the coefficient of thermal expansion of concrete, i.e. 10×10^{-6} / ^oC

ΔT is the temperature variation, for the case of Abu Dhabi UAE, 60^oC

For $L = 360$mm, thermal shrinkage ΔL computes as 0.216mm.

Hence along the periphery the shrinkage crack is of the order of 0.108mm.

Considering the smaller dimensions of the concrete slab as compared to a conventional concrete slab considered for canal / reservoir lining, the crack width is comparatively small. Furthermore, owing to the texturing of the geocell walls, there will be intimate adherence to the cell wall. The HDPE of the geocell is flexible material and can suitably contract and expand with the expansion and contraction respectively of the concrete. Hence while theoretical computation shows a crack width of 0.1mm, the actual width will be far less, if not 0.00mm. Even then, as a matter of practice, a geomembrane or equivalent material is spread below the geocell-concrete liner. The geomembrane serves the purpose of a barrier as well as a separator.

Hence shrinkage cracks are not an issue.

CASE STUDIES

The following case studies highlight reservoirs and holding ponds lined with geocells infilled with concrete. These projects cover vast areas and reflect the challenges in concreting the geocells, particularly over the bed and lower reaches of the side slopes. Sequencing of laying the geocells and concreting required planning. Using the concrete placing boom is a great advantage from quality considerations and time.

ABU DHABI MUNICIPALITY: STORM WATER HOLDING POND

Geocells were placed over a layer of StrataWrap™ impervious sheet and anchored at the crest. A major challenge on this Project was to limit crack width of the concrete for a wide temperature variation. The calculation for crack width highlighted above relates to this Project, where the Consultant required that a temperature variation of 60^{o}C be considered for the pond in empty condition.

Progressive pictures of the geocell-concrete lining construction are shown in Fig. RE-2.

Fig. RE-2(a): Geocells on slope

Fig. RE-2(b): Bed being concreted

Fig. RE-2(c): The concrete placer boom at the bed

Fig. RE-2(d): The overall picture

Fig. R-E2: Abu Dhabi Municipality storm water holding pond concreting

ARS Metal Captive Power Plant, Tamil Nadu raw water reservoir

The raw water reservoir for the ARS Netal captive power plant was lined with geocells infilled with concrete, placed over untextured HDPR geomembrane sheets. The sequence of construction is highlighted in Fig. RE-3.

-

Fig. R-E-3(a): Geomembrane base

-Fig. RE-3(b): Geocells laid on slopes and anchored at the crests

Fig. RE-3(c): Concrete infilling

Fig. RE-3(d): The completed concrete lining

JOCIL RAW WATER RESERVOIR

As in the previous case study, concrete infilled geocells were placed over untextured HDPE geomembrane. Construction aspects are highlighted in Fig. RE-4.

Fig. RE-4(a): Geomembrane base

Fig. RE-4(b): Placing geocells in position

Fig. RE4(c): The completed reservoir filled with water

OPG Power Plant, Gandhi Dham raw water reservoir

The concrete infilled geocells were placed over untextured HDPE geomembrane sheets. The sequence of constructing the lining is highlighted in Fig. R-E5.

Fig. RE-5(a): Original ground conditions

Fig. RE-5(b): Geocells laid on geomembranes overdressed slope

Fig. RE-5(c): Infilling with concrete

Fig. RE-5(d): The completed reservoir

Fig. RE-5: Construction sequence of the OPG reservoir

CONCLUSIONS

Geocells with large weld spacing and plain concrete infilling of M-15 grade have been successfully used for man-made water bodies. The cells of the perforated and textured geocells hold the infilled concrete in dry as well as submerged conditions. Owing to the small size of individual slabs, the crack width is of the order of 0.01mm only and is likely to be less in practical conditions owing to the texturing of the cell walls.

SECTION XIII: GEOCOMPOSITE DRAINAGES FOR REINFORCED SOIL SYSTEMS

(Code: DR)

PREAMBLE

Controlling ingressed water and preventing build-up of excess pore water pressures have always been major challenges in any soil structure system and its foundations. Hence drainage is an important aspect in the design and detailing of unreinforced and reinforced soil systems.

