Guidelines on Geo-synthetics for Coastal Protection and Port Works

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NATIONAL TECHNOLOGY CENTRE FOR PORTS WATERWAYS AND COASTS (NTCPWC)

SAGARMALA PORT-LED PROSPERITY

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Preface:

Based on the discussion held with the Director - Technical Development Wing under Ministry of Shipping Govt. of India, Department of Ocean Engineering and NTCPWC, IIT Madras have prepared the guideline for using Geo-Synthetics in ports, coasts and waterways.

The draft guideline is prepared by an Expert Panel consisting of Prof. R. Sundaravadivelu FNAE, Emeritus Professor, Dept. of Ocean Engineering, IIT Madras, Prof. K. Murali, Head, Dept. of Ocean Engineering, IIT Madras & Chairman of NTCPWC, Prof. Nilanjan Saha, Dept. of Ocean Engineering, IIT Madras, Mr Guru Vittel, CRRI and members from industries and government organisations.

The draft guideline was circulated on 29.07.19 to all the major ports, Maritime Boards, anti sea erosion division and Fishing Harbour division in coastal states and the Chief Engineer Andaman & Lakshadweep Harbour Works, to submit their comments/suggestions.

The comments and suggestions are incorporated in the revised version of the guideline. The use of geosynthetics is included in IS4651- Part 2 in 2020. In addition, the panel has decided to conduct a webinar to enable broader discussions for fine-tuning the revised version of the guideline.

The Department of Ocean Engineering organised a two-day webinar in association with NTCPWC on the 10th & 11th of December 2020. Elite panelists from various Educational Institutes, Major Ports and geotextile industries and about 60 engineers from both Govt. and private sectors attended the webinar.

The panellist presented the various case studies and design methodology. The sessions of the webinar were handled by Prof. R. Sundaravadivelu, IITM, Prof. K. Murali IITM, Prof. S. R. Gandhi, Director SVNIT Surat, Mr Guru Vittel CRRI, Dr Vijaya NIOT, Prof. K. Ilamparuthi Anna Univ (Retd), Prof. Nilanjan Saha IITM, L&T, Prof. Sivakumar Babu G L, IISc and Prof. S. A. Sannasiraj IITM. There was an open discussion with the panellist and attendees on the guidelines.

The comments from the delegates during and post-webinar are incorporated. The webinar presentation is also included. The guideline prepared, includes the following:

i. Role of geosynthetics in coastal protection and ground improvement

ii. Geotextile material (Geotube, Geogrid, Gabion etc.) for different types of structures used in ports, coasts and waterways

iii. Detailed Specification of design and construction methodology for the use of geotextile in ports, coasts and waterways.

Dr. P.V. Premalatha, Dr. S. Sherlin Prem Nishold and Mr. S. Kreesa Kumaran have coordinated the preparation of this report.

The final version of the Guidelines on Geo-Synthetics for Coastal Protection and Port Works dated 30.06.2021, is submitted to the Ministry of Shipping for publication.

Prof. R. Sundaravadivelu

Prof. K. Murali

Prof. Nilanjan Saha

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Disclaimer

This is only a guideline for the application of geosynthetics,

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Abbreviations

AIC - Asphalt Interlayer Composites

AOS – Apparent opening size

b_{fill} -The filled width of the geotextile tube in m.

CBR - California bearing ratio

d_s - Depth of water at the structure toe under the DWH condition.

D_{100} -Diameter of the geotextile tube at 100% filling in m

D - Diameter

D – Water depth

d_{15}, d_{50}, and d_{85} – Diameter of particle size fractions corresponds to 15%, 50% and 85% retention

DT - Double Twisted

DWL - Design water level

f - Degree of filling concerning the cross-sectional area

GEC - Geosynthetic-encased stone columns

h_{fill} -The filled height of the geotextile tube in m.

H_S -Significant wave height in meters (m)

H – Wave height

H_b – Breaking wave height in meters (m)

HDPE - High density polyethylene

HSPW - High strength polymeric woven

HTV – Heavy transport vehicle

JGT - Jute geotextiles

JNPT - Jawaharlal Nehru Port Trust

k_D -Stability coefficient (the stability coefficient is assumed as 2)

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L - Wavelength

LS - Longitudinal section

MARV - Minimum average roll values)

MDD - Maximum dry density

MORTH - Ministry of Surface Transport Specifications for Road and Bridge Works

NHAI -National Highway Authority of India

O_{95} is apparent opening size of the geotextile filter

PET - Polyethylene terephthalate

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PP- Polypropylene

PVC - Poly vinyl chloride

PVD - Prefabricated Vertical Drains

PWD - Public Works Department

RCC - Reinforced cenemt concrete

RECP - Rolled Erosion Control Products

SPV - Special Purpose Vehicle

TRM - Turf Reinforcement Mats

W_r - Unit weight of armour is t/m^3

W_S - Unit weight of seawater t/m^3

W_{50} -Weight of an individual armour unit of gabion box (tons)

Γ - Density of the fill material

θ - angle of the structure measured with horizontal.

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Chapter 1 - Introduction and Terminology

1.1 Introduction

Geosynthetics have dramatically transformed the practice of geotechnical engineering around the world. India is presently experiencing a boom in infrastructure development, which has led to enhanced demand for cost-effective new construction materials like geosynthetics. These products have made it possible to introduce novel concepts for the construction of different civil engineering structures. The usage of geosynthetics offers several advantages like (a) technical superiority with a variety of engineering products/ techniques, (b) faster construction, (c) flexibility of the structures which perform well during earthquakes and (d) better control of the quality of products, in a given application. Geosynthetics can perform one or more of the following functions to improve the structure's mechanical and/or hydraulic behaviour in which it is incorporated – soil reinforcement and slope stabilisation, separation/filtration, drainage, erosion control and moisture barrier (geomembranes).

The bulk of commercially available geosynthetics are manufactured from petro-based synthetic polymers such as polypropylene, polyester, polyvinyl chloride, polyamide, polyethylene, etc. Geosynthetics produced from these materials have good permeability (unless it is a membrane), adequate wear and tear resistance, and a very long life. However, these polymers get degraded when exposed to ultraviolet light for a prolonged period. In addition to synthetic materials, natural materials like coir and woven jute nettings have also been used especially for erosion control applications.

Despite its good track record in many other countries and India, geosynthetics could not become popular in our country. This is due to the non-availability of proper guidelines on design and construction standards, lack of testing facility for geosynthetics, and absence of information/data regarding such structures' long-term behaviour and probably due to higher initial costs.

1.2 Geosynthetics - Classification and Definition

Geosynthetics are synthetic products used in various geotechnical applications for enhancing the serviceability/performance of an engineering structure. Geosynthetics can be classified in many ways based on manufacturing process, materials used or function. The most commonly adopted classification of geosynthetics is based on their function and manufacturing procedure. (Ref: HRB Special Report No. 12[17], 'State-of-the-Art: Application of Geotextiles in Highway Engineering' and IRC SP:59[19] Guidelines for the use of Geosynthetics in Road Pavements and Associated Works)

Geotextile can be defined as permeable synthetic textile used with foundation, soil, rock, earth, or any other geotechnical engineering-related material as an integral part of an infrastructure project or structure. The geotextiles are generally classified by the manufacturing process and grouped into two categories: woven and non-woven. Depending on the manufacturing process, the other types of geotextiles available are (i) knitted geotextiles and (ii) composite geotextiles.

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Woven geotextiles are manufactured by weaving weft threads through warp threads. The strength of geotextile in the machine direction is usually larger than the cross-machine direction.

Non-woven geotextiles are produced from randomly distributed continuous filaments or staple fibres, which are bonded together chemically, thermally or mechanically.

Knitted geotextiles consist of a single strand systematically intertwined with itself and is manufactured with a knitting machine instead of a weaving loom.

Composite geotextiles (multi-layered geotextiles) are manufactured by combining layers of different types of geotextiles. The components can be combined by needling, stitching, chemical bonding or heat bonding.

Geogrids are polymers formed into an open, grid-like configuration, i.e., they have apertures between individual ribs (or strands) in the transverse and longitudinal directions. Geogrids are mainly made from polymeric materials, typically polypropylene (PP), high-density polyethylene (HDPE) and polyester (PET). Geogrids can be classified according to (a) manufacturing process (woven, knitted, bonded, welded, extruded), (b) directional behaviour (uniaxial, biaxial, triaxial), (c) bonding between ribs and (d) Polymer (polypropylene, polyester, PVC, etc.). Biaxial geogrids have significant strength in both the machine and cross-machine directions. In contrast, uniaxial geogrids exhibit the primary strength in the machine direction with minimal strength, enough to maintain the aperture structure in the cross-machine direction.

Geostrips is another form of geosynthetic reinforcement primarily made of synthetic material in strips and is made from high tenacity polyester yarn, contained in a suitable polymer sheath.

Geomembranes is a very low permeability or impervious synthetic membrane liner or barrier used to control fluid migration across the plane. These are made from relatively thin continuous polymeric sheets, but they can also be made from the impregnation of geotextiles with bitumen, polymer sprays or as multi-layered bitumen Geocomposite. Continuous polymer sheet geomembranes are by far the most common.

Geocomposites are drainage composites formed by combining geotextiles or geomembranes with a core of geonet. Prefabricated Vertical Drains (PVD)/Band Drains and Fin Drains also come under the category of Geocomposite.

Geonets consists of integrally connected, parallel sets of ribs overlying similar sets at various angles formed by a continuous extrusion into a netlike configuration for in-plane drainage of liquids. Geonets are often laminated with geotextiles on one or both surfaces and are referred to as drainage composites. There are three categories of geonets.

- Bi-planar geonets: These are the original and most common types and consist of two sets of intersecting ribs at different angles and spacings. The ribs themselves are of different sizes and shapes for different styles.
- Tri-planar geonets: These have parallel central ribs with smaller sets of ribs above and beneath, mainly for geometric stability.
- Other geonets: These newer geonet structures have either box-shaped channels or protruding columns from an underlying support network.

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Geocells (also known as Cellular Confinement Systems) are three-dimensional honeycombed cellular structures that form a confinement system when in-filled with compacted soil or aggregate. They are extruded from polymeric materials, such as HDPE, into strips welded together ultrasonically in series. The cells, the geocell structures, are created in a continuous process without any subsequent welding. The strips are expanded to form the stiff (and typically textured and perforated) walls of a flexible 3D cellular mattress. The material is UV stabilised with carbon black.

Geomats are two dimensional or three-dimensional mats with specified thicknesses, made of multifilaments with layers of geogrids folded and knitted or bonded together with apertures to allow vegetation growth for erosion control application. The geosynthetic mat consists of UV stabilised non-biodegradable polypropylene/ polyethylene or similar polymer fibres, which is extruded or heat bonded to provide a dimensionally stable matrix. A tension element like steel wire mesh or geogrid shall be included in these mats as reinforcement, where these mats are required to possess more strength against erosive forces, like in steep slopes or heavy rainfall areas.

Bituminous Pavement Reinforcement Materials: There are three products for strengthening of bituminous pavement layer, which are described below:

1. **Paving fabric** is made from the fibres with the non-weaving process (needle-punch and heat-bonded) and applied by providing tack coat over bituminous layer.

2. **Paving grids / Glass-Fibre grids** are manufactured from glass fibres. These are resistant to chemical attack (i.e., from flux oils, paraffin's or any other solvents used in bituminous binders), mildew and rot. They are employed for reinforcing the bituminous pavement layer.

3. **Asphalt Interlayer Composites (AIC) / Composite Paving girds** comprise of paving grid combined with a paving fabric is called a Composite. AIC are made from non-woven geotextiles knitted with fibre-glass yarn to provide the dual function of reinforcement and a crack arresting layer.

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Chapter 2 - Coastal Erosion and Role of Geosynthetics

2.1 Coastal Erosion

Coastal beach erosion occurs in various forms around the world. This phenomenon gets more acute during cyclones and, in turn, causes damage to infrastructure facilities, including roads. This is due to the severity of waves and storm surges, which result in coastal erosion. Coastal erosion (and accretion) happens because coastal sediments are constantly in motion as an effect of tides, waves, winds, and currents. Coastal structures are built to prevent erosion of shorelines and restore the eroded beaches to their initial phase. The conventional coastal structures (i.e., breakwater, groynes, revetment, and seawalls) have been constructed using wood, rock, and concrete. The basic approach to mitigate coastal erosion related problems is to provide suitable cover to the soil. The measures to control coastal erosion can be categorised as structural and soft/ non-structural. These can be taken up together or separately also. Structural measures for arresting coastal erosion are sea wall, revetment (rock armour, gabion mattress or precast concrete block revetment systems), offshore breakwater, groynes, etc. Soft measures generally adopted to prevent coastal erosion are artificial nourishment of beaches, vegetative cover such as mangrove plantation, etc.

A rubble mound structure consists of a mass of rock material, generally of low permeability at the centre and gradually modifying to larger stable material on the outside, up to a suitable crest height. This structure limits an acceptable degree of overtopping and the transmission of wave energy and provides stability under the severest wave attack.

Generally, in conventional solutions such as simple rubble mounds (as shown in figure 2.1), the core consists of general rock fill and outer armour protection for long term design against wave and current attacks. Instead of providing a rock armour layer, the latest and environmentally friendly technologies, e.g., geosynthetics, can also be adopted for the construction of the armour protection layer. The rock-fill core can be replaced by geotextile products such as bags or tubes covered by rock armour or shroud armour.

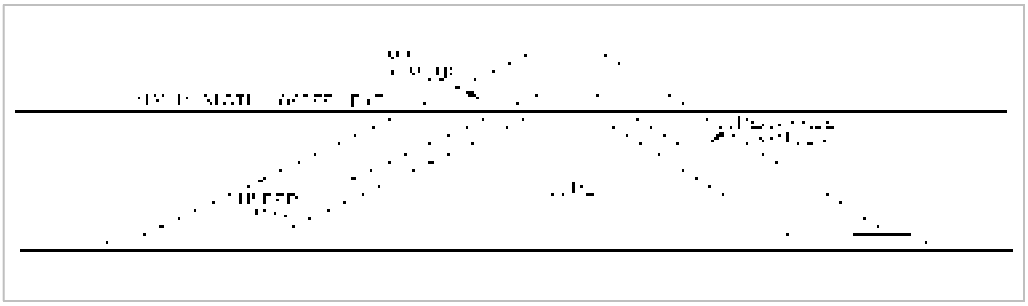

Figure 2.1 Simple rubble mound construction

2.2 Wave Generation in Sea

A disturbance of the water surface causes waves. These waves become prominent during cyclones because of surges and high-speed winds. Most waves are generated by wind. After waves are formed, they can propagate across the surface of the sea for thousands of miles. When waves break on a shoreline or coastal structure, they have fluid velocities and accelerations that can impart significant forces. The wave period of individual waves remains

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constant through the transformations until breaking, but the direction of propagation and the wave height can change significantly. As a wave moves into shallower water, the wavelength decreases and the wave height increases. Waves break at two general limits:

- In deepwater, waves can become too steep and break when the wave steepness defined as H/L, approaches 1/7 (where H = height of the wave, i.e., the distance between the crest of the wave and water surface, L = wavelength defined as the distance between two successive wave crests).
- In shallow water, waves break when they reach a limiting depth (d) of water.

This depth-limited breaking is essential in the design of coastal revetments protecting highways. For an individual wave, the limiting depth is roughly equal to the wave height and lies in the range given below:

$$0.8 < (\frac{H}{d})_{Max} > 1.2 \quad (2.1)$$

A practical value of wave height that can be considered when there is mild sandy slope offshore is:

$$(\frac{H}{d})_{Max} \approx 0.8 \quad (2.2)$$

2.3 Systems for Protection of Coastline Against Sea Erosion

The systems adopted for protection against water erosion comprise two different parts – the outer revetment or armour layer to absorb the hydraulic energy of water flow velocity and/or the wave energy; and the inner part of the filter layer. Revetment systems in rip-rap blocks, prefabricated concrete elements or gabion mattresses or RCC/stone masonry walls are most commonly used as armour layers. The function of the inner filter layer is to prevent soil particles from being eroded and allow the free escape of internal water simultaneously. Conventionally several layers of granular material with well-designed grain size distribution and thicknesses are used for this purpose. Geotextiles can be successfully adopted to replace such granular filter material. They are now increasingly adopted due to various technical advantages, cost benefits, ease of installation, faster completion of the project, and superior long-term performance.

2.3.1 Bulkheads and revetments

The distinction between revetments, seawalls, and bulkheads is one of functional purpose. Revetments are layers of protection on the top of a sloped surface to protect the underlying soil. Seawalls are designed to protect the beach against large wave forces. Coastal revetments or seawalls are generally constructed to protect the toes of coastal cliffs, bluffs, dunes, etc., and fortify coastal embankments and flood levees. Bulkheads are designed primarily to retain the soil behind a vertical wall in locations with less wave action. Bulkheads are mostly adopted where wave heights are small, whereas seawalls are common where wave heights are pretty large. Revetments are often common in intermediate situations such as on bay or lake

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shorelines. Seawalls can be rigid structures or rubble-mound structures specifically designed to withstand large waves. Vertical sheet pile seawalls with concrete caps are common but require extensive marine structural design. A more common seawall design type is a rubble-mound that looks very much like a revetment with larger stones to withstand the design wave height. Thus, the two terms, seawalls and revetments, can be used interchangeably with the former typically used for the larger wave environments.

2.3.2 Seawall

Seawall (as shown in figure 2.2) is useful in protecting the specific area from erosion due to waves and storm surges. Conventionally sea walls are constructed as rubble mound seawalls, concrete seawalls or by using tetrapods. Another way is to construct seawalls by adopting the stone masonry technique or using reinforced cement concrete. Such sea walls can also be constructed using geotubes filled with sand and geotextile sand containers as the core. As sand is one of the cheapest building materials available near coastlines, it can be obtained in large quantities, is easy to process, and reusable. Seawall can also be constructed using gabions when wave heights are low, typically less than about 1.0 m. Seawalls constructed using gabions are permeable and flexible; thereby, they would be able to withstand differential settlement without loss of structural integrity. Provision of filter layer behind the seawall is essential to prevent piping of sand and subsequent destabilisation of structure. Sometimes a combination of seawall constructed using masonry or reinforced cement concrete is further protected on the seaside using gabions or concrete blocks/ tetrapods. The masonry or gabion seawalls design is to be carried out in a manner similar to the design of retaining walls to ensure stability against overturning, sliding, excessive foundation pressure (bearing capacity failure), and water uplift. Additionally, 'Wave flume studies may also have to be adopted to arrive at the satisfactory design of stone, rock and/or concrete armour units. Figure 2.3 shows the cross-section of a typical sea wall.