The conventional drainage within any soil structure, reinforced or otherwise, is the graded filter bay and / or blanket, comprising of graded natural material, often two or three types in layers in a designed drainage. This system has been specified since over hundred years, along with other methods such as perforated or open jointed pipes. However, there are inherent disadvantages and areas of dispute:

1. Gradation of filter materials is one design aspect which is rarely followed in current practice owing to wide empirical margins along with quality issues at site. More often than not, if the fill material is essentially (non-plastic) fines, a nonwoven geotextile is recommended as separator with only grammage specified, but neither the essential apparent opening size (AOS) nor the permeability.
2. Placing vertical or inclined bays of graded granular materials is difficult. This is particularly so where horizontal soil reinforcements are to be considered. Invariably such bays are immediately behind the block or panel fascia of a reinforced soil structure. If the congruent soil on the upstream side of the water flow is fine grained, a separator geotextile is essential. This renders placement more difficult.
3. Getting the required grade(s) of granular filter material is cumbersome on an infrastructure development site. Furthermore, the gradation of the upstream congruent soil may vary, location to location on a project. This would require simultaneous review of filter gradation.
4. From a practical point of view, contamination of the filter medium / media is difficult to avoid altogether.
5. Issues highlighted above require frequent and rigid quality control checks.
6. Granular materials conventionally used as drainage media are scarce. Such materials are often difficult to procure even over long leads. Furthermore, there are Government restrictions on quarrying, and in some States, even ban on exploitation of sand and gravel.

These issues underscore the need to consider geosynthetic composite drainage systems. Such geocomposites are in common use and need no elaboration in this Section. This Section highlights certain innovative applications of geocomposite drainage systems in India, covering certain site conditions.

TYPES OF DRAINAGE

Drainage systems include:

1. External Drainage:
   a) Along the structure surface.
   b) At the end of an approach ramp.
   c) At the toe of the reinforced soil system.

2. Internal Drainage
   a) Within the backfill of a reinforced soil retaining wall system.
   b) At the interface of the backfill and the reinforced soil fill.
   c) Within the reinforced soil fill:
      i. Behind the fascia.
      ii. Transverse across the body of the reinforced soil.

CONSIDERATIONS FOR DRAINAGE DESIGN

At any point of time, damage due to water can be due to:

1. Build-up of hydrostatic pressures against which the structure has not been designed for.
2. Unrouted and arbitrary flow of water that can cause erosion, scour and undermining of the structure.
3. Both of the above.

Drainage system is required to reduce, if not eliminate, development of hydrostatic stresses within the soil structure which would otherwise require appropriate design. Ignoring provision of appropriate drainage while neglecting designing for hydrostatic stress build-up has led to catastrophic failures of reinforced soil approach ramps on various classes of roads including major highways.

Unrouted surface flow can cause erosion or undermining, which would be detrimental to the stability of the structure particularly during construction itself. Drainage issues during construction can be quite different from post-construction and over the life of the structure. Construction activities for highway ramps invariably stretch through the monsoon months. In regions of high rainfall, drainage during construction needs particular attention and detailing, even though temporary, particularly where the fill is fine grained (and obviously non-plastic). However, the drainage system devised should not foul with the construction activities to the extent possible.

Each project site has its own nuances particularly when the subject is evacuation of water and relieving excess pore water pressures. Hence every project site will require its unique solution for drainage and pore water pressure relief, using appropriate basic systems. Conditions often exist which may need inclusion of a combination of basic systems and innovation is the key.

The following are conditions requiring drainage to reduce or eliminate hydrostatic and pore water pressures.

1. Surface water flowing towards the structure.
2. Catch basins / drop structure within the system.
3. Fine grained soil as reinforced fill.
4. Snow melt saturation along the Himalayan reaches.
5. Heavy precipitation.
6. Rise in ground water table.
7. Drainage outlets from the structure.
8. During construction, particularly where the reinforced fill and / or backfill is fine grained (and non-plastic).

Much of these are catered to by conventional practice. This Paper confines to innovative methods considering geosynthetics only. The geosynthetic drainage media considered are:

1. Geonet, a composite of nonwovens sandwiching a highly pervious core.
2. A geo-composite comprising of a nonwoven blended on to a uniaxial or biaxial geogrid.