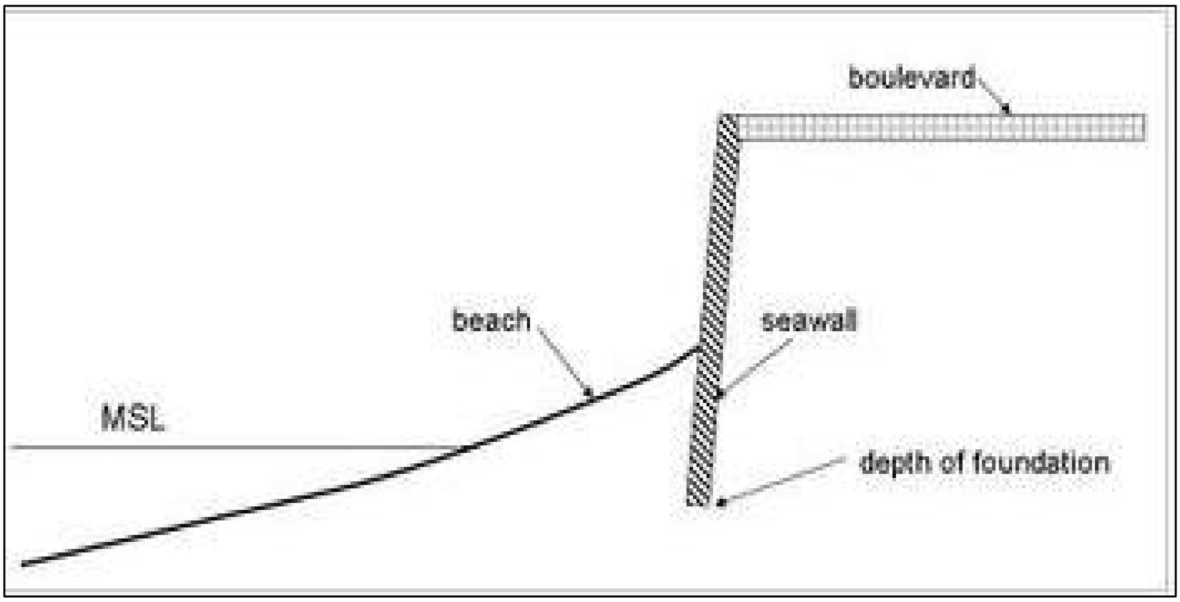

Figure 2.2 Typical Sea wall

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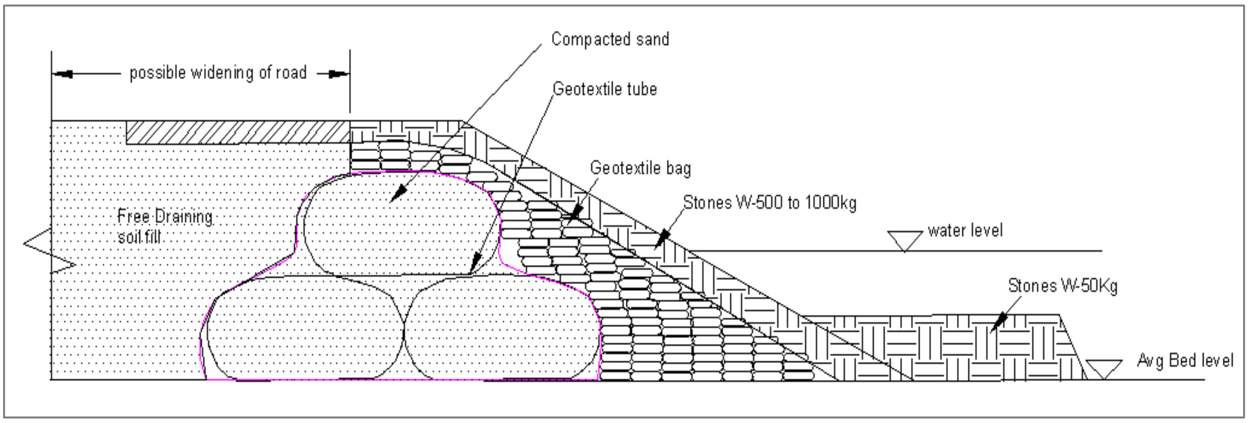

Figure 2.3 Cross section of a typical sea wall

2.3.3 Breakwater

Breakwaters are coastal structures constructed to protect an area from the effects of waves. Coastal breakwaters have the primary function of sheltering a coastal development by preventing long-shore currents from causing erosion and reducing wave energies impacting a shoreline. Breakwaters are adopted to protect a ship berthing area, train and prevent silting of river mouths' entrance, or prevent erosion of coastlines. However, adverse effects are observed on the downdrift side, and it should be avoided unless their primary purpose is to protect a specific area at the cost of adjoining areas. An offshore breakwater may be constructed to prevent beaches or coastlines from erosion by wave activity. The offshore breakwaters are submerged structures located at a certain distance offshore to dissipate wave energy before reaching the shoreline. The broken waves would not have the energy to erode the beach or coastline, and the coastline may even increase in extent due to accretion. It is an expensive option and needs regular maintenance.

2.3.4 Coastal Dikes and Groynes

Coastal Dikes (or levees) are constructed along estuaries and coastlines to protect flooding of low-lying areas. The top of flood control dikes or levees should be higher than the design high water level plus a safety freeboard. Coastal Groynes are finger-like hydraulic structures that jut perpendicularly out of coastlines, as shown in figure 2.4. Their primary engineering function is to interrupt or reduce long-shore sediment transport. The sectional requirement of a groyne structure is the same as that of a basic dike structure. Coastal groynes and dikes can be used for the following purposes:

Land reclamation: The shortage of land along certain coastlines requires land to be reclaimed from the sea. A system of dikes and groynes can be constructed to restore sediments. Reclamation dikes are cofferdam units that retain fill while protecting against wave attack during construction before placement of long-term armour protection cover. Conventionally, the cofferdam units are constructed of rock-fill material in much the same way as the rock fill core of groynes, jetties, etc. Depending on the grading of the rock fill material, geosynthetics can be used to replace conventional materials.

Artificial Islands - Dikes are used to forming the shorelines of the artificial islands in very much the same way as land reclamation techniques. Figure 2.5 shows a typical sea dike cross-section.

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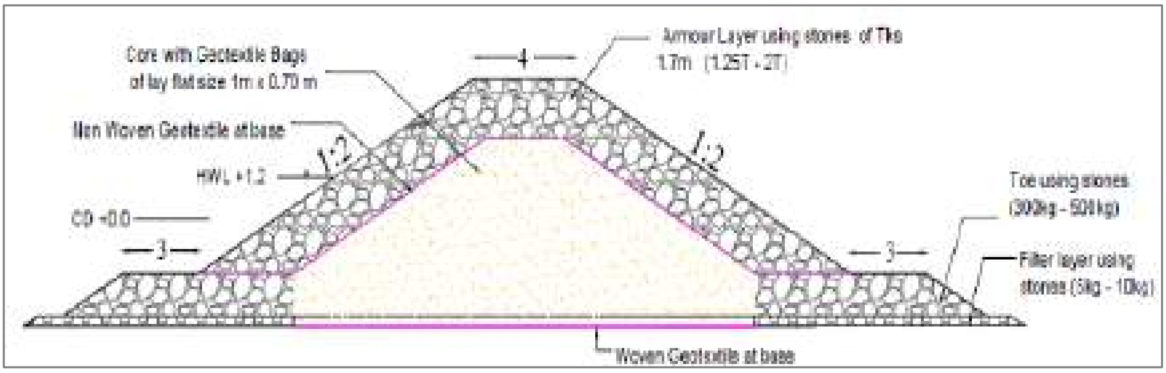

Figure 2.4 Groynes cross section with inner core layer replaced with geotextile bags and tubes

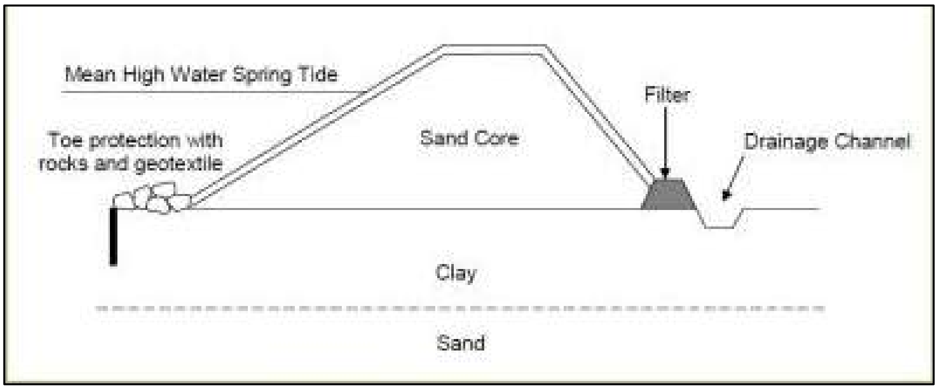

Figure 2.5. Typical sea dike cross-section

2.4 Use of Geosynthetic products for Sea Erosion Protection

Coastal and waterways protection applications comprised the earliest use of geosynthetics. Over the last few decades, numerous coastal and waterway protection projects have utilised geosynthetics. Geosynthetics can be used as components of coastal and waterway protection measures in two different ways – they can be used as filters within coastal and waterway protection structures, and they can also be used to create revetment systems (containers) to act as mass-gravity protection works. Geotextiles can also be prefabricated onshore into a large panel of fascine mattresses that can be floated out to sea. Alternatively, one may use geotubes, geotextile sand containers and geotextile bags to replace the rock-fill core of breakwater. As offshore breakwaters tend to involve construction in relatively deep waters, construction using conventional materials can be challenging. Geotextiles can be specially designed to make installation in deep waters an efficient and straightforward task.

During the construction of structural measures to control sea erosion, the problem generally faced is the non-availability of construction materials like big size boulders, sand, etc., within reasonable and cost-effective distance. This problem can be sorted to a great extent by using geosynthetic revetment systems. The most universal and widely used geotextile containers are well-known, ubiquitous sandbags that are seen worldwide for shoring up flood defences in times of natural calamity. The dominant geosynthetic material used for making revetment systems is geotextiles, which are robust and permeable materials. Three types of geotextile revetment systems differentiated by geometrical shape are available. They are geotubes, geotextile containers and geotextile bags. Geotubes are tubular containers that are filled in-situ on land or in water. Geotextile containers are large volume containers that are filled above

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water and then deposited into the submarine environment. Geotextile bags are small volume containers filled on land or above water and then pattern-placed either near water or below water level. The geotextile revetment systems have the following advantages:

1. They are resistant to chemical attacks occurring in usage, especially to alkalis and acids.
2. Geotextiles are durable when exposed to elements of nature like – sun, precipitation, etc. However, ultraviolet radiation reduces their strength in the long term. Hence, they need to be treated to enhance their ultraviolet resistance if they are exposed to the sun during their service life.
3. They are resistant to organic attacks like bacteria and fungus and are not attractive to rats or termites.

Geotextile containers behave as mass-gravity elements that can resist hydraulic forces. For these applications, the geotextile skin should have specific mechanical, hydraulic and durability requirements. A distinction must be made between those applications where the geotextile containment is required for only temporary use and those applications that require long term performance. For temporary works, the requirements of the geotextile container itself are fairly basic as it only has a short life over which it has to perform. However, for long term applications, the performance requirements of the geotextile container are severe. Regarding long-term performance, the distinction must also be made according to the type of hydraulic environment acting on the geotextile container. For example, the action of still water, or intermittent water flows, will have a different effect on the geotextile container than the action of breaking waves.

The flood control dikes can also be built with a geotube core. Instead of the sand core with toe protection of geotextile, geotextiles are used as a filter layer while constructing dikes and groynes to prevent sand beneath from being eroded. Alternatively, geotube systems may be used in place of coastal groynes. Geotextiles may be laid on the inner side of the reclamation dike to prevent the washout of sand fill through the rock fill dike. Geotextiles may also be laid over the seaside of the reclamation dike before placement of the armour protection. Alternatively, geotube Systems may be used to replace the rock fill reclamation dike.

2.4.1 Geotubes

Geotubes are large cylindrical structures made using high strength woven geotextile material. In-situ dredged material is used to fill the tubes. Geotubes may be used for a range of coastal and waterway protection applications where barrier type, mass-gravity, structures are required. The dredged material is usually pumped in a slurry form from the nearby area and consists of a mixture of sandy soil and water. The geotube, being permeable, enables the excess water to pass the geotextile skin while the fill attains a compacted, stable mass within the tube. Figure 2.6 shows the typical components of a geotube. For coastal and waterway applications, the type of fill used is sand, or a significant percentage of fill material would be sand. The reason being sand can be compacted to a good density by hydraulic means. Moreover, the sand has good internal shear strength, which gets further improved by the presence of confining geotube skin. This type of fill, once compacted, will not undergo settlement and only would change the shape of the filled-up geotube. The tube is filled by direct coupling to a hydraulic pumping system conveying dredged material. They are designed with appropriately sized openings called 'Filling Ports'. The geosynthetic tubes retain the fill-material while allowing water to permeate through the tube wall. After dewatering, typically, very little consolidation will occur in pure sands, while it may be as much as 70 per cent in the case of the tube filled with fine-grained organic material. The openings, fill ports provided in geotubes are at a spacing of about 8 to 10

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m for filling dredged material. Special high strength seaming techniques are adopted in their manufacturing process to resist pressure during pumping action. Geotubes permanently trap granular material in both dry and underground construction. Geotubes are generally about 1 m to 3 m in diameter, though they can be custom made to any size depending on their application. Geotubes ranging in diameters from 1.5 m to 5.0 m are available for coastal and waterway protection applications. The stacking of geotubes one over another can also be made to construct structures of higher heights. Geotubes may be used for a range of coastal and waterway protection applications where barrier type, mass gravity structures are required. Geotubes can be used for the construction of groynes, offshore breakwater, etc. When geotubes are used as offshore breakwater structures, the dimensions of geotubes are to be chosen in such a way that waves break over the geotubes.

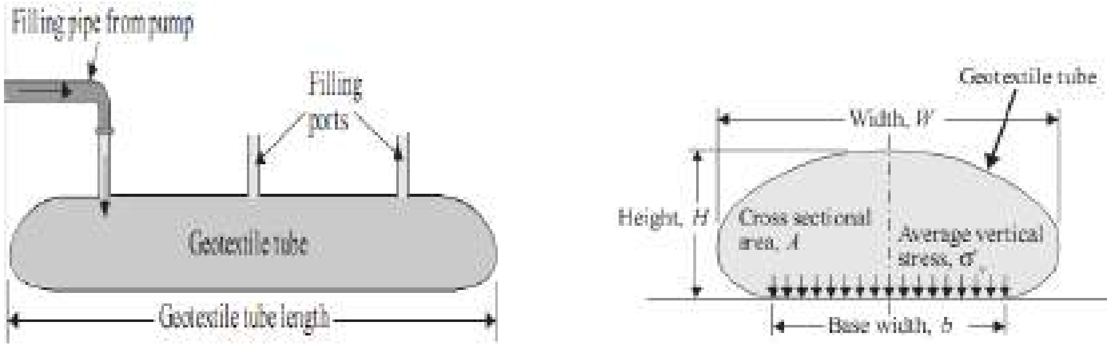

Figure 2.6. Typical Components of a geotube (Lawson CR 2008)

Geotubes are normally described in terms of either a theoretical diameter, D or a circumference, C. These two properties represent the fundamental characteristics of geotubes that are not of direct interest when it comes to engineering parameters for coastal and waterway protection applications and also in its filled condition. When the geotube has been filled with sand, it assumes an oval shape. The oval tube's width and height are of importance from the engineering performance point of view. Table 1 lists relationships between the fundamental geotube characteristics and engineering parameters. The relationships are applicable to geotubes with a maximum strain of about 15 per cent, low unconfined creep, and filled to maximum capacity with sand. It is also assumed that the foundation beneath the tube is a flat, solid surface.

Geotubes are used for revetments, whereas their contained fill provides stability. They have been used for both submerged as well as exposed revetments. For submerged revetments, the geotube is covered by local soil and is only required to provide protection when the soil cover has been eroded during periods of intermittent storm activity. Once the storm is over, the revetment is covered by soil again, either naturally or by maintenance filling.

For exposed revetments, the geotube is exposed throughout its required design life. To prevent erosion of the foundation soil in its vicinity or undermine geotube revetment, it is a standard practice to install a scour apron. The scour-apron usually consists of a geotextile filter layer that passes beneath the geotube, and it is anchored at the extremity by a smaller sized geotube, called an anchor tube.

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Table 1 Basic Engineering Parameters for geotubes filled with Sand (Lawson CR 2008)

Engineering parameter	**In terms of theoretical diameter (D)**	**In terms of tube circumference (C)**
Maximum filled height, H	H ≈ 0.6 D	H ≈ 0.19 C
Filled width, W	W ≈ 1.4 D	W ≈ 0.45 C
Base contact width, b	b ≈ 0.9 D	b ≈ 0.29 C
Cross-sectional area, A	A ≈ 0.65 D^2	A ≈ 0.07 C^2
Average vertical stress at the base, σ	$\sigma_v \approx 0.72\ \gamma$ D (γ = density of the fill)	$\sigma_v \approx 0.24\ \gamma$ C (γ = density of the fill)

Revetments using multiple-height geotubes are also often constructed. Here the geotubes are staggered horizontally to achieve the required stability. During the construction of the multiple-height revetment structure, one must exercise considerable care to ensure that the water emanating from the hydraulic filling of the upper layer of geotubes does not erode the underlying soil and undermine the lower ones. In this way, these tubes can be utilised to construct offshore breakwaters, protection dikes, containment dikes, and groynes, as shown in figure 2.7 to figure 2.10.

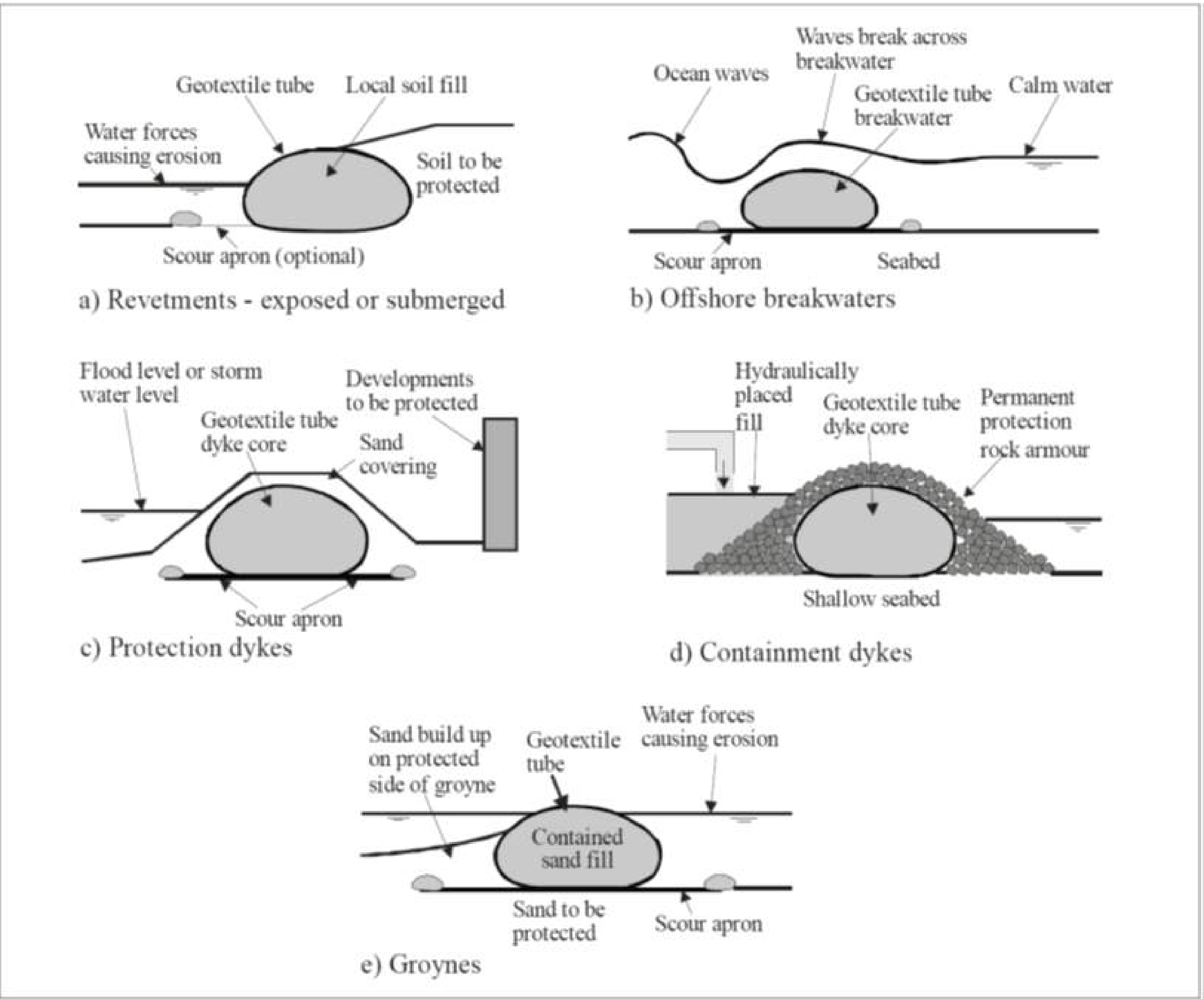

Figure 2.7. Applications of geotubes for coastal protection (Lawson CR 2008)

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Figure 2.8. Typical use of multiple geotubes for coastal embankment construction

Figure 2.9. Geotubes with gabions as armour protection layer

Figure 2.10. View of geotubes covered with armour protection layer of gabions

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2.4.2 Technical Specifications for geotubes

All the geotubes should be new and undamaged. The materials that are not new and damaged will be not be accepted. These shall be as per the manufacturer's design drawings for a particular application.

- Material shall have no environmental impact, such as pollution or leaching.
- The company providing the material must demonstrate the geotextile products that have been used and proven in the past for onshore/offshore/ nearshore applications, whichever is applicable.

The geotubes are fabricated with the standard sheets of high-strength geotextile material to form a tubular shape or as specified by the designer. The woven geotextile shall be made of multifilament polyester/ Polypropylene fibres. The percentage elongation of tubes should be greater than 13 per cent so that the system will not become rigid but less than 24 per cent so that the system will not be too flexible. The dimensions of the finished geotube shall be as per the requirements of the designer and supplier fabricating capacity. The geotube shall be fabricated in a way to ensure an efficient and durable containment system suitable for riverine or coastal applications. The geotubes are often filled hydraulically with a slurry of sand and water, although one may use many other fill materials. The length of the geotube should not be less than 10m, but not to exceed 30m, for ease in placement and handling.

The specifications of geotubes differ for aggressive, non-aggressive and very harsh coastal conditions. geotubes fabricated as geocomposites consist of double-layer geotextiles made of a polyester high-tenacity woven geotextile coupled to non-woven polypropylene geotextile. They are stitched by the needle punching method and are suitable for very harsh coastal conditions. The geotubes shall be constructed to meet the properties mentioned in Table 2 for aggressive conditions and Table 3 for non-aggressive (Typical) conditions. Figure 2.11 shows Geotubes stacked one over another.

Table 2 Properties of geotube for aggressive condition

Property	Test Method	Units	Values
Tube Circumference	Measured	m	2.3 to 25 m (Specified dimensions may vary by ±5%)
Fill Port (diameter)	Measured	cm	30 to 45
Tensile Strength (MD)	ISO 10319[21] / ASTM D 4595[1]	(kN/m)	175 (Minimum)
Tensile Strength (CMD)	ISO 10319[21] / ASTM D 4595[1]	(kN/m)	175 (Minimum)
Elongation – MD	ISO 10319[21] / ASTM D 4595[1]	%	15 (Maximum)
Trapezoidal Tear Strength	ASTM D 4533[2]	kN	2.7 x 2.7 (Minimum)
Puncture Strength	ASTM D 4833[3]	kN	1.8 (Minimum)
Seam Strength (Factory)	ASTM D 4884[4]	kN/m	105 (Minimum)
Apparent Opening Size (AOS)	ISO 12956[22] / ASTM D 4751[5]	(mm)	0.425 (Maximum) (No. 40 US Sieve)
Water Flow Rate	ASTM D 4491[6]		240 l/minute/m^2 (Minimum)
Accelerated UV Resistance	ASTM D 4355[7]		65 per cent (Minimum)
Length		m	10 to 30 m

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Table 3 Properties of geotube for non-aggressive (typical) condition

Property	Test Method	Units	Values
Tube Circumference	Measured	m	2.3 to 25 m (Specified dimensions may vary by ±5%)
Fill Port (diameter)	Measured	cm	30 to 45
Tensile Strength (MD)	ISO 10319[21] / ASTM D 4595[1]	(kN/m)	70 (Minimum)
Tensile Strength (CMD)	ISO 10319[21] / ASTM D 4595[1]	(kN/m)	95 (Minimum)
Elongation – MD	ISO 10319[21] / ASTM D 4595[1]	%	20 (Maximum)
Trapezoidal Tear Strength	ASTM D 4533[2]	kN	1.8 x 1.2 (Minimum)
Puncture Strength	ASTM D 4833[3]	kN	1.2 (Minimum)
Seam Strength (Factory)	ASTM D 4884[4]	kN/m	60 (Minimum)
Apparent Opening Size (AOS)	ISO 12956[22] / ASTM D 4751[5]	(mm)	0.425 (Maximum)
Water Flow Rate	ASTM D 4491[6]		240 litres/minute/m^2 (Minimum)
Accelerated UV Resistance	ASTM D 4355[7]		65 per cent (Minimum)
Length		m	10 to 30 m

NB: *since the maximum circumferential stress is at the side of the filled Geotextile, the seam position should be in such a way that, it does not coincide with the maximum stress position*

Figure 2.11. Geotubes stacked one over other

2.4.3 Geotextile containers

Geotextile containers, like geotubes, are mass-gravity structures used in coastal and waterway protection applications, e.g., offshore breakwaters, containment dikes, artificial reefs, slope buttressing, etc. As their name may imply, geotextile containers are large volume containers filled above water and then positioned and placed at reasonable water depth. They are filled with sand/ dredged material. These are either made from a combination of woven and non-woven geotextiles (depending on the fill characteristics) or from only high strength woven geotextiles. The volumes of these containers more commonly range from 100 m^3 to 700 m^3, although containers as large as 1000 m^3 have also been installed. To facilitate the installation of geotextile containers, an efficient and practical installation system is required. To date, this has been achieved by using split-bottom barges. This entails the filling of the geotextile container in a split bottom barge. The container is then sealed, and the barge is positioned at the correct dumping location. The split bottom of the barge is then opened, and the container is dropped into the seabed.

2.4.4 Geotextile Bags

Geotextile bags, like containers, are either fabricated from high-strength woven and non-woven polypropylene/polyester geotextiles or a combination of them. Geotextile bags are used at seashores or bunds adjacent to rivers to protect from erosion, especially during emergencies. Geotextile bags have been used as revetments, breakwaters, etc., as erosion protection measures during regular periods also (figure 2.12 to figure 2.14). Geotextile bags provide stability and prevent further erosion. Geotextile bags are filled off-site and then installed to the geometry required similarly to geotextile containers. They have to be filled to maximum volume and density with sand in an identical manner to geotubes for best performance. However, geotextile bags have two major differences from other geotextile containment techniques – they can be manufactured in a range of shapes, and they are installed in a pattern-placed arrangement that significantly improves their overall stability and performance. Geotextile bags ranging in volume from 0.05 m^3 to about 5 m^3, which are pillow-shaped or box-shaped or mattress shaped, are available, depending on the required application. The filling of geotextile bags with dry sand becomes more difficult as the volume of the bag increases, but one can efficiently fill by using a sand+water mixture (hydraulically filling the sand into a bag). The filled density and volume properties are essential for maximising stability and minimising the effects of fill liquefaction and loss of shape of the geotextile bags. To ensure that the contained fill is maintained in its dense state, the geotextile skin should have adequate tensile strength.

Figure 2.12. Shore reclamation using geotextile bags

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Figure 2.13. Geotextile bag

Figure 2.14. Geotextile bags for coastal protection

One significant advantage of geotextile bags is that these small volume units can construct hydraulic and marine structures that require adherence to designed geometrical shapes accurately. This makes them preferable to large volume units such as geotextile containers when specific slope and height tolerances are attained. Another advantage of small volume units of geotextile bags is that maintenance and remedial works can be carried out easily by replacing the failed bags. This is much simpler than carrying out remedial works on large volume containment units. The temporary walls can be constructed from sand/geotextile bags to protect structures from flooding or provide additional height to existing levee systems when floodwaters reach critical levels. However, unless emergency placement is planned well in advance under trained personnel, most sandbag barriers are not constructed according to proper practices, leading to leakage and failures. Double layer geotextile bags using woven (tape by tape) and non-woven geotextiles are used for harsh conditions. The requirements for a sustainable and well-performing geosynthetic bag system are given below:

- Adequate weight to sustain the uplift pressure due to hydraulic forces
- High abrasion resistance
- Adequate puncture strength
- High elongation to absorb the hydraulic energy
- Lower apparent opening size to retain even the finer soil particles

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- High permittivity and transmissivity
- Adequate UV Resistance
- Stable in a wider pH range

2.4.5 Specifications for Geotextile Bags

Geotextiles used to manufacture the geotextile bags should have high mechanical and hydraulic properties for enhanced durability as well as enhanced puncture, abrasion and UV resistance characteristics. Geotextile should be inert to biological degradation and resistant to naturally encountered chemicals, alkalis, and acids. The important considerations which should be addressed while specifying geotextile bags are given below:

- PP (polypropylene) and PET (polyethylene terephthalate) are the commonly used polymers for these works; however, through successive field experience, PP is found to have better performance in case of high alkaline conditions. Therefore, it is always advisable to specify on PP for Geotextile bags in alkaline conditions.
- The virgin fibres with more than 70% UV resistance shall be used as raw material for making the fabric and geotextile bags, and no recycled fibre shall be allowed for making geotextile bags.
- Staple Fiber is much better and found to have superior performance for riverbank and coastal application than continuous fibres, which degrades when contact with water.
- There are several bonding mechanisms available; however, the appropriate for river application is Needle Punching.
- Important Properties for the performance of Geotextile bags are- UV Resistance, CBR Puncture, Tensile Strength, Seam strength, Apparent Opening size and Permeability.

Geotextile used to manufacture Geotextile bags made of non-woven material may conform to the properties listed in following Table 4. The specifications for composite geotextile bags to be used in harsh conditions are specified in Table 4. Geotextile bags should fulfil the following requirements:

a. Geotextile bags shall be made from needle-punched non-woven fabric (Types 1 to 3), and composite bags shall be made of woven (tape by tape) and non-woven fabric manufactured from ultraviolet stabilised Polyester or Polypropylene. The constituent fibres shall be identified by the microscopic and confirmatory tests specified in IS 667[29], depending upon the end-user requirements and shall conform to the requirements specified in Table 4. The geotextile bags shall be inert to commonly encountered chemicals, resistant to rot and mildew, and shall have no tears or defects which adversely affect or alter their physical properties.
b. All property values except apparent opening size in these specifications represent Minimum average roll values (MARV). The average test results from any sampled bag in a lot shall meet or exceed the minimum values specified in this standard. In case of apparent opening size, the MARV shall represent the Maximum Average Roll Value.
c. Polymers used in the manufacture of geo-bags shall consist of long-chain synthetic polymers composed of 100 per cent virgin polypropylene or polyester. In any case, recycled polyester shall not be permitted because of its inherent non-uniformity and substandard quality compared to virgin polyester fibre. Polypropylene/polyester fibre generally used is virgin exclusively as it is not recycled.
d. Geotextile bags shall be dimensionally stable and retain their geometry under manufacture, transport, and installation.

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Geotextile bags shall be prefabricated using UV stabilised PET/PP thread. The geotextile bags shall have a seam with double line chain stitches along the edges on two sides with a stitch density of 20 stitches/dm + 2. The sewing shall be done at a minimum distance of 10 mm from edges by using a ring spun polyester/polypropylene thread, as the case may be, of linear density 1500-2500 Denier for bags up to 400 g/m^2 and of 2500-3500 Denier (278 – 389 Tex) for bags greater than 400 g/m^2. The distance between the two rows of stitches shall be 5 to 10 mm. Stitch lines on both sides of the bags shall continue beyond the bag's open mouth and end in a loose loop of the thread of length 25 to 50 mm. The ring-spun polyester/polypropylene thread used for stitching shall be UV stabilised. The stitching shall be uniform without any loose thread or knot. The geotextile bags having double layers of both woven, non-woven and composite materials should conform to the properties listed in Table 4 given below:

2.4.6 Gabions and Revet Mattress

Gabions are mesh-like structures filled with relatively small size stones compared to larger boulders. They are an attractive alternative for various erosion control and scour protection applications. They comprise a rectangular box made from mechanically woven double twisted hexagonal mesh filled with rock or stones. A terminal wire used to edge the wire mesh perpendicular to the double twist by mechanically wrapping the mesh wires around it at least 2.5 times. The hexagonal shape of the mesh helps in achieving uniform distribution of forces. By holding the small stones together, gabions function like large boulders but at the same time facilitates easy construction and offers a flexible structure. Thereby gabions provide a technically satisfactory and cost-effective solution. Gabions can be made from either polymeric material or double twisted steel wires having zinc+polymer coating. Gabions are generally available in a prefabricated collapsible form with the bottom and four sides held together by appropriate binding and with a flip-open top lid. The gabion becomes a large, flexible and permeable building block that can build a broad range of structures.

Because of their inherent flexibility, gabion structures can yield to earth movement and retain their total efficiency while remaining structurally sound. They are quite unlike rigid or semi-rigid structures, which may suffer complete failure when even slight changes occur in their foundation. Besides the above, gabions can be easily lifted by cranes, they are suitable for underwater construction, and several gabions can be tied together to create continuous, integral structures. The pervious nature of gabions gradually absorbs the heavy wave impact than an impervious structure. The terms specific to gabions/ revet mattress are explained below:

a) **Double Twisted (DT) Wire Mesh**: A non-ravelling mesh made by mechanically twisting continuous pairs of wires through three one-half turns (commonly called double twisted) connected to the adjacent wires to form hexagonal-shaped openings.
b) **Mesh size:** The average distance measured at right angles between twisted sides over meshes.
c) **Selvedge wire:** Terminal wire used to edge the wire mesh perpendicular to the double twist by mechanically wrapping the mesh wires around it at least 2.5 times.
d) **Edge wire:** Terminal wire used to edge the wire mesh parallel to the double twist by continuously weaving it mechanically into the wire mesh.
e) **Lacing wire:** Zinc and PVC coated metallic wire interconnects empty units to close and secure stone-filled units and for stiffeners.
f) **Stiffener:** A length of zinc and PVC coated steel wire used to support facing by connecting the front panel to the back or side panel of a gabion or across the corners of the gabion cell. Stiffener formed at the project site must be of wire having the same diameter and composition as for the lacing wire.

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Figures 2.15, 2.16 and 2.17 show the use of gabions for riverbank protection, view of a gabion box, and view of a revet mattress.

Table 4 Properties of composite geotextile bags

Properties	Reference for Test Method	Unit	Values	
			Non-Woven	Woven (Tape)
Properties of Geotextile				
Polymer Type			PP	PP
Mass per unit area	IS 14716[30]:1999/ ISO 9864[23] / ASTM D 5261[8]	gsm	≥140	≥210
Wide Width Tensile Strength	IS 13102 [31] (Part 5)/ ISO 10319[21] / ASTM D 4595[1]	kN/m	≥9	≥40
Wide Width Tensile Elongation	IS 13102 [31] (Part 5)/ ISO 10319[21] / ASTM D 4595[1]	%	≥50	≤25
Trapezoidal Tear Strength	IS 14293[32]:1995/ ISO13434[25] / ASTM D4533[2]	N	≥200	≥500
Puncture Resistance	IS 16348[33]:2015/ ISO 12236[26] / ASTM D 6241[9]	N	≥300	≥500
Apparent Opening Size	IS 14294[34]:1995/ ISO 12956[22] / ASTM D 4751[5]	mm	≤0.212	≤0.42
UV Resistance	ASTM D4355[8]	%/hrs	70/500	70/500
Properties of Geotextile Bag				
Seam Type			Double Seam	
Preferably flat dimensions			2.00m x 1.50m	

Table 5 Requirement of needle punched nonwoven geotextile bags

Sl. No.	Properties	Test Method	Unit	Requirements		
				Type 1	Type 2	Type 3
Mechanical Properties						
1	Wide Width Tensile Strength, *Min* (MD/CMD)	IS 13102 [31] (Part 5)/ ISO 10319[21] / ASTM D 4595[1]	kN/m	15	20	24
2	Elongation, *Min* (MD/CMD)	IS 13102 [31] (Part 5)/ ISO 10319[21] / ASTM D 4595[1]	%	50	50	50
3	Seam Strength, *Min*	IS 15060[35] /ISO 10321[24]/ ASTM D 4884[22]	%	80		
4	Trapezoidal tear strength, *Min MD/CD*	IS 14293[32]:1995/ISO 13434[25]/ ASTM D4533[5]	N	340	450	600

5	CBR Puncture Resistance, *in*	IS 16348[33]:2015/ ISO 12236[26]/ ASTM D 6241[9]	N	3000	4000	4700
6	CBR Burst Elongation, *Min*	IS 16348[33]:2015/ ISO 12236[26]/ ASTM D 6241[9]	%	70		
Hydraulic Properties						
7	Permittivity, *Min*	IS 14324[36]:1995 / ISO 11058[27] / ASTM D 4491[6]	l/s	1.25	1.10	1.00
8	Water Permeability at 100 mm Water Head, *Min*	IS 14324[36]:1995 / ISO 11058[27] / ASTM D 4491[6]	m/s	60	40	30
9	Apparent Opening Size (AOS), *Max*	IS 14294[34]:1995/ ISO 12956[22] / ASTM D 4751[5]	µm	75	75	75
Physical Properties						
10	Thickness under 2kPa, *Min*	IS 13162[37] (Part 3)/ ISO 9863-1[28] / ASTM D 5199[10]	Mm	3.0	3.0	4.0
11	Polymer Type; Polyester (PET) or Polypropylene (PP)	IS 667[29]		PET/ PP	PET/ PP	PET/P P
12	Mass per unit area, *Min*	IS 14716[30]:1999/ ISO 9864[23] / ASTM D 5261[8]	g/m^2	300	400	600
13	Length of geotextile bag, a) Small Size b) Large Size	IS 1954[38]	M	a) 1.00 b) 2.00		
14	Width of geotextile bag, a) Small Size b) Large Size	IS 1954[38]	M	a) 0.70 b) 1.50		
15	Weight of filled geotextile bag, *Min* a) Small Size Bag b) Large Size Bag	Electronic Weighing Balance	kg	a) 126 b) 1350		
16	Durability	Shall be durable for a minimum of 10 years in natural soil with 4≤pH≤9 and soil temperature between -20°C to 25°C.				
17	UV Resistance after 500 h exposure, *Min*	ASTM D 4355[7]	%	70	70	70

Figure 2.15. Use of gabions for river bank protection

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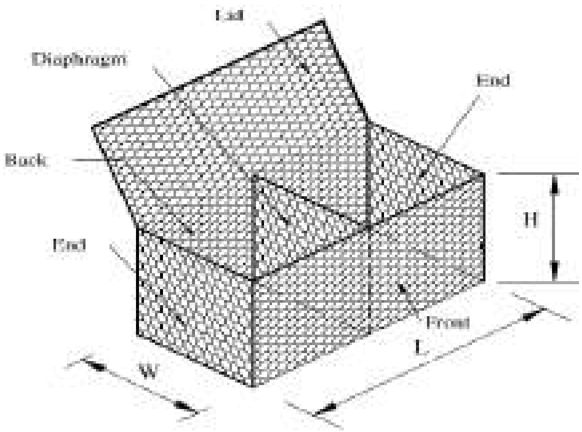

Figure 2.16. View of a gabion box

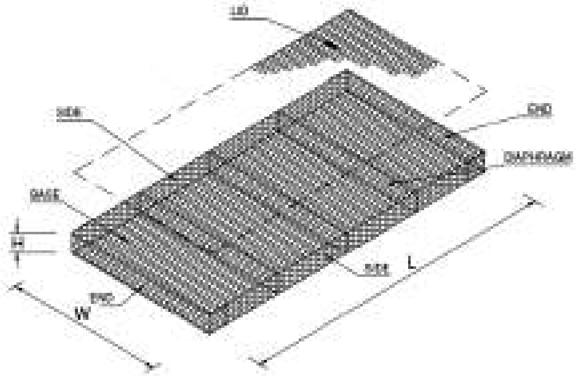

Figure 2.17. View of a revet mattress

IS 16014[33] provides specifications for zinc+polymer coated steel wire gabions. Compared to steel wire gabions, polymeric gabions have advantages like superior corrosion resistance, ability to withstand the acidic and alkaline environment, excellent durability, excellent flexibility to take the shape of ground contour, etc. However, due to their very high flexibility, these gabions may not be as much amenable to the construction of retaining structures as compared to steel wire rope gabions. Revet mattresses are essentially smaller sized gabions in the form of a wire mesh container uniformly partitioned into internal cells with relatively smaller height in relation to other dimensions, having smaller mesh openings than the mesh used for gabions. It is a specialised form of gabion and has a relatively larger plan size and shallower depth. Their primary function is to protect the bed of the watercourse from excessive scour.