INTERNAL DRAINAGE WITHIN REINFORCED SOIL

BEHIND MODULAR SEGMENTAL CONCRETE BLOCKS FASCIA

As a rule, a drainage bay is essential behind precast concrete panel or modular precast concrete block fascia. This bay is of high permeability material as compared to the reinforced fill. Hence the bay functions as a sink to collect and transmit the ingressed water from the reinforced fill and route it out of the system through vertical and horizontal joints of the precast concrete units of the fascia.

Traditionally gravel, being a highly pervious and self-compacting material (an essential characteristic since any form of mechanical compaction will displace the fascia) is the preferred material for the drainage bay. However, gravel is scarce, and it also requires a deeper insight into its gradation, particularly when used with fine grained soils such as non-plastic silts and pond ash.

The current trend is to use a drainage geo-composite, a geonet in lieu of gravel. The general make-up of the geonet is a highly pervious polymer core sandwiched between two layers of non-woven geotextile. While there is a tendency to specify these nonwovens by their grammage, the appropriate specification should be according to the field application requirements, essentially puncture resistance and AOS. A typical geonet is shown in Fig.DR-1.

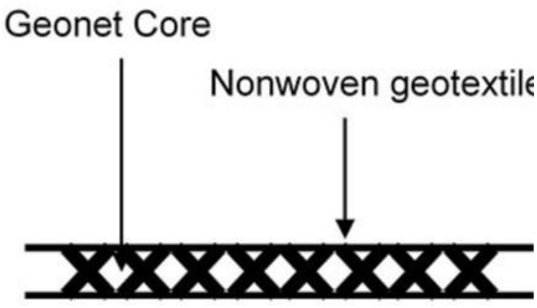


Fig DR-1(a): Typical schematic of a geonet composite

Fig DR-1(b): StrataDrain™ geonet; note the impression of the core within (courtesy Strata Geosystems)

Fig DR-1(c): Typical core (courtesy Strata Geosystems

The edges of the geonet strips comprise of the outer two nonwovens only, flaps without the core. This is essential for lapping / stitching when two geonet strips are placed in series for continuity, or in parallels for wider coverage. When in series, the two core ends are butted together and the flaps are lapped where possible.

Fixing geonet drainage behind panel fascia may not be of innovative significance, where openings are made to accommodate the connector systems and suitably covered to prevent fines from entering the geonet. The current challenge is to place geonet behind segmental concrete blocks particularly at the horizontal joints where the continuous geogrid reinforcement is connected.

A procedure to place the geonet effectively behind the segmental concrete blocks is proposed:

The geonet strips are placed in horizontal rows against the blocks. The core width of 200mm will be placed against the segmental block height of 200mm as seen in Fig, DR-2. Flaps are provided along the length of the geonet strips, both sides of the width for parallel strip lapping. At the reinforcement level, the flap adjoining the block will be folded inward and tucked down. A gravel bay of cross section size 100mm x 100mm is placed at the geonet-geogrid junction within the reinforced fill material as seen in Fig. DR-2.

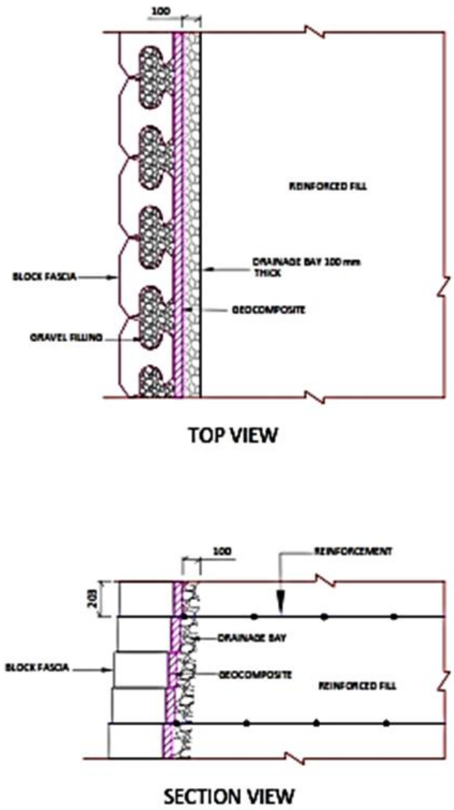


Fig. DR-2: Typical detail of geonet as drainage bay behind a segmental concrete block fascia

After placing the geogrid in position and keyed by placing the upper row of blocks, the geonet strip just above the geogrid reinforcement is placed. The fascia side lower flap is tucked inward and the upper gravel bay, 100mm x 100mm is placed as in Fig. DR-2.