2.4.7 Geotextiles as Filters

Geotextiles are used as filter layers for the construction of coastal structures. Below the revetments (either stone/rock or concrete armour units), filters are invariably required to prevent soil washout. Traditional granular filters usually consist of several layers of stone aggregates. If the water forces are strong enough and the soil to be protected is fine-grained, then up to four layers of granular materials may be required to satisfy the hydraulic design requirements. Hence, this kind of relatively complex structure can be expensive and challenging to construct.

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Furthermore, granular/ aggregate filters are difficult to place on steep slopes, cannot always be installed in tidal zones, and the laying process demands reliable and expert supervision. Geotextiles can be used as substitutes for one or more granular underlayer materials below revetments. Geotextiles offer many advantages over granular filter materials:

- They enable design flexibility with regard to the choice of the size of the granular material in the layer immediately adjacent to the geotextile filter.
- They are easier to install to specific geometrical configurations than granular materials – in many cases below water level.
- In general, in-situ quality control test requirements for geotextiles are nominal.

Where geotextiles are used as filters for coastal and waterway protection, their primary function is to prevent soil erosion through the protection structure and thus prevent instability. In the case of geotextile filters, the hydraulic characteristics like apparent opening size and permittivity are most important. The selection of filter fabric with the correct opening size depends on the percentage of finer material available in bed material. In order to fulfil its function, the geotextile material has to be robust enough to resist mechanical stresses applied to it during installation. Secondly, the geotextile material must have required hydraulic properties to perform as a filter material. Thirdly, the geotextile must have adequate durability to maintain its mechanical and hydraulic properties throughout the design life of the revetment. The criteria for the selection of filter fabric can be based on IRC SP: 59[19]. As the weight of the stones/ drop height increases, a thicker geotextile having a greater mass per unit area would be required. Another important property would be trapezoidal tear strength. Normally, geotextiles having trapezoidal tear strength varying from 200 to 600 N (ASTM D 4533[2]) are used in coastal works. When determining the appropriate hydraulic properties for the geotextile revetment filter, consideration must be given to the critical hydraulic regime that will act on the revetment structure over its design life. Table 6 list the geotextile filter hydraulic properties requirements according to the type of hydraulic regime. When several different hydraulic regimes occur at the same location, then the most critical hydraulic regime (1 being the least critical and 3 being the most critical) should be chosen for the design. While installing a geotextile filter, it is to be first laid out on the soil surface prior to placing stones and rocks. For good long term performance, the geotextile filter should be covered with an adequate thickness of granular material to ensure that it remains protected from the effects of long term exposure to ultra-violet (UV) rays. The minimum thickness of stone coverage above the geotextile filter to protect against UV radiation should be at least two times the maximum stone size in the rock armour layer. During installation, it may be inevitable that the geotextile filter would be exposed to UV rays, and this condition may extend, depending upon the pace of construction. To cover such eventualities, the UV stability of the requirement of geotextile to be used should meet the specification requirements as per IRC SP:59[19]. The geotextile filter coverage beneath the revetment armour layer should extend beyond the zone of erosion. This would ensure that the revetment structure will remain stable throughout the life of the structure.

Table 6 Geotextile Hydraulic Property Requirements under Different Regimes

1	Water current flows parallel to revetment face	
	Non-dispersive soil	$O_{95} \leq 0.35$ mm
	Dispersive soil	$d_{15} \leq O_{95} \leq d_{85}$
2	Gradual reversing water flows	$d_{15} \leq O_{95} \leq d_{85}$
3	Impacting wave activity	$d_{15} \leq O_{95} \leq d_{50}$

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d_{15}, d_{50}, and d_{85} are percentile particle size fractions to be protected
O_{95} is apparent opening size (AOS) of the geotextile filter (ASTM D 4751[5])

2.4.8 Geotextile filters for breakwaters

For rubble mound and caisson wall breakwaters, geotextile filters are placed on top of the sea bed prior to the construction of the breakwater. In this location, the primary role of the geotextile filter is to prevent erosion of sea bed and the undermining of the breakwater. The geotextile filter is usually prefabricated onsite into a fascine mattress structure to facilitate installation on the seabed. This technique involves the fabrication of geotextile filters into large sheets on land and attaching an interconnecting grid of fascines, bamboo or timber. The resulting mattress is then pulled into the water and floated into place, and sunk on the sea bed. This technique has proved to be an efficient and cost-effective means of installing geotextile filters on the sea bed. The tensile stresses imposed on the geotextile filter during the fascine mattress installation procedure are relatively high. Consequently, woven geotextiles with wide-width tensile strengths ranging from 80 kN/m to 200 kN/m are normally used for this type of application.

Offshore breakwaters also may be constructed to protect beaches or coastlines from erosion by wave activity. In such cases, the breakwaters would be submerged structures that force the waves to break when passing thus, expending much of their wave energy. The broken waves would not have the energy to erode the beach or coastline, and the coastline may even extend outwards into the sea as a result.

2.4.9 Geotextile filters for containment dikes

To reclaim land from the sea, it is common to first construct a containment dike around the extremity of the reclamation area. Soil or sand fill is then dry dumped or hydraulically pumped into the containment area to form dry land. The function of the containment dike is to prevent loss of the placed soil or sand fill into the surrounding water. The nature of the containment dike is slightly different depending on whether the reclamation occurs in relatively deep water or in shallow water. Where land reclamation occurs in relatively deep water, the size of the containment dike is fairly large and may require two or more stages to complete the structure. Commonly, the dike consists of a rubble mound of dumped rock with a geotextile filter placed across the base of the dike. The role of the geotextile filter is to prevent the loss of reclamation fill through the rubble mound dike and the erosion of the sea bed beneath the rubble mound. The geotextile filter across the base of the dike can also prevent the loss of the rubble mound material into the sea bed if the foundation is soft. For permanent protection, a rock armour layer may be placed on the outside of the rubble mound depending on the water forces acting on the structure.

Where land reclamation occurs in relatively shallow water, the containment dike is normally constructed in a single stage. Commonly, the bund consists of a rubble mound with a geotextile filter placed across the base of the dike. Again, for permanent protection, a rock armour layer may be placed on the outside of the rubble mound depending on the water forces acting on the structure.

It is not uncommon for the base geotextile filter beneath the containment dike to have different properties on different faces. Normally, the base geotextile filter is installed in a manner similar to the breakwater structure, which may require a fascine mattress approach to installation. This imparts relatively high tensile stresses on the geotextile filter during installation. Therefore,

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woven geotextile filters with wide-width tensile strengths between 80 kN/m and 200 kN/m are usually used for this purpose.

2.4.10 Installation of geotube

Geotubes must be handled with care. One shall not handle them with hooks, tongs or any other sharp instruments or shall not drag them along the ground. In order to load and unload the material, appropriate handling equipment and techniques recommended shall be used (figure 2.18). Proper care shall be taken during the unloading of material at the site upon delivery; the rolls will be lifted with the help of slings attached to the hook of the crane, as shown in the photo below.

Figure 2.18. Material unloading using crane at site

The surface upon which the geotextile may rest shall be levelled adequately and prepared to a relatively smooth condition and shall be free of obstruction that could snag and tear the fabric like ruts, erosion rills, obstructions, depressions or debris, or protrusions. Geotubes shall be stored in areas where water cannot accumulate and protected from conditions that will affect the properties or performance of the geotextile (figure 2.19).

Figure 2.19. Storage of geotubes

a. Preparation of geotubes on land

Each geotube before the start of the work shall be unrolled and laid out flat on the land. Thorough checking of the tubes will be carried out, and once verified and confirmed by the ground personnel, the tubes shall be folded in concertina style. The tubes shall now be considered ready for transportation to the boat or dumb flat pontoon or appropriate vessel.

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These will be kept in order/sequence of planned installation in the vessel (figures 2.20 to 2.22).

Figure 2.20. Geotube laid flat on levelled surface

Figure 2.21. Rolling out of geotubes

Figure 2.22. Lifting of tubes in concertina style

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b. Placement of the geotubes on the sea bed and temporary fixation

The folded geotubes shall then be carried to the location, which can be identified as per the GFC's and marked appropriately on the sea bed with the help of divers. The geotubes shall be carried to the location with boats or vessels (dumb flat pontoon with a frame fitted or crane arrangement). According to the groyne location site, individual units to be worked the area of tubes shall be marked off with buoys around the perimeter as temporary reference points or as per the requirement. The concrete mooring blocks (temporary fixing arrangement for geotube) shall be placed on the sea bed around the footprint of the section on which work is planned to be carried out. A concrete mooring block having dimensions of approximate radius 400mm and height 1000mm weighing approximately 800-1000kgs shall be cast as per requirements. The lifting eyes shall be cast into the block using 16mm diameter reinforcement steel bars. The lifting eye at the top shall stand 150mm out of the block. Similarly, side-eyes shall also be cast at 300mm from the bottom of the block. Please refer to the figure below. The blocks shall be used to fix and hold the tubes onto the sea bed prior to the start of the pumping activity. The blocks help in deploying and hold each geotube in position. Figure 2.23 shows the lifting of tubes in a concertina style.

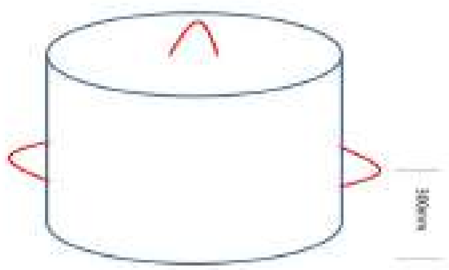

Figure 2.23. Lifting of tubes in concertina style

c. Preparation of Sand slurry and transportation of slurry

The slurry preparation pit will be prepared near the shoreline to keep the pumping distance minimum. The water pumps located near the shoreline pump the seawater into the slurry pit for slurry preparation. The slurry pit will have a screening arrangement to separate the impurities and have clean sand for the slurry. The dry sand shall be transported to the location and, with the help of excavators, shall be dropped inside the slurry pit. The slurry pit will have water-filled $1/3^{rd}$ of the height before dropping the sand in the pit. The water shall be pumped into the pit as soon as the first load of the sand is dropped inside. The water ratio to sand will be kept from 40:60 to 30:70 by volume, and the same shall be maintained throughout the project duration for all types of slurry preparations. The slurry pit will hold suspended a series of pumps for slurry transportation. Since the slurry is to be transported to a 500m distance, one should select the capacity of the pumps appropriately to achieve the desired output.

d. Filling of geotubes

The slurry will be transported via a hosepipe of suitable diameters to maintain the optimum pressure at a horizontal lead of approximately 500m. The pressure at the outlet of the hose pipe shall not be high in order to damage the seams of the geotubes. Each geotube shall be prefabricated with a series of filling ports along its length. These ports allow a dredge line/hose pipe to be inserted into the geotubes, through which the slurry of sand and water would be pumped, as shown in figure 2.24. In general, it is possible to completely fill the geotube using

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only a single filling port. It is essential that all ports are open during the filling process to act as outlet ports. This shall ensure that the pressure inside the geotube does not cause damage to the fabric of the geotextile, and there is even distribution of sand along the entire length of the geotube. Care shall be taken to ensure that the pumping pressure does not affect the seams of the geotubes.

Figure 2.24. Filling of tubes

After filling activities, monitoring of crest level shall be carried out to ensure that appropriate height of geotube is reached after filling. The filler ports (figure 2.25) shall then be closed thereby, ensuring the complete closure of geotubes.

Figure 2.25. Filler port

Figure 2.26. Installed groyne using geotube

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The engineer-in-charge shall be notified before the preparatory activities and prior to starting the filling of geotubes. All the tubes will be filled evenly as possible at an approximate level provided in the GFC's. Once the tube is filled, the engineer-in-charge shall be notified and provided with the details of the filled geotube. Figure 2.26 shows the installed groyne using a geotube.

2.4.11 Summary of Installation Guidelines for geotubes

- Geotubes used in coastal applications are most often filled hydraulically with a slurry of sand and water.
- A scour apron may be necessary to provide the geotube to prevent the undermining effects of scouring. Scour apron is made of high strength woven geotextile designed to protect the foundation of the main geotube from scouring. Scour aprons are typically anchored by small tubes at the water's edge called scour tubes.
- The foundation for the placement of the geotube and it's scour aprons shall be smooth and free of protrusions that could damage the geotextile.
- Suitable material for filling the tubes should not contain more than 15% fines (percent by weight by passing the No 200 sieve) to minimise subsidence of the tubes after filling. One shall conduct the gradation testing of hydraulic fill materials, and a minimum of one gradation test shall be performed for every 1000 linear feet (300 m) of the fill tube. Discharge pressure at geotube fill port shall not exceed 35 kPa.
- The geotubes require an alignment within + 600mm of the baseline. The filled tubes shall have an effective height of ± 0.5 feet (150 mm) of the specified elevation.
- The main geotube and the scour apron shall be deployed along the alignment and secured in place as necessary to assure proper alignment after filling. No portion of the tube shall be filled until the entire tube segment has been fully anchored to the foundation along the correct alignment.
- The ends of tubes can be overlapped or butted together. An overlap of 1.5 m is recommended to ensure proper terminal connection. Beneath the geotube, the ends of each geotextile scour apron also shall be overlapped a minimum of 1.5 m. The effective height of the tube structure at the overlap is typically 80 per cent of the specified height.
- The underwater alignment of the geotube can be achieved by placing temporary guides on either side of the tubes. Before filling the tube with the sand slurry mix, the alignment correction should be carried out by filling the tube with water. If there are scour tubes along with the main tube, the scour tubes should be filled prior to the main tube.
- After completing the deployment and anchorage of the geotubes, the filling with sand and water mixture should be started. The mixture shall contain 5 to 15 per cent of sand. The inlet port pressure should be limited to 35 kPa.
- After filling the tube, the port sleeves shall be closed and attached to the main tube. The closing of the fill ports can be done by sewing or knotting by rope or nylon cables.

2.4.12 Geotextile Bags

The geotextile bags will be procured and stored in the storage yard. The bags are filled manually with dry sand. The bags are open at the top ends from where dry sand is filled. The top end of the bag is stitched, and the open end is closed once the desired dimension of the bag is achieved (figure 2.27 to figure 2.29).

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Figure 2.27. Bag storage at site

Figure 2.28. Stitching of bags at the site

Figure 2.29. Storing of geotextile bags post filling

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2.4.13 Gabions - Construction Guidelines

Gabions are delivered at the worksite in bundles along with the lacing wire. Bundles must be unloaded using any available machine and stored close to the site where they are to be installed. While unloading, care shall be taken that gabion bundles are not damaged. Maximum 4 bundles shall be stacked over each other. Gabion bundles shall be stacked on a levelled ground with wooden rafters/sleepers between them for easy handling and preventing any damage to the mesh and other components like the selvedge wire. Storage can be outdoors as long as the bundles are not laid over abrasive or chemically aggressive surfaces (corrugated, asphalt, oils, grease, chemicals, etc.). The following are the steps in the basic procedure for the erection and installation of structures. At the worksite, bundles should be opened and gabions assembled according to the following instructions:

1) Excavate the formation to the required level, as shown in the drawings. If a good bearing stratum is not observed after the excavation up to mentioned levels, then the same should be brought to the notice of the engineer-in-charge for further advice. The surface of formation should be free from any harmful material and unwanted foreign objects. Loose pockets, if any, should be excavated and filled with suitable granular or backfill material.
2) The place should be cleared of standing water to ensure the proper placement of gabions. In case dewatering is not possible, then gabions shall be installed underwater.
3) Compact the formation using a vibro-roller of 8 to 10 tonnes capacity. The density of compacted formation should be greater than or equal to 95% of the modified Proctor value.
4) The survey team should mark the outer alignment of the gabion facia as per the requirements in the drawings.
5) The gabions are unfolded and laid over an even, level and hard surface (compact ground). One or two persons may be deployed to straighten mesh panel activity, depending on the size of gabions. The gabions should be properly assembled and wired together so that all four corners match and form an exact rectangular shape and size as designed (Figure 2.30).

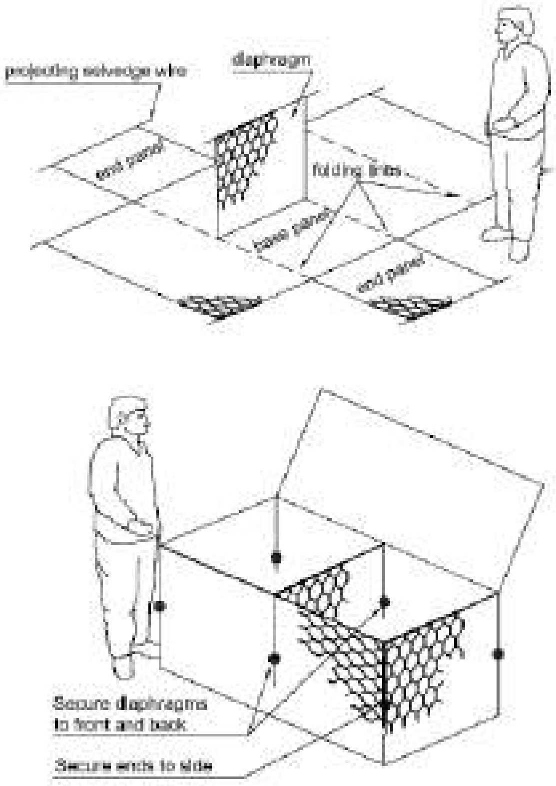

Figure 2.29. Gabion installation steps

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6) The side panels are opened up to form a box by tying the upper corner of each pair of side panels using the thick selvedge wires.
7) The edges are laced together, starting from the top corner, in a continuous operation using the alternate single and double loop at the spacing of one mesh length, i.e., 100mm to 125mm. The ends of the lacing wire are secured at each corner and turned into the gabions. One should not use individual ties of lacing wire. Alternatively, steel rings mentioned IS16014[39] could be used, which are attached using the mechanical tool (Figure 2.31 a & b).
8) Similarly, all the other sides are tied up, and the box structure made, with the top end open to fill the boxes.

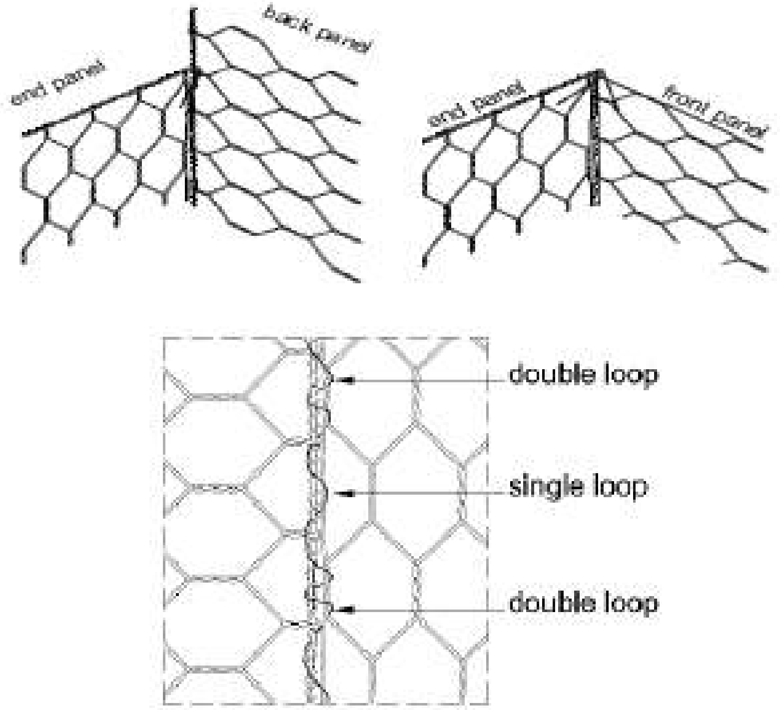

Fig 2.31(a): Assembling details with lacing wire

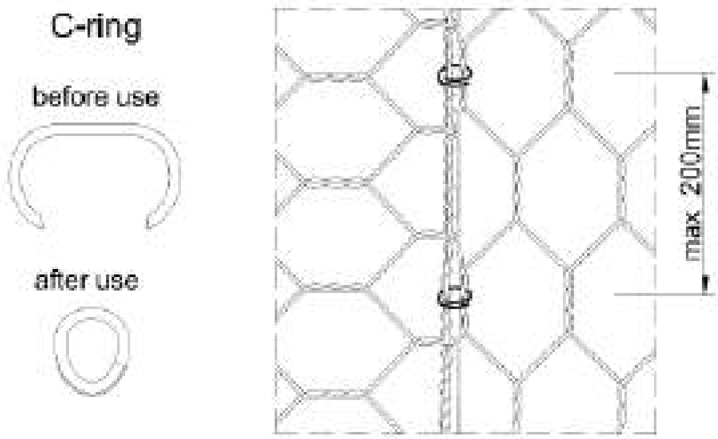

Fig 2.31(b): Assembling details with C-rings

Figure 2.30 (a & b) Assembling details with lacing wire and C-rings

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9) The filling is then carried out with stones of dimensions larger than the mesh opening, making sure that there are minimum voids. For a better finish, the stone fill to the front face is selected and placed carefully to give the best appearance. When using quarried/crushed rock, the rock shall be selected in such a way that at least one face is flat-shaped. This will not be applicable when using rounded stones from rivers. Front loader or back loaders or excavator can be used to fill gabions, maintaining porosity. For a better alignment of the face, it is advisable to include internal tie wires while filling the gabions. These horizontal tie wires prevent gabion deformation during the filling stage.
10) In the case of one-meter-high boxes, gabions should be filled to one-third height and the tie wires fixed (Figure 2.32). The exact sequence is repeated at two-thirds height. For 0.5 m high, the gabion requires only one row of tie wires at half the height of the gabion.

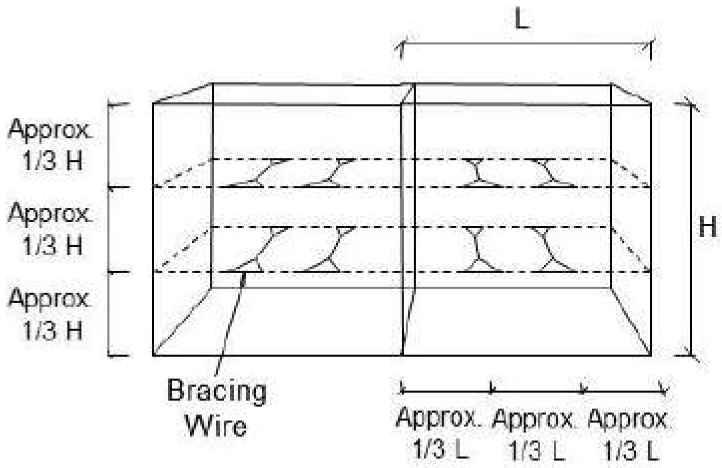

Figure 2.31. Bracing wire details

11) The filled layer should never be more than 300 mm higher than any adjacent cell. To avoid such circumstances, the filling pattern as shown in below Figure 2.33 be followed. The gabion is over-filled by approximately 25 mm to 50 mm, to admit for settlement of the infill (due to self-weight).

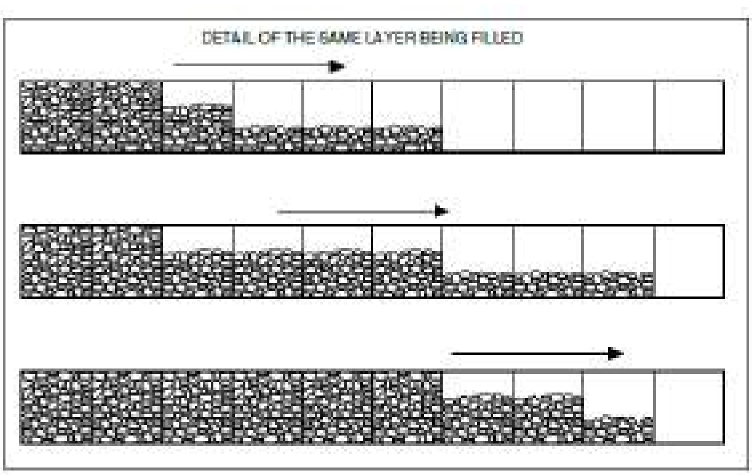

Figure 2.32. Filling pattern of gabion boxes

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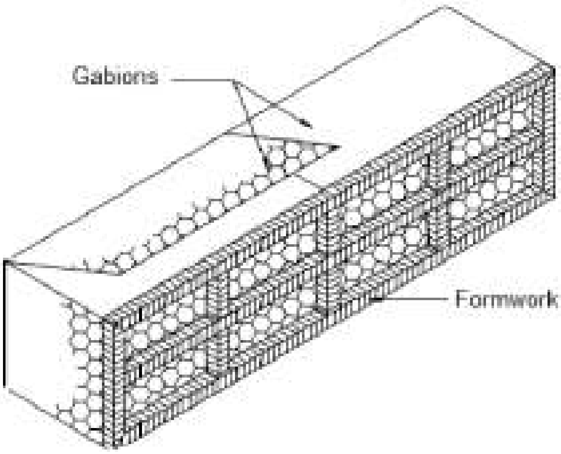
12) MS pipe/frame formwork shall be provided at the gabion facia to achieve an excellent aesthetic appearance and keep the bulges within the specified tolerances. The formwork enables to achieve uniformity in the gabion box dimensions during the filling and placement of stones. As far as possible, a fair face of large flat stone should be placed at the exposed faces only.
13) The lids are folded back and laced first to the front panels, then to the side ones and to any existing diaphragm to close the Gabions (figure 2.34). The gabions in the upper layer shall be connected to the top of the gabions in the lower layer along the front and back edges of the contact surface using the same connecting procedure. Facia of adjoining units must be securely joined together along the vertical facing and top edges of their contact surfaces. Figure 2.35 shows the lid closing procedure at the site.



a) Temporary Formwork

b) Use of pipes

Figure 2.33. Use of steel frame or pipes to avoid gabion bulging during filling operation

Figure 2.34. Lid closing procedure at the site

14) Other boxes are also placed and packed in position following the same procedure to complete the wall.

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15) The gabions should be covered on the inner face with a non-woven geotextile before placing and compacting the backfill.
16) The soil for structural fill shall be placed and compacted to 98 per cent of modified Proctor density in the area behind the gabion units.
17) Compact the structural fill using Vibro-roller of 8 to 10 tonnes capacity. Compacted density shall not be less than 95% modified Proctor density for the formation. The compaction within 1.5m of the rear of the face element shall be carried out using vibratory plate compactors or walk-behind rollers having 1-tonne capacity. Figure 2.36 shows a typical photo of a non-woven geotextile below the revet-mattress during construction.

Figure 2.35. Typical photo of non-woven geotextile below revet mattress during construction

2.4.14 Summary of Installation Guidelines for Gabions

1. The gabion is opened and unfolded on a hard surface, and the creases should not form as the box is pressed out.
2. The front and rear sides, ends and diaphragm are lifted into position to form a box shape.
3. The edges should be laced together starting from the top corner in a continuous operation using alternate single and double twists at the spacing between 100 mm and 150 mm. Individual ties of lacing wire must not be used. Alternatively, fasteners can be used for tying the edges together. The manual or pneumatic tool should be used for fastening.
4. The initial layer of gabions should be placed on a flat surface.
5. Several gabions are placed in position and secured together as described in 3 above.
6. The end gabion is partly filled with suitable stone to form an end anchor, and bracing wires are fixed at 300mm spacing to prevent the front side from bulging. The gabions are tensioned by applying a load to the end remote from the anchor gabion, ensuring the load distributed over the whole area of the last gabion.
7. The facing stone should be having a regular shape and dressed with the help of a skilled mason to get a good appearance and alignment.
8. The batter has to be maintained as per the design.
9. One-meter-high gabions should be filled to one-third height braced, then filled to two-third height and braced again. The half-meter height gabion requires only one row of bracing at 250 mm height. The gabion is overfilled by approximately 50 mm to 75 mm to allow for settlement of the infill (due to self-weight)

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10. The mesh lid is folded down, stretched into position, and the lid laced to the front sides and the top of the diaphragms.
11. The remaining row of gabions may then be filled sequentially.
12. It is essential that each gabion is properly laced and fixed to adjacent gabions above, below and on each side.

Procedure for Revet Mattress laid in dry conditions:

1. The mattress is opened and unfolded on a levelled surface, and the creases are not required to form while the box is pressed out.
2. The front and rear sides, ends and diaphragm are lifted into position to form a box shape.
3. Mattresses are laid directly onto the ground to be protected, so the slope must be stable and must not be so steep as to cause the revetment to slide. These units are typically laid down the slope of the bank at right angles to the current. However, if the banks and the bed are to be covered completely, the units on the bed itself should be laid in the flow direction.
4. The edges are laced together starting from the top corner in a continuous operation using alternate single and double twists at the spacing between 100mm and 150mm. Individual ties of lacing wire must not be used. Alternatively, fasteners can be used for tying the edges together. The manual or pneumatic tool should be used for fastening.
5. A number of mattresses are placed in position and secured together as described in 3 and 4 above.
6. The mattresses are filled with stones of a suitable size – about 1.5 to 2.5 times the dimensions of the mesh.
7. The corners should be adequately filled, and compaction is not necessary.
8. The mattresses are then covered with lids folded down, stretched into position, and laced with the usual lacing operation.
9. It is essential that all mattresses are correctly laced and fixed to adjacent mattresses along all corners with the identical lacing operation.

2.4.15 Issues to be Considered While Installing Gabions

1) In port works, prefilled gabions (figure 2.37) may have to be used. The contractor shall use adequate machinery for such installations and obtain Engineer's prior approval for his installation methodology.
2) The contractor shall be required to provide a complete design with working drawings in required numbers. This shall be supported by a certificate from the supplier of gabions and revet mattresses that he has proven experience in supplying and designing gabions for retaining walls / scour protection/erosion control for at least three major projects in India and providing technical back up like designing using in-house facilities and technical assistance at the site. The supplier shall produce authentic documentary evidence to prove his experience in supplying, designing and providing technical assistance at the site for at least three Major Projects in India.
3) The contractor shall take prior approval from the Engineer-in-charge before placing the order on the Manufacturer/Suppliers of Wire mesh Gabion materials. The engineer-in-charge will give the approval in writing after satisfying himself about the conditions described above.
4) The contractor shall provide a 'Performance Guarantee' for all the works, including gabion and revet mattresses under regular maintenance.
Figure 2.37 shows a Prefilled Gabions Placed in Water

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Figure 2.36. Prefilled gabions placed in water

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Chapter 3 - Geosynthetics for Ground Improvement

3.1 Need for ground improvement measures in bulk handling yards, roads.

Deep deposits of soft clay are found all along with the coastal and delta areas of the country. Major ports of India are located where soft soil deposits abound. Structures such as bulk handling yards (container yards, ore stacks, etc.), buildings and roads constructed over soft soils deposits in port areas experience failures due to low bearing resistance or large total and differential settlements. Specifically, the construction of embankments over soft subsoil often causes difficulties in design and construction because of the low shear strength and high compressibility of such soils. Embankments on soft soil may fail due to:

- Failure of soft subsoil in shear (failure in bearing capacity)
- Sliding of embankment fill and underlying soft subsoil (failure along a slip circle)
- Excessive settlements and also lateral displacements

With increasing developmental activities near ports in our country, there has been a tremendous rise in the demand for land. As a result, construction of all types of structures, particularly industrial structures and transport corridors, are being taken up even on soft soil deposits. At many places on the eastern coast, the sub-soil is so soft that it cannot support even 1 m height of the embankment. If the thickness of poor clay layer is less and is at shallow depth between 1 to 3 m, then it can be removed and replaced by soil having good shear strength. But where the depth of sensitive clays exceeds 5 m, it becomes essential to adopt ground improvement techniques to achieve the required bearing capacity and reduce the total and differential settlements.

3.2 Ground improvement options suitable in port areas

The available solutions to deal with compressible soils like marine clay deposits can be divided into four different categories as follows:

- Bypassing the compressible soil with providing alternate alignment
- Accepting the time constraints and adopting pre-compression by surcharge load alone
- Constructing bridge with deep structural foundations
- Accelerating the compressibility of the in-situ soil

The first two solutions may not be feasible always and may lead to a very steep increase in project cost. The use of a deep foundation is an effective, although expensive, means of bypassing compressible soils. Decreasing the post-construction compressibility of in-situ soils has also been achieved through several methods such as stone columns, dynamic compaction, deep soil mixing, grouting, etc. By these methods of stabilization shear strength and bearing capacity are also improved. However, these methods are costly to implement and may not be effective at all locations. As a result, ground improvement using vertical drains, basal reinforcement and using either cased or non-cased stone columns have been widely accepted for ground improvement techniques for soft clay deposits in port areas.

3.3 Ground improvement using vertical drains

Vertical drains have been used for more than half a century to promote the rapid primary consolidation of thick, soft clay deposits like marine clays, where preloading alone will be insufficient. Sand drains were the earliest type of vertical drains used for the consolidation of the soft clay layer. Installation of sand drains is usually done by drilling boreholes in soft clay and backfilling the borehole using sand of specified gradation. The major problem, in this case, would be the formation of cavities due to bulking of sand. Polymeric vertical drains (PVD),

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also known as 'Band drains', have now virtually replaced sand drains/ sand wick techniques for ground improvement because of easy and speedy installation, less disturbance to the surrounding soil and lower cost.

3.3.1 Band drains (PVD)

Band drains consist of a plastic/polymeric core formed to create channels or paths which are surrounded by a thin geotextile filter jacket as shown in Fig 3.1. Typically, the size of band drains is 10 cm in width and 3 to 9 mm in thickness. The primary functions of jacket and core as summarized by Ladd C.C (1991) are as follows:

Jacket separates the flow channels of the core from the fine-grained soils in which the drain is installed. It acts as a filter and separator thereby allow easy passage of water into the core and prevents migration of fines of soil respectively. Also prevents closure of flow channels of the core under lateral soil pressure.

Core provides internal flow paths along the length direction of the drain and maintains the shape and configuration of the drain. The strength of the core material offers resistance to buckling and longitudinal stretching (tensile stress during installation) of the drain.

The primary use of band drains accelerates consolidation and decreases embankments' settlement time significantly over soft soils. By doing so, band drains also accelerate the rate of strength gain of the in-situ soils. Band drains are used in consolidation situations where the soil to be treated with moderate to highly compressible soil with low permeability and fully saturated in its natural state. The soil should be either normally consolidated or under consolidated before loading. The loading should exceed maximum past consolidation pressure for the band drains to be beneficial. Figure 3.1 shows a Typical Polymeric Vertical Drain (Band Drain).

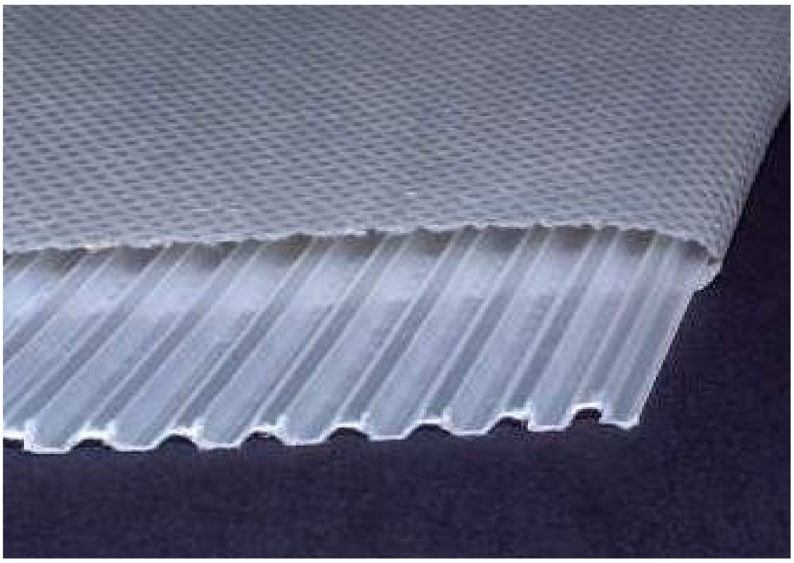

Figure 3.1. A Typical Polymeric Vertical Drain (Band Drain)

Displacement methods generally install band drains. The mandrels used with band drains are hollow and usually rectangular or trapezoidal in cross-section. The mandrel covers and protects the band drain material during installation. All installation methods employ some form of anchoring system (generally using a disposable end shoe) to hold the drain in place when the

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mandrel is withdrawn. The commonly used methods employ an installation mast (called 'Stitcher') which contains the material reels, mandrel and provision for installation force. Added to this is a carrier, a crawler excavator or crawler crane, depending somewhat on the installation depth. Usually, for drain installation depth up to 20 m, the mast can be mounted on a crawler excavator. Drains requiring depth greater than 20 m often require an installation mast mounted to a crane to provide stability. The most important criteria for the method of installation are the size of the installing mandrel. The mandrel should be kept to a minimum size, usually not greater than 80 cm^2, unless a larger size is required for penetrating to a greater depth. Although the equipment is available to work over slopes, a level granular surface containing no large obstructions is ideally required for band drain installation. A sufficient headroom is also required for its installation. A thumb rule for headroom required would be 3 m longer than the depth of installation. Band drains have been installed up to 60 m depth by using specialized equipment.