The objectives of the proposed detailing are as follows:

1. This detailing is particularly essential for fine grained non-plastic material, commonly used as reinforced fill. These include non-plastic fine sands and silts, common to the Punjab and the Indo Gangetic belts. Pond ash is also being commonly used owing to its non-plastic characterisation and its consistent excellent consolidated and detained friction angle. To a large extent, the gravel bays prevent fines from eroding into the core and reduce the drainage efficiency of the geonet.
2. The gravel bays provide continuous drainage path between the two adjacent strips in parallels, which cannot be butted owing to the geogrid reinforcement in between. The water will effectively drain through the stone columns (Fig. DR-3) formed by the profile of the segmental concrete blocks infilled with granular material. The gravel bay also ensures continuity of the drainage bay, as in the traditional drainage bay system.

3. Settlements of the reinforced fill can create detrimental kinks in the geogrid, particularly where the reinforced fill is fine grained, or pond ash. The gravel bays are comparatively noncompressible. Hence these bays render a smoother downward profile to the geogrids at the fascia instead of a sharp kink.

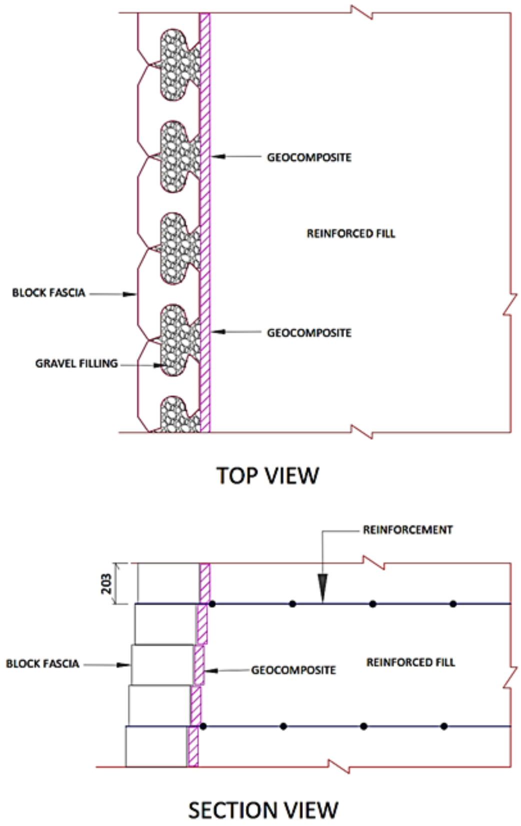


Fig DR-3: Gravel Bay and the segmental block stone column

Horizontal bands of geonets may be provided to relieve built-up pore water pressures. However, from the construction point of view, every layer of geonet is an additional activity that demands additional time on the project schedule.

Designs with block fascia often require secondary reinforcement to cater to local internal stability. An innovative solution is to provide a draining non-woven mat adhered onto / blended with the geogrid secondary reinforcement of the required style, as illustrated in Fig. DR-4. While the geogrid fulfils the requirements of secondary reinforcement, the non-woven will have adequate drainage properties to route the water to the drainage bay / geonet behind the block fascia.