It is essential to recognize that band drains serve no structural function. Providing a shorter drainage path provides a faster release of excess pore pressure, thereby resulting in faster settlement and quicker strength gain through consolidation. For sites with a stability problem, the soil will initially have the same strength with or without the band drains installed. Further band drains do not play any role in secondary consolidation and are not suitable for the soil like peat and organic clays as the major part of settlement is from secondary compression. Therefore, in cases where secondary consolidation is expected to be significant, it is necessary to provide excess surcharge and/or extended waiting periods before the final construction. It is not recommended to install band drains where pre-drilling is necessary for installation. A drainage layer of coarse sand or gravel is provided above the ground to drain water from band drains. Generally, the sand layer is provided for a thickness of 0.5 to 1.0 m. Figure 3.2 shows typical equipment adopted for band drain installation.

Figure 3.2. Installation of band drains

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3.3.2 Typical Specifications for Band Drains (PVDs)

Band Drains (PVDs) installation and material specifications should conform to Section 700 of MORTH[48] (Ministry of Surface Transport) Specifications for Road and Bridge Works. The following additional points are suggested:

1) The contractor shall furnish all necessary labour, equipment, and materials and perform all operations necessary to install vertical drains following the details shown on the plans and the requirements of these specifications. The drains shall consist of a band-shaped plastic core enclosed in a suitable filter material and shall be spaced and arranged as shown on the plans or otherwise directed by the engineer.

2) The drains shall be installed with approved equipment of a type that will cause a minimum disturbance of the subsoil during the installation operation. The prefabricated drains shall be installed using a mandrel or sleeve that will be advanced through the soil to the required depth. A constant load or constant rates of advancement methods are the preferred methods. A vibrator will be used in areas where constant rate/load cannot install the drains to the designed depths. Drains that cannot be installed to design penetration using only constant rate/load methods may be advanced with the use of a vibrator that is mounted to the machine.

3) A drain may be abandoned before reaching design penetration when the installation rate is less than 15 cm per second with the full static force and maximum dynamic forces applied. Use of falling weight impact hammers will not be allowed. One must provide provision for introducing water into the top of the mandrel. The mandrel shall protect the prefabricated drain material from tears, cuts, and abrasions during installation and shall be withdrawn after installation of the drain.

4) The drains will be located, numbered and staked out by the engineer. The contractor shall take all reasonable precautions to preserve the stakes. The location of the drains shall not vary by more than 150mm from the locations indicated on the drawings or as directed by the engineer.

5) Drains shall be installed from the working surface to the depth shown on the drawings or to such a depth where the soil resists a reasonable effort at further penetration. The engineer may vary the depths, spacing, or the number of drains to be installed and may revise the plan limits for this work as necessary.

6) During the drain installation, the contractor shall provide the engineer with suitable means of making a linear determination of the quantity of drain material used. Splices or connections in the drainage material shall be done in a workmanlike (professional) manner and ensure continuity of drain material. There should be a 100-200 mm length of drain material protruding above the natural ground surface at each drain location. The drain material shall be cut neatly at its upper end.

7) The contractor shall be permitted to use auguring or other methods to loosen stiff upper soils before installing the drains, provided that such auguring does not extend more than 600 mm into the underlying highly compressible soils.

8) Where obstructions are encountered below the working surface that cannot be penetrated using standard and accepted procedures, the contractor shall complete the drain from the elevation of the obstruction to the working surface and notify the engineer's representative.

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At the direction of the engineer, the contractor shall then install a new drain within 500mm from the obstructed drain. The contractor shall be paid for all obstructed drains at the contract unit price unless the drain is improperly completed.

9) The contractor shall observe precautions necessary for the protection of instrumentation devices. After instrumentation devices have been installed, the contractor shall replace at his cost any equipment that is damaged or becomes unreliable as a result of his negligence.

10) Vertical drains will be measured by the linear foot (meter) for the entire length of the drain, complete and in place.

3.3.3 Installation Guidelines for Band Drain (PVDs)

a) Setting out, clearing and grubbing

1) Staking out the centre line assuming the stretch to be four lanes
2) Installation of reference pillars beyond four-lane section
3) Clearing the area by removing plants, trees, vegetation, and grubbing not exceeding 150 mm thick.
4) Marking toe line of two lanes (widening section) on ground
5) On reference pillars, lines to be marked indicate the initial embankment levels, sand blanket and embankment's pre and post-settlement.
6) Establish temporary benchmarks at 5 locations uniformly distributed over the stretch.

b) Laying the bottom layer of the embankment

Laying embankment to a thickness of 730 mm in layers (compacted layer not exceeding 200 mm thickness) compacting to at least 95% of MDD (maximum dry density) conforming to MORTH[48] (Ministry of Surface Transport) specifications to facilitate movement of machinery and easy draining out from sand blanket/geosynthetics.

c) Installation of band drains

1) All the band drain locations are to be marked precisely as per the drawing with reference to existing reference pillars. The area is to be divided into three zones depending upon the height of the embankment to be constructed.

2) Band drain locations (grid) are laid out on the ground using 200 mm long (approx) steel rods, which can be later utilized as anchors for band drains. The care must be taken to ensure the distance between marker rods to be 1 m using a template.

3) Hydraulic stitcher mounted on suitable equipment is positioned.

4) The tip of the band drain (200 m to 300 m roll depending on the brand) is fitted to the side of the stitcher and guided into the mast, and is led down the mandrel. (Mandrel is a steel box section pushed down the hydraulic stitcher for installing the band drain).

5) The band drain is attached to the mouth of the mandrel using the 10 mm/ 12 mm diameter steel rod (which was used during layout for band drains) on which the band

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drain is rolled over with a lap of approximately 20 cm and locked in the mandrel) or any other suitable disposable end plates can also be used as an anchor.

6) The stitcher is marched to the exact location of the band drain and erected vertically using the hydraulics of the base machine. The mandrel, along with the band drain, is pushed into the soil till refusal. One must take care that the band drain width is perpendicular to LS (or is along the cross-section).

7) The depth to which band drain is driven is to be recorded, as per the lug travelling down the stitcher on the face of which scale is indicated. A record is maintained of the depth, location and date and other details in the Main Borehole. During the process, it is to be seen if the band drain has come up while the mandrel is pulled back, and measurement to be corrected accordingly.

8) The mandrel is pulled back immediately, leaving the anchor rod with one end of the band drain in stiff/hard soil. If the anchor rod is not embedded correctly, the band drain comes out, and the same is to be driven again in the near vicinity within 10 to 20 cm.

9) As soon as the bottom of the mandrel is pulled up, the drain is cut off at 0.45 m above the ground level using a template.

10) Average (for five measurements) distance between the bottom of mandrel and ground level is noted/recorded to assess the quality of band drain correctly. The debris around the band drain shall be cleared away and not allowed to contaminate the sand blanket.

11) The stitcher is relocated to the following location, and the cycle is repeated. The edge of one roll of band drain is pushed into the new roll by about 45 cm and stapled to keep the process continuous.

12) At the end of each roll, the length is reconciled with the number, depth of band drain and wrapping on anchor rod etc., and the same is recorded.

d) Laying of sand blanket

1) After installing the band drain, a blanket of coarse sand as per specification (Zone I sand with a maximum of 5% passing 300 micron sieve) is laid, taking care to see that the band drains are maintained near vertical (not less than 600 to the horizontal).

2) To keep the band drains vertical, a mound of sand can be used to support the same or otherwise, a stick/rod of one-meter length could be used before placing the sand. The rod should be removed before laying the geotextile.

3) The sand blanket is to be constructed in suitable layers.

4) During construction of embankment in layers and while providing sand blanket, laid down quality control tests are conducted to ensure the quality of the material and construction.

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e) Laying of geotextile

The finished sand blanket is covered with an approved geotextile layer, as shown in the drawings. Before laying the geotextile layer, it is to be ensured that the top surface of the sand blanket is smooth, free of sticks, sharp objects and debris, which may damage the fabric. The manufacturer's instructions are to be adhered to while laying the geotextile.

f) Laying upper layer of embankment and subgrade

The embankment level before the settlement is to be kept to such a height as directed by the engineer.

The embankment to be constructed to the required height layer by layer, ensuring at least 95 percent compaction as per MORTH[48] (Ministry of Surface Transport) specifications clause No 305. Tolerances in longitudinal and transverse directions are to be checked and laid down; quality control tests to be conducted and results kept on record. 500 mm thick subgrade is to be constructed with approved material layer by layer as per MORTH[48] (Ministry of Surface Transport) specifications with at least 97 percent compaction. The levels are to be checked to be within tolerance limits, and quality control tests to be conducted and kept on record.

g) Finishing of formation

After the subsoil has achieved the desired degree of consolidation as certified by the engineer, finish the embankment to the required camber and slopes before laying the first pavement layer of the sub-base course.

3.3.4 Stone columns

Stone columns comprise boreholes of designed diameter made at a specified distance apart in the soft soil, backfilled using stone aggregates and compacted. The diameter of stone columns varies from about 0.4 m to 0.7 m, and their spacing varies from 1.5 m to 3.5 m. This method is used in soft subsurface soils to accelerate settlement and provide a sufficient increase in strength to minimize settlement and prevent deep-seated shear failure. However, the stone column technique would be comparatively costlier than providing polymeric vertical drains. Hence stone column technique is selectively adopted to support structures that are sensitive to a large amount of settlement or cases where it is also required to increase the bearing capacity of the sub-soil. IS 15284[40] (Part 1) provides guidelines for the design and construction of stone columns.

At locations where undisturbed shear strength of clayey soil (S_u) is lower than 15 kPa, providing stone columns may result in considerable wastage of stone aggregates and geosynthetic-encased stone columns (GEC) may be adopted in such places. The seamless casing is obtained through a woven geotextile, made of high tenacity PolyEthylene Terephthalate (PET), and twisted on a circular loom. The main advantage of GECs in comparison with non-geosynthetic encased stone columns is that the confinement stress in the soft soil can be much less due to the radial confinement effect of the encasement. The geosynthetic encasement also prevents the lateral squeezing of aggregates when the stone columns are installed in very soft soils, leading to minimal loss of aggregates and quicker installation.

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3.3.5 Basal Reinforcement

Geosynthetic reinforcement at the embankment base is often referred to as basal mattress and can be considered as comprising of a reinforcing element such as geogrid or high strength geotextile or similar reinforcing element placed in a frictional layer which is generally a gravel layer. It is also a necessary and common practice to provide a geotextile separation layer at the interface of soft subsoil and the gravel layer.
The use of reinforcing layer served the following functions:

- Construction is facilitated as machinery can move conveniently above the basal mattress for placing the fill.
- Basal mattress as above provides good drainage.
- The tensile reinforcement provides an improvement in the rotational stability of the embankment.
- Field experience shows a partial control of the differential settlement.

Geosynthetic elements are chemically inactive, non-biodegradable and hence durable. The shear resistance of the foundation generally governs the design of the embankment on soft ground. The inclusion of geosynthetic reinforcement at the foundation level could enhance the performance of the embankment as it resists the shear failure in the embankment as well as in the soft soil. Basal reinforcement stabilizes an embankment over the soft ground by preventing the lateral spreading of the fill, extrusion of the foundation and rotational failure (figure 3.3). This stabilizing force is generated in the reinforcement by shear stresses transmitted from the foundation soil and fill, which place the reinforcement in tension. It has also been experienced that the reinforcement can also partially reduce the differential settlement due to better distribution of stress over the soft soil. "IRC 115[20], Guidelines for the Design and Construction of Geosynthetic Reinforced Embankments on Soft Subsoils" can be referred to for design and specifications.

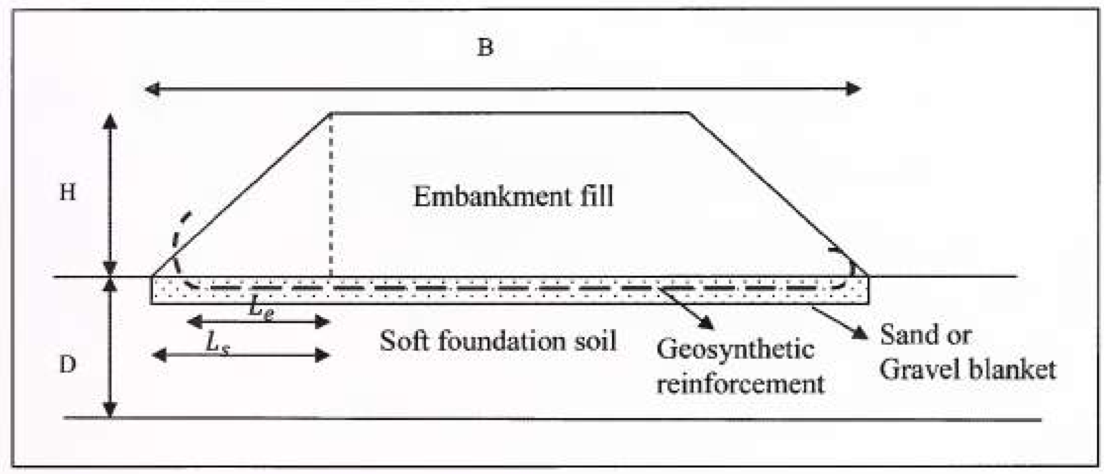

Figure 3.3. Basal reinforcement provided as a layer for stability of embankment

3.3.6 Geosynthetic Materials for Basal Reinforcement

The basal reinforcement acts similarly to the rigid layer and helps distribute the embankment load onto the subsoil evenly. The soft soil strength increases through consolidation, which is a time-dependent phenomenon. Reinforced embankment over weak foundations is to be analyzed for safety in bearing capacity. Three different geosynthetic materials can be used as basal reinforcement - geotextiles, geogrids and geocells.

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a) Geotextiles

High strength polymeric woven (HSPW) geotextiles can be used as basal reinforcement. Such geotextiles should be adequately anchored at the ends (toe of the embankment). Such type of geotextile has been successfully used as basal reinforcement in road projects mentioned below:

- About 35,000 square metre of HSPW geotextile was used as basal reinforcement for the road project at the JNPT Port Connectivity Project of NHAI at Mumbai.
- About 14,000 square metre of HSPW geotextile was used as basal reinforcement for road embankment on soft soil at Amona – Khandola Bridge Project in Goa, for Goa PWD (Public Works Department).
- About 1,00,000 square metre of HSPW geotextile were used as basal reinforcement for road embankment on soft soil for the four lanes and strengthening existing two lanes of NH-7 for Tuticorin Port Road Company an SPV (Special Purpose Vehicle) of NHAI.
- About 1,24,000 square metre of HSPW (High strength polymeric woven) geotextile was used as basal reinforcement in Visakhapatnam Port Connectivity Road Project.

The woven geotextiles are an efficient and cost-effective alternative in basal reinforcement applications in comparison to other types of geosynthetics. The slit-film geotextiles offer an added advantage in separation due to their superior strength and lower cost. While woven high-strength geotextiles openings are large enough to allow the passage of water, they are small enough to impede all but the most minute soil particles. Therefore, separation of the weak and potentially wet soils from your base material is assured. Specifically, the geotextile eliminates the pumping of weak material into your base that can occur with a geogrid. One reliable and quantifiable measure of performance among all types of reinforcing geosynthetics is Ultimate Tensile Strength (ASTM D4595[1] /ASTM D 6637[15]). With this criterion, high strength woven geotextiles outperform polypropylene geogrids by as much as 40%.

b) Geogrids

Geogrids are relatively stiff, netlike materials with large apertures, typically 10mm to 100mm between the ribs. The ribs can be fabricated from many different materials, and rib crossover joining or junction-bonding methods can vary. The primary function of geogrids is reinforcement. Geogrids were used for reinforcement in preference to metallic bars and strips for better suitability. Figure 3.4 shows both the mono-oriented and bi-oriented extruded types of geogrids. Each type begins as a geomembrane sheet that has a uniform and controlled pattern of pre-punched holes.

The punched sheet is drawn mechanically to the required extent. This stress causes the holes to deform and elongate in the direction of movement. In the uniaxially deformed product, circular holes become elongated eclipses. The eventual draw ratio, approximately 8 to 1, causes the polymer to reach a post-yield state. Its molecular structure is highly elongated in a referential direction where strength, modulus and resistance to creep increase dramatically over the original non-deformed material. In bi-oriented geogrids, the pre-punched polymer sheet is drawn longitudinally and then transversally, forming near square or rectangular apertures. A similar increase in the mechanical properties is obtained in bi-oriented geogrids. A high-density polyethylene sheet is used for mono-oriented geogrids, and a polypropylene sheet is used for bi-oriented geogrid. Mono-oriented geogrids are used in reinforced earth wall construction, whereas bi-oriented geogrids are used as basal reinforcement for load distribution.

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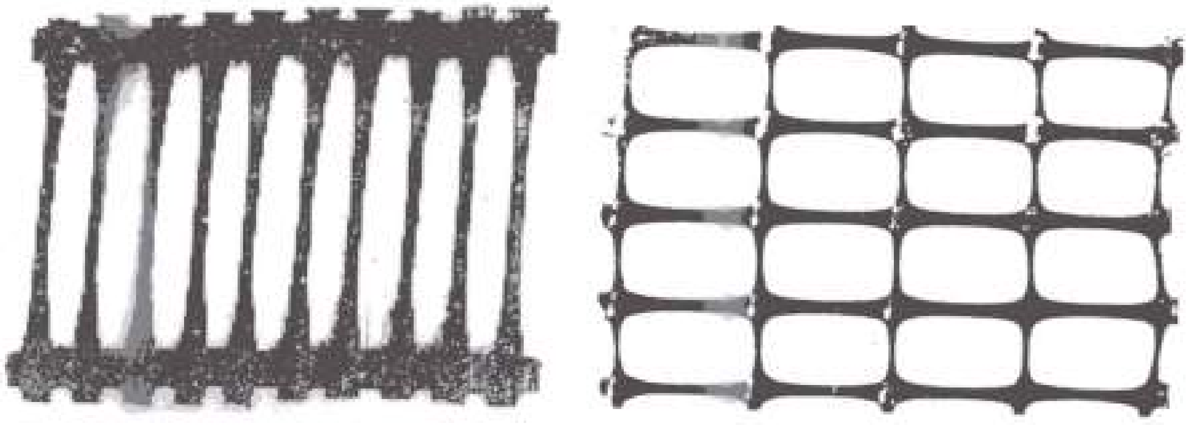

Figure 3.4. Bi-oriented and mono oriented extruded geogrids

The second category of geogrids is more flexible-- textile-like geogrids. These use bundles of polymer or bitumen-coated polymeric fibres as the reinforcing component. In this process, hundreds of continuous fibres are gathered together to form yarns woven into longitudinal and transverse ribs with large open spaces between them. The cross-overs are joined by knitting or intertwining before a subsequent coating protects the entire unit. The usual coating materials are bitumen, latex, or PVC. Such types of geogrids are known as 'Woven geogrids'. In such geogrids, polymeric yarn having high tensile strength is used, and several such yarns are combined to make up one strand of woven type of geogrids. Several such strands are then woven (interlocked) to prepare a woven type of basal reinforcement geogrid.

Geogrids rely on "interlock," a process whereby the soil particles or aggregate "lock" into place within the geogrid's open structure or apertures. It is critical to have the soil particles or aggregates of sufficient size to interlock with the geogrid properly. Often, the actual particle size is not known and/or varies widely. As a result, the amount of interlock provided may be in question.

c) Geocells

Another form of basal mattress that may be used in embankments is a geocell mattress, a three-dimensional honeycomb structure formed from a series of interlocking cells. It should have adequate tensile strength. Geocells are strong, lightweight, three-dimensional systems fabricated from ultrasonically-welded High-Density Polyethylene (HDPE) strips that are expandable on-site to form a honeycomb-like structure. Geocells are filled with compact non-cohesive soils which are confined within the cellular walls. The composite forms a rigid to semi-rigid structure. The depth of the geocells and the size of each cellular unit can vary as per design requirements. Generally, the infill is sandy or gravelly material. However, the infill may be plain concrete depending on the application, such as erosion protection, water channel formation, etc. The surface of the geocell is textured to increase soil-geocell wall friction. The geocell wall is punctured to assist in immediate dissipation of developed pore water pressures due to increased stresses within the infill of the individual cells. Figure 3.5 shows geocells as closed-form as supplied from the factory, and the open form ready for infilling. Geocells can be used to great advantage considering that:

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1. Geocells are the only prefabricated three-dimensional geosynthetics with significant third dimension properties.
2. They are easily transported as flat strips, welded width-wise at regular intervals, and therefore, logistics for large quantities is not a problem.
3. Geocells are easy to install and do not require skilled labour. They can be installed in any weather condition.
4. The infill has essentially to be non-cohesion material; however, the material could be recycled material.

Figure 3.5 shows geocell - (a) closed form as supplied from factory, and (b) open form ready for infilling.

(a) **(b)**

Figure 3.5 Geocell (a) closed form as supplied from factory, (b) open form ready for infilling

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Chapter 4 - Geosynthetic Usage for Roads in Port Areas

In addition to considerably heavy traffic loading due to container and HTV movement, coastal (port area) roads face adverse environmental conditions like high rainfall, high water table, soft clayey sub-soil prone to settlement, salty environment, etc. As a result, the service life of coastal (port area) roads is limited, and such roads often provide serviceability lower than roads built in lands. The design life of coastal (port) roads can be improved by geosynthetic products for reinforcing the pavement and ensuring proper drainage. Geosynthetics can be used for road works in port areas in various manners as enumerated below:

4.1 Geosynthetics in Bituminous Layers

The primary purpose of incorporating geosynthetics in a bituminous layer is to reduce reflective cracking in bituminous overlays and to resist moisture intrusion into the underlying pavement structure. Geosynthetics can be part of an overall rehabilitation strategy that will include placing a new wearing/surface course below the hot mix bituminous layer. Paving fabric is laid in between two bituminous layers as part of pavement strengthening to provide a water-resistant membrane and crack retarding layer. Tack coat used to impregnate the fabric and bond the fabric to the pavement shall be paving grade bitumen being used at the site for bituminous work. The introduction of paving fabrics has been reported to reduce the required design overlay thickness. However, these guidelines do not recommend any reduction in design overlay thickness, and rather its introduction is to enhance the performance of the pavement.

Geogrids: The main function of a geogrid in a hot mix bituminous layer application is to retard the occurrence of reflective cracking. In evaluating the appropriateness of use, cracking in the existing structure should be limited to cases in which the crack faulting does not fluctuate significantly with traffic loading and crack width does not fluctuate significantly with temperature differentials.

The pavement should be structurally sound, with existing cracks limited to less than 9 mm in width. Hence, bituminous layers with low to moderate alligator cracking or random cracking may benefit from grids. In contrast, widely spaced thermal cracking or underlying rocking/ faulted PCC slabs will probably not benefit. It is necessary to repair localised highly distressed/ weak areas and apply a level-up course of bituminous overlay before using the geogrid.

A variety of geogrids, including glass geogrids, are available for this kind of application. Installation of this type of product has proven to be problematic. It will result in premature failure (fatiguing) of the surfacing overlay where a lack of bonding (surface to the grid to level-up) occurs. It is highly recommended that the manufacturer's installation procedures be strictly followed and that a manufacturer's representative is present during the planning and construction process.

Paving composites and membranes: These products provide a moisture barrier in addition to varying degrees of resistance to reflective cracking. Application guidelines are similar to those recommended above for the geogrid. The impermeable qualities of these products can be a double-edged sword in which they prevent trapped moisture within the structure from transpiring out. If the lower mixes are susceptible to moisture, this may result in debonding of bituminous overlay and/or stripping of bituminous layers below the product, especially if the lower mixes are moisture susceptible.

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If the surfacing overlay is permeable and surface moisture cannot readily escape the section laterally (mill and inlay technique is especially prone), stripping of the surface mix may also occur. Detailed specifications and construction methodology for paving fabrics used in bituminous layers are given in MORTH[48] (Ministry of Surface Transport) Specifications for Road and Bridge Works, Section 700 and IRC SP:59[19].

4.2 Geosynthetics in Granular Pavement Base / Sub-base

Geosynthetics are placed in granular pavement bases to perform one or more of the following functions: reinforcement, separation, and filtration. Base reinforcement refers to the addition of a geogrid or composite or a geocell at the bottom. It also refers to a base course to increase the structural or load-carrying capacity of a pavement system by transferring load to the geosynthetic material. The primary mechanism associated with this application is lateral restraint or confinement of aggregates in the base. Wherever weak subgrades exist, the geosynthetics can increase the bearing capacity.

There have been assertions that the resultant increase in restraint or confinement should allow for the design of thinner structures using these products versus structural designs which do not. However, their benefits may only be noticeable over the long term, and there appears to be an absence of long-term controlled monitoring.

a) Geogrids

The primary function of geogrid is improving the geotechnical material property by reinforcing and stabilizing. Based on the function and applications of geogrid, its properties are classified generally into three categories like physical, mechanical and endurance properties.

Many of the physical properties of geogrids, including the type of structure, rib dimensions, junction type, aperture size, and thickness, can be measured directly and relatively straightforward. As reinforcing is the primary function of geogrid, the tensile strength of a single rib or at the junction of geogrid is considered as mechanical properties of geogrid. The tensile strength is the maximum resistance to tensile stresses mobilised under applied loads and/or installation conditions. Nodal strength tests and wide width strength tests are general tests to be conducted to determine the tensile strength of geogrids. When assessing a geogrid's tensile strength, the initial tendency is to pull a single rib in tension until failure and then to note its behaviour. A single rib tension strength test merely uses a constant rate-of-extension testing machine to pull a single rib to failure, as described in ASTM D 6637[15].

The wide-width tensile strength of a geogrid in its machine direction is important for unidirectional geogrids. For the bidirectional geogrids, both machine and cross-machine directions are important. ISO 10319[21] or IS 13325[44] or ASTM D 6637[15], wide-width strength testing of geogrids, can be referred to determine the tensile strength of geogrids.

Before laying of geogrid, the surface shall be adequately prepared, cleaned and dressed to the specified lines and levels as shown on the drawings. The geogrid shall be laid within the pavement structure as shown on the drawings. Geogrid reinforcement shall be placed flat, pulled tight and held in position by pins or suitable means until the subsequent pavement layer is placed. No vehicle shall be allowed on geogrid unless covered by at least 150 mm thick sub-base material.

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b) Geocells

Geocells are three-dimensional honeycomb-like cellular confinement systems. Geocells are fabricated from ultrasonically-welded HDPE strips that are expandable at the site to form the honeycomb cellular structure. The cells of a geocell system are filled essentially with cohesionless soil for rigidity. The cell walls are perforated for pore water pressure relief and soil-to-soil interaction. The walls are also textured for better soil-cell wall interaction.

Geocells have been used to reinforce components of flexible pavements (figure 4.1), thereby reducing the pavement thickness for the required traffic, resulting in savings of project time and overall costs. The maintenance required is also reduced. Geocells have also extensively been used for paving in container yards where thick concrete layers have been done away with, thereby reducing the time and cost of construction. Geocells along embankment slopes have been successful in preventing bank erosion.

Figure 4.1 Geocell panels being laid out for road

Detailed specifications and construction methodology for geosynthetics to be used as reinforcement in pavement base / sub-base layers are given in IRC SP:59[19].

4.3 Geosynthetics for Separation / Drainage

Geosynthetics used for separation have classically been applied to prevent subgrade soil from migrating into the unbound base (or subbase) or prevent aggregates from migrating into an unbound base (or subbase) the subgrade. A small quantity of fines introduced into the granular base can significantly reduce the internal friction angle and render the flex base weaker. Potential for these circumstances increases where wet, soft subgrades exist. Typically, a geocomposite will be used for this application, placed at the subgrade/unbound base interface. Geotextile separators act to maintain the permeability of the base materials over the life of the section, and they allow the use of more open-graded, free-draining base and subbase materials.

Apart from separation, geotextile can also function as a drainage layer. While both woven and non-woven geotextiles can be used for separation function, non-woven geotextiles are considered to be the better choice for drainage applications. Using a geosynthetic drainage layer can enhance the efficiency of conventional drainage layers like granular sub-base layers.

Detailed specifications and construction methodology for geosynthetics for separation/drainage in a road pavement are given in IRC SP:59[19] and MORTH[48] (Ministry of Surface Transport) Specifications for Road and Bridge Works, Section 700.

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4.4 Embankment Slope Protection against Soil Erosion

Road embankments experience a high degree of damage due to erosion from torrential rains in coastal areas, and hence erosion protection of embankment slopes should receive special attention. Erosion due to surface runoff would be the principal cause for the failure of road embankments in the aftermath of a cyclone disaster in coastal areas. The nature of the soil and the impact of raindrops are determinant factors in the erosion process. Silty and sandy types of soils are more susceptible to erosion than clayey soils. Distress in the form of rills to gullies and finally to erosion ditches develop when the intensity of rainfall is high, and the slope is steep. These problems will impair slope stability if not controlled with proper protective measures. The surface protection of embankment against the action of rain and wind is usually achieved by promoting vegetation growth. Geosynthetics that can be adopted for erosion protection of roads built in cyclone-prone areas are briefly described below. IRC SP:59[19], 'Recommended Practices for Treatment of Embankment and Roadside Slopes for Erosion Control' can be referred to for more details.

a) Promotion of vegetative turfing by using jute/ coir netting

The growth of appropriate vegetation on the exposed soil surface is facilitated by the use of natural (agro-based) geotextiles such as open weave jute geotextiles (JGT) or coir netting. Such nettings laid on slopes provide a cover over exposed soil, lessening the probability of soil detachment and reducing the velocity of runoff, the principal agent of soil erosion. Natural geotextiles may bio-degrade within one to three years. Despite this, agro-based geotextiles facilitate the rapid growth of dense vegetation during its service life. Once dense vegetation develops on the slope, plant cover would prevent erosion and be self-sustaining. Hence biodegradability of jute/ coir nettings cannot be considered as a drawback in areas that experience adequate precipitation to ensure green vegetation cover throughout the year.
For more details and specifications of this technique:
IS: 14986[41] 'Guidelines for application of Jute Geotextile for rainwater erosion control in the road and railway embankments and hill slopes',
IS: 15869[42] 'Open weave coir Bhoovastra-Specification' and IS 15872[43] ' application of coir geotextiles (coir woven Bhoovastra) for rainwater erosion control in roads, railway embankments and hill slopes-Guidelines'.

b) Erosion control using two dimensional (2–D) synthetic geogrids/ Geosynthetic nettings

Geosynthetic nettings/ geogrids can be used to promote vegetation growth on barren slopes similar to biodegradable nettings. Under erratic weather conditions, successful vegetation growth and its sustenance depend on un-seasonal rainfall. Hence, longer life of reinforcing material would be required for ensuring vegetation growth apart from contribution from the mesh towards reduction in velocity of surface runoff. Agro-based nettings may fail to provide erosion prevention in areas that experience a repetitive change in climate, prolonged drought in particular. The use of polymer geogrid mesh provides permanent protection as it is not biodegradable, long-lasting and has an almost unfailing success rate for vegetation growth, year after year.

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c) Three-dimensional erosion control mat / Rolled erosion control products

Relying upon vegetation growth alone may sometimes be unpredictable and unreliable as it may be extremely difficult to achieve 100% vegetation coverage, leaving exposed areas vulnerable to erosion. Furthermore, vegetation may sometimes dry up or become diseased, reducing its erosion control capability. Reinforced vegetation (or reinforced grass) is a better method that can be adopted for enhancing slope stability and erosion control. Such erosion control products are usually three-dimensional mats, having multi-filamented materials of specified thickness. Such materials are Rolled Erosion Control Products (RECPs)/ 3-D Mats and 'Turf Reinforcement Mats' (TRM). While mats made using natural fibres last for one to two years, polymeric mats are used in situations where such products are required to last for a longer time. 3-D mats having a wide-ranging variety of strengths are available. The material used for manufacturing these mats also varies. Hence following general specifications are given in Table 7 for guidance. However, field conditions like harsh areas/ high survivability requirements may warrant the use of 3-D mats with tensile strength as high as 35 kN/m or even more.

Table 7 Property Requirements for 3-D Mat

3-D Mat Property	**Specified value***	**Test Method**
Minimum Tensile Strength	2 kN/m	ASTM D 5035[11]
UV Stability (Min % tensile strength retention)	80%	ASTM D 4335[12] (500-hour exposure)
Minimum thickness	6.5 mm	ASTM D 6525[13]
Mass per unit area (Minimum)	250 gm/ m^2	ASTM D 3776[14]

* Minimum Average Roll Values, machine direction only for tensile strength test

d) Preformed polymer geosynthetic cells (geocell) or webs

Often, embankments are constructed in areas where vegetation may be challenging to grow, and therefore, the erosion problems might be severe due to water bodies. It may also be not possible to mitigate potential erosive forces that are likely to overcome the strength of the root system. In such cases, 'geosynthetic cells (geocell)' can be adopted. However, geocells would be relatively more costly than all other techniques outlined above.

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Chapter 5 - Design Guidelines

5.1 Application of geotubes and gabion box for coastal protection

Geotubes are large container bags made up of woven and non-woven geotextile fabrics; those can be filled with a large number of granular materials which are partially permeable. A successive utility has been reported by Heerten *et al.* (1984). Since geotubes are thick flexible sheets, one can arrive at any successive height depending on the filling percentage. The primary reasons for the failure of geotube are poor construction, improper alignment and false stacking, and further those results in tearing, bursting, punching, slope instability, and excessive settlement due to heap of geotube under wave attack and scouring. For a better understanding and good design, the following factors have to be taken into consideration.

The major advantage of gabions is that the structure's build is pervious, allowing free water flow from the rear. This is a helpful feature in sudden drawdown conditions. However, migration of soil particles needs to be prevented through the body of gabions by judicious use of non-woven geotextiles behind the gabions.

Steel gabions are boxes or baskets of steel wire infilled with cobble and/or gravel-sized particles (see IS 16014[39]). High-quality steel wire is profoundly galvanised for use in a saline environment. An additional protective polymeric coating is also applied for gabions that are to be used in a more aggressive environment or where a longer design life (more than ten years) is required.

Polymer gabions are prefabricated collapsible sausages fabricated by a unique process in which the ropes are woven into a continuous net with square apertures. The construction of the nets is woven jointly with the intersection of the ropes to form square/Rhomboidal mesh. The Polymer nets are used to form gabions of desired sizes and are filled with boulders. Polymer gabions are manufactured from polypropylene ropes, adequately stabilised for degradation against ultraviolet radiation. The diameter of the border and body ropes and aperture size are designed based on the type of application, installation method, and expected load conditions. The polymer used in polymer gabions is inert to the corrosive coastal environment; polymer gabions are highly preferred for coastal protection works and are ideally suited for use as an armour layer in resisting all erodible effects of waves. Polymer gabions can be done in-situ with boulders, or they can be pre-filled and then installed in place with the help of a crane. Adjacent polymer gabions are tied together with polypropylene rope to form a continuous monolithic structure.

5.2 Design water depth and wave height

The water level is subject to seasonal variation, tide, wave, wave current, yearly fluctuation and sea-level rise. Generally, water levels are higher in the monsoon and lower in the non-monsoon. Hence, in order to incorporate flood level and tides, the past 50 years of flood data is collected to arrive at significant water levels and tidal variations and combined to obtain the design water level (DWL). DWL is the maximum possible water level that estimates the risk to the structure at which the structure is designed to withstand the forces. The stability of a structure is directly related to the DWL and the forces exerted by the design wave. The

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higher the wave, the larger the forces, and therefore based on 100-year wave data, the corresponding significant wave height is rated as design wave height (DWH). The following formula will provide a reasonable and conservative design wave height, H_b for the breaking wave for simple design conditions.

$$H_b = 0.78 d_s \tag{5.1}$$

where, d_s - Depth of water at the structure toe under the DWH condition.

5.3 Significant wave height

Significant wave height (H_S) is the standardised statistical wave height of the random waves in a sea state. It is defined so that it more or less corresponds to what a mariner observes when visually estimating the average wave height. One can arrive at the significant wave height from the maximum wave height. The conservative formula for estimating the significant wave height is:

$$H_S = H_b / 1.8 \tag{5.2}$$

5.4 Design of gabion box armour

The geotube core is overlaid with gabion boxes, and these boxes are made of flexible polymer ropes of 16mm in diameter. These nets are used to form cages of desired sizes and are filled with boulders. They are ideally suited as an armour layer in resisting all erodible effects of waves. Boulder fill can be done in-situ, or gabion boxes are pre-filled and installed in place with the help of a crane, adjacent gabions are fastened together to form a continuous monolithic structure, and the polymer is inert to the corrosive coastal environment. The major advantages with gabion boxes are it allows immediate dissipation of hydrostatic pressure; it absorbs the impact of dynamic wave forces. The gabion boxes are highly flexible, durable and resistant to the marine environment. Their weight is assumed from Hudson's formula (The Kd value can be assumed as 2.0 in the absence of detailed model studies for the gabion boxes on stability coefficient).

$$W_{50} = \frac{W_r H_s^3}{k_D \left(S_r - 1\right)^3 \cot\theta} \tag{5.3}$$

where

W_{50} is the weight of an individual armour unit of gabion box (tons)

H_S is the significant wave height in meters (m)

k_D is the stability coefficient (the stability coefficient is assumed as 2)

θ is the angle of the structure measured with horizontal.

W_r = Unit weight of armour is t/m^3

$S_r = W_r / W_S$, where, the W_S = Unit weight of seawater t/m^3

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The weight of the gabion box can be calculated considering the 30% void ratio. Generally, armour stones are provided in the range of 0.75 to 1.25 times W_{50}. The individual weight of the gabion box can be increased by 50% if uniform size boxes are used.

5.5 Design of geotube

The estimation of sand-filled geotextile tube dimension shape can be calculated using the Timoshenko method. The dimensions (and thus the resultant shape) of a geotextile tube for the filling levels between 60% and 100% is a function of the radius (R) of the circular diameter of a 100% filled geotextile tube. The geotube fabric is available in woven and non-woven geotextile fabric; however, the woven geotextile fabric is preferred for the marine environment. The selection of geotube fabric can be chosen based on the type of structure, load, application, and the environment in which it is going to be exposed. The required tensile strength of the geotube also needs to be ensured during the design; the tensile strength of the geotextile fabric may reduce in a longer duration. Hence the tensile strength may need to be designed in consideration of the long-term degradation of tensile strength and creep behaviour. For the structural design of any coastal application, the following guidelines need to be satisfied:

Filled height $h_{\text{fill}} > \left(1-\sqrt{1-f}\right)D_{100}$ (5.4)

Filled width $b_{\text{fill}} < h_{\text{fill}} + 0.5\pi\left(D_{100} - h_{\text{fill}}\right)$ (5.5)

Here,

h_{fill} -The filled height of the geotextile tube in m.

f -Degree of filling concerning the cross-sectional area

D_{100} -Diameter of the geotextile tube at 100% filling in m

b_{fill} -The filled width of the geotextile tube in m.

5.6 Design of geotextile filter

The formula used for defining the geotextile pore size and particle size of the infill material for the free drainage and to prevent the formation of filter cake for dynamic wave load can be attained by

$$O_{90} < 1.5 D_{10} C_U^{0.5} \quad (5.6)$$

The filter criteria for geotube fabric depend on the infill material; the infill material characteristics need to be chosen on the aperture opening size of the geotube fabric. The infill material needs to be chosen based on the retention criteria, which means the infill material should be retained even under extreme environment and loading.

Here, O_{90} - A pore size of the geotextile tube.

D_{10} - Diameter corresponding to 10% finer.

C_U - Coefficient of uniformity of the infill material.

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5.7 Design scour depth

Scour mechanism is caused by waves breaking across the sloping front of the breakwater head. The geometry of a steep breakwater face causes later water motion that forms the tongue of the plunging breaker into a rounded re-entrant jet that impacts the bed at a steep angle and mobilises sediment. The maximum scour depth by breaking waves on the sloping structure was calculated using the following empirical equation:

$$\frac{S_m}{H_S} = 0.01\left(\frac{T_p\sqrt{gH_S}}{h}\right)^{3/2} \quad (5.7)$$

Where,

S_m is the scour depth in meters

H_S is the significant wave height in m

g is the acceleration due to gravity in m/s^2

T_p is the nearshore wave period in seconds

h is the water depth at the structure toe in m.

5.8 Design of gabion box toe and apron

Toe mound will act as a protection to the structure from scouring action of waves, normally scouring action will tend to remove subsoil below the structure this indent to instable the stability of the structure, further results in excessive settlement due to heap of geotube. The width of the toe mound shall be maximum of twice the wave height or 0.4 times the design water depth. The height of the toe mound shall be more than 0.5 times the width.

5.9 Design of revetment thickness

The minimum revetment thickness for coastal application can be estimated using IS.14262[45].1995 Indian Standard for Planning and Design of Revetment Guidelines. 2019.

$$\text{Revetment thickness } (t) = \frac{v^2}{2g(S_m - 1)} \quad (5.8)$$

where,

v = Maximum wave velocity (C_g) or water particle velocity can be considered.

g = acceleration due to gravity 9.81 m/s

S_m = Specific gravity of mattress $S_S(1-e)$

S_S = Specific gravity of stone in t/m^3

Factored thickness (t) = Factor of safety× Revetment Thickness

For the factored thickness, a minimum factor of safety of 1.5 may be considered, and the same may be subjected to vary based on the severity of the wave condition.

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Chapter 6 - Construction Methodology

Chapter 6 describes the infield application of the geotube-gabion seawall at a site in Pentha, Orissa state of India. The construction methodology adopted in the field, as well as the outline of field study., the challenges encountered over the site and its remedial measures, are taken, are mentioned as per field conditions. One such novel remedial measure undertaken due to scarcity of space is integrating the earthen embankment with the present one and thereby performing a numerical analysis (refer THOE13D015[49]) for the complete one. Also, the adequate toe protection methodology is given in form of gabion toe as well as sheet pile toe. Any novel design cannot be complete without its application in practical scenario and this work and presents the salient conclusions drawn from the study through experimental and numerical investigations.

6.1 Rubble mound sea wall with gabion box toe armour

The west coast region of south Tamil Nadu -- Kanyakumari is a highly erosion-prone zone. It is located over the Indian Ocean and Arabian Sea region; this coastal region is affected both by the southwest moon soon and northeast monsoon. The shoreline is located very proximally to the high tide zone, and it is densely populated. Fishing is the main occupation of the hamlets—the coast experiences two monsoons, namely southwest from June-September and northeast from October –December. The SW monsoon is severe along the coast, creating heavy erosion resulting in loss of valuable lands, roads, worship places and houses. Figure 6.1 shows the geographical location of Kanyakumari. As mitigation of coastal protection, it is proposed to construct a seawall with a typical cross-section as in figure.6.2. Hence as a part of designing remedial measures, various studies were taken up to evolve a suitable solution for arresting erosion.

Figure 6.1 The map showing the geographical location of town Kanyakumari

Kanyakumari is located in the reach of the Indian Ocean on the south, the Arabian Sea on the west and the Bay of Bengal on the east. This region is densely populated, and the shoreline is

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proximal to the high tide zone. Fishing is the principal profession of the communities the shore faces two monsoons, namely southwest from June-September and northeast from October – December. The SW monsoon is usually severe along the shore, creating heavy erosion resulting in loss of valuable areas, roads, worship places and houses. Kanyakumari is one such affected location adjacent to Indian ocean along the east coast, which has a latitude of 8° 05' N and longitude 77°33' E on the west coast of the Indian Ocean (see Figure 6.1). As a part of designing remedial measures, studies were taken up and evolve a suitable solution for arresting coastal erosion. Suitable remedial measures are suggested based on the beach slope, wave climate, wind, tide, and using the hydrographic data.

Design of rubble mound seawall with gabion toe

The typical cross-section shown in figure 6.2 is designed as a site-specific cross-section for a beach slope of 1:10. The bottom layer is designed with a filter layer of stone size 150 mm, the core layer of stone size 50 kg to 152 kg and the armour layer with stones of size 400kg to 500kg. The toe apron is constructed with a gabion box of 1mx1m1m size, and each gabion box is filled with 150mm above size stones. These types of typical cross-sections were constructed all over the Kanyakumari coastal region as a coastal protection structure. This structure can be adopted for a short length of 10m to 20m from the shoreline. These structures are highly efficient in energy dissipation and protect the landmass from further erosion. This kind of structure is highly advisable for the flat beach.

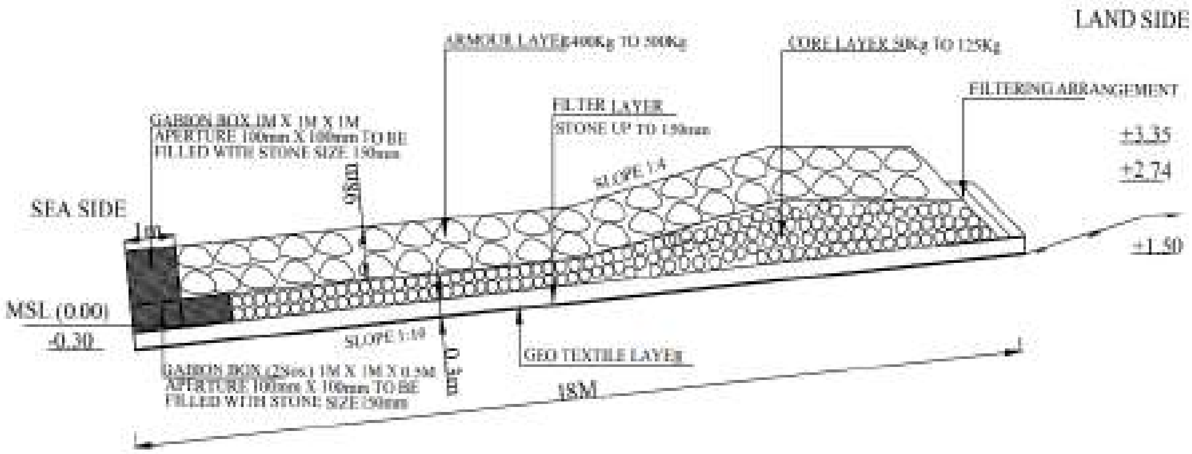

Figure 6.2 The typical cross-section of rubble mound seawall with gabion toe

6.2 Saline embankment with geotube and gabion box

Pentha (20°32.5'N 86°47.5'E) is a small coastal village in Kendrapara District of Odisha State 8.6km distance in N 117° direction from Rajnagar Town with 50,000 fishermen living there. Agriculture is a secondary occupation for these people after fishing. The coast near the Pentha village is subjected to severe erosion for the past 25 years. The saline embankment to protect the village is eroded, and a retarded embankment is built. The retarded embankment is likely to erode if not protected. The Government of India had received credit from the International

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Development Association for the Integrated Coastal Zone Management Project (ICZMP), and the Government of Odisha intends to utilise a portion of the funds for the construction of a suitable geotube embankment on the seaside of retarded embankment. The geotube embankment will protect the retarded embankment, dissipate wave energy, eliminate overtopping of seawater and pollution of agricultural land, protect against erosion etc. Figure 6.3 shows the geographical alignment of constructed geotube embankment with the low tide and high tide reaches over the shore.

Figure 6.3 The proposed location of geotube embankment

6.2.1 Reason for erosion

The tidal range pertains to Pentha varies with the high tide of +3.6 m above (MSL) and -0.5 m below (MSL) as low tide. The bathymetry is perfectly parallel to the shoreline, and the beach slope is gentle, which results in the formation of regular waves at equal intervals; since the slope is gentle, the wave breaks in the longshore bar and due to higher wave celerity and it plunges over foreshore up to berm, this results in movement of sediments from the onshore and transported back to foreshore during backwash. This results in longshore transport (littoral drift). The continuous affected of cyclones and storm surge, which associated with a low-pressure weather system, that causes the water to pile up higher than the ordinary sea level and tends to increase the wave height which is a predominant reason for the erosion of beach berms and dunes since storm surge waves are of high intensity which breaks after the longshore bar, which tends to increase gradient in transport rate in the direction of the net transport. The major reason for the erosion of Pentha is due to the circulation of currents in between the two river clusters, which discharge water into the sea.

6.2.2 Design of geotube embankment

Geotube is a large container that is made up of woven and non-woven geotextile fabrics; those can be filled with granular materials which are partially permeable. A successive utility has

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been reported by Heerten *et al.* (1984). Since geotube are thick flexible sheets, they can be used to arrive at any successive height based on the percentage of filling. The major reasons for the failure of the geotube are due to poor construction, improper alignment and due to false stacking. Further, that results in tearing, bursting, punching, slope instability, excessive settlement due to heap of geotube under wave attack and scouring and considering all the above possible failures, the design parameters are chosen for the design. The typical design cross-section is shown in figure 6.4 as reference. This typical cross-section is site-specific and different cross-sections are adopted at different locations based on the land availability, beach slope and topography. The construction included two steps: the initial part is to strengthen the existing earthen bund as temporary protection, and the second part is the geotubes core with gabion protection for wave energy dissipation.

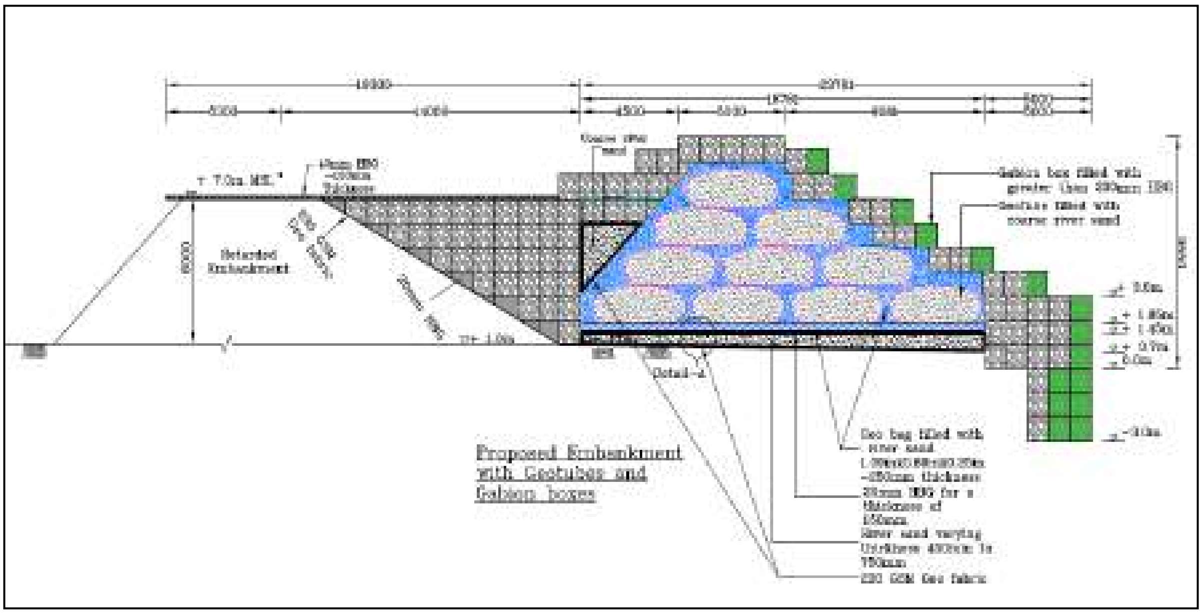

Figure 6.4 A typical design cross-section of a geotube embankment

The geotube filled with granular material will act as an impervious core, and polypropylene rope with tar coated gabion filled with a stone size of 200 mm above is used for wave energy dissipation. The size of the gabion is estimated based on the energy required to be dissipated. This project was completed in 2016 and sustaining well even after it was exposed to more than six cyclones.

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Chapter 7 - Annexure

Webinar on Draft Guidelines on Geosynthetics for coastal protection and port works - 10th and 11th Dec 2020

7.1 Index for Annexure Content

S.No	Content	Link
	Day 1	
1	Introduction - Prof. K. Murali IITM	presentation content\1_Prof Murali formatted.ppt
2	Case Studies on: Coastal Erosion and Remedial Measures- Prof. S. R. Gandhi- Director SVNIT Surat	presentation content\2_prof srgandhi formatted.pptx
3	Introduction and terminology- Mr. Guru Vittel CRRI	presentation content\3_Guru vittal approved.pptx
4	Geosynthetics Part 2 of IS 4651 and Sea wall at Pentha – Prof. R. Sundaravadivelu IITM	presentation content\4_PENTHA_SEAWALL_formatted.pptx
5	Design guidelines for geotextile applications underwater – Dr. Vijaya NIOT	presentation content\5_NIOT Prof vijaya formatted.pptx
6	Geosynthetics for ground improvement – Prof. K. Ilamparuthi Anna Univ (Retd)	presentation content\6_illamparuthi approved.ppt
	Day 2	
7	Design of bunds for river bank protection - Prof. Nilanjan Saha IITM	presentation content\7_Prof Nilanjan Saha formatted.pptx
8	Geosynthetics usage for roads in port areas - Prof. Sivakumar Babu G L, IISC	presentation content\8_Sivakumar babu formatted.pptx
9	Slope stability analysis and design of Posonallah guide bund along Dibang river, Arunachal Pradesh - Prof. R. Sundaravadivelu IITM	presentation content\9_Prof RSVelu Dibang formatted.pptx
10	Coastal erosion and role of geosynthetics - Prof. S. A. Sannasiraj IIT M	presentation content\10_Geosynthetics-sas approved.pdf

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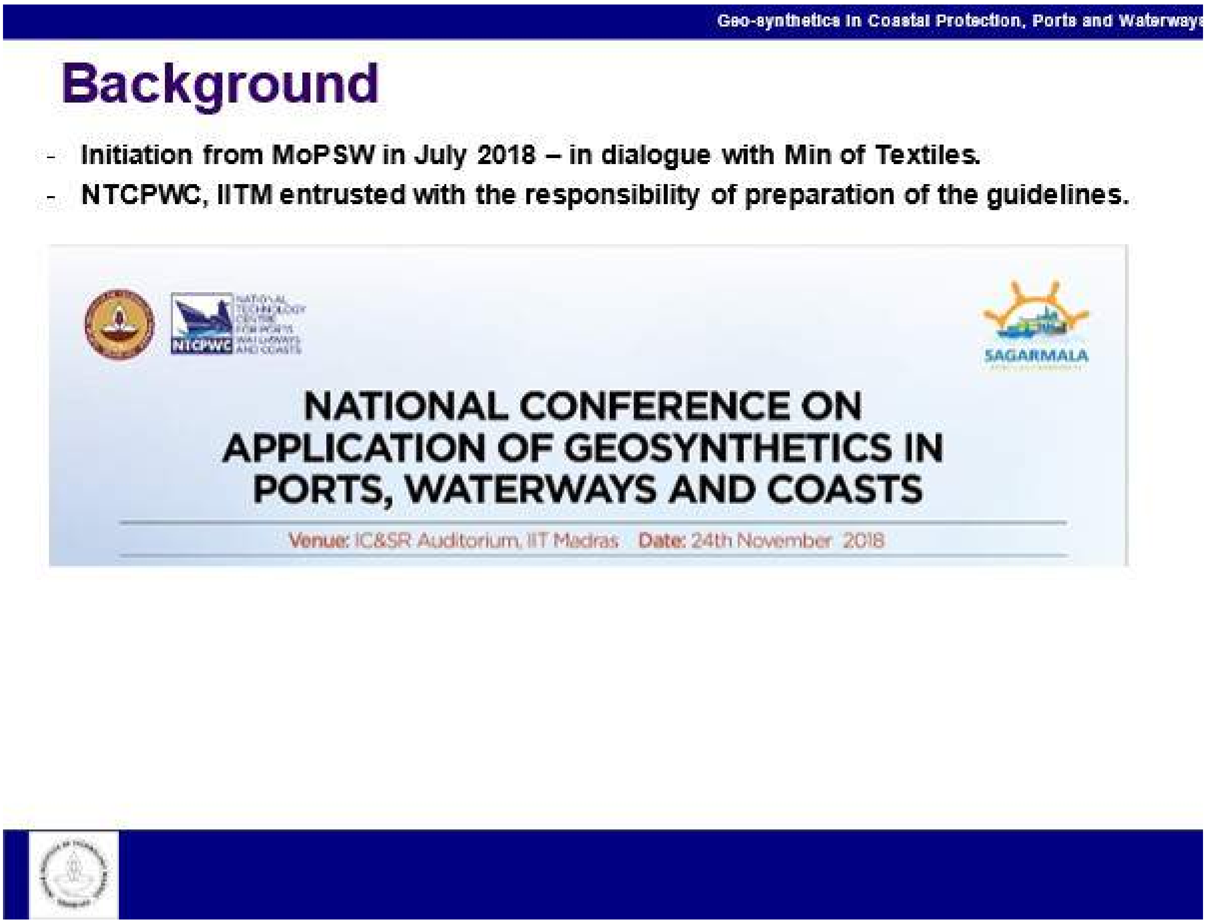
7.2 Summary of presentations

7.2.1 Introduction by Professor K.Murali, HoD, Ocean Engineering, IITM

Professor K.Murali, HoD, Ocean Engineering, IITM, gave the welcome address and briefed about the initiation to prepare geosynthetics guidelines from MoPSW in July 2018. The presentation highlighted that NTCPWC, IITM was entrusted with the responsibility of preparation of the guidelines for Application of Geosynthetics in Ports, Waterways and Coasts. Key points about the National Conference on Application