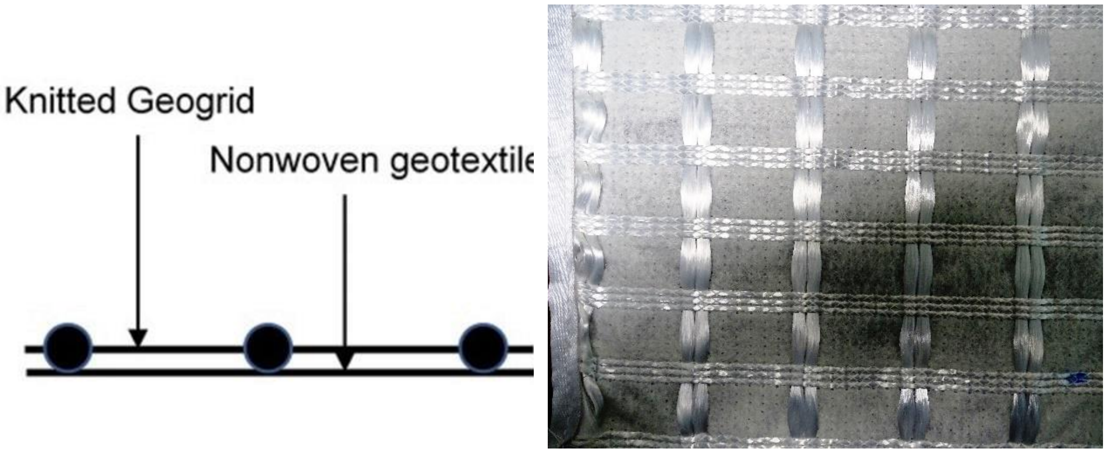


Fig. DR-4: Typical geogrid-nonwoven composite (Courtesy Strata Geosystems)

Likewise, this composite may be considered as primary reinforcement while considering only one side of the geogrid interacting with the fill material. The short-term strength of the geogrid will be selected according to design requirements. However, the entire sets of tests to determine connection strength and pull-out strengths need to be conducted for design parameters.

A composite comprising of high strength biaxial geogrid and nonwoven of appropriate permittivity may be considered as basal reinforcement. The nonwoven may also function as a separation layer. However, where drainage is concerned, for the basal application, the geonet discussed above will be more effective. Suitable detailing will be required for designing the outlet for the routed water and simultaneously provide adequate anchorage for the geogrid that will be subject to tensile forces due to incipient failure along the critical slip circle.

It is prudent to place the geo-composite with the nonwoven surface at the top to reduce the installation damage to the geogrids.

CONSCLUSION

Unless hydrostatic conditions prevail more as a rule, hydrostatic pressures are not considered in normal design of a reinforced soil structure. However, there can be conditions when water may seep into the system. Excess hydrostatic pressures / pore water pressures can damage the structure; there are several case histories to prove this point.

Owing to poor drainage characteristics, fine grained non-plastic soils such as fine sands and silts, and pond ash used as reinforced fill are susceptible to excess pore water pressures. An intermediary drainage would accelerate pore water pressure dissipation. The two methods suggested would help in pore water dissipation, and also ensure that the construction schedule is not jeopardised.

EPILOGUE

(Code E)

While realising the potential that polymeric geosynthetics have for us, one must realise that our task is not complete. One cannot but help relate the last verse of Robert Frost's "Stopping by Woods on a Snowy Evening":

> The woods are lovely, dark, and deep,
>
> But I have promises to keep,
>
> And miles to go before I sleep,
>
> And miles to go before I sleep.

While applying geosynthetics to civil engineering issues, one cannot remain complacent with current developments. There is a lot of promise ahead with geosynthetics; new basic polymers, new product designs – the sky is the limit for innovation.

Unfettered innovation must continue relentlessly and must be encouraged by the various Ministries and statutory bodies such as the Bureau of Indian Standards, and the Indian Roads Congress, which must be broad minded and open to new progressive ideas.

As mentioned in the body of the Paper, a major task pending is to determine the "zone of influence" for the modified M_R with types and styles of geogrids and geocells in various standard pavement components and other materials.

The Tryst is ongoing. Yes, a lot more remains to be done……

ACKNOWLEDGEMENT

Even after 49 years of professional life immersed in Geotechnical and Civil Engineering, this Honour is certainly not the end; I see this as a beginning to a whole new discipline, and I am proud to be a part of its genesis.

Having been selected to deliver the Lecture at this prestigious forum is indeed a privilege, and I see this as an honour of my lifetime. For this, I express my deep gratitude to the Indian Geotechnical Society and to the IGS Mumbai Chapter for its unstinting support all along.

This Lecture Paper was possible only with the encouragement, ideas, and support of the Management of Strata Geosystems (India) Private Limited. I have freely indulged into Strata's Case Studies, and innovative ideas and applications. After all, the themes of this Lecture Paper are Conservation of natural resources and Innovation; two of the three key mantras at Strata Geosystems, the third being Quality.

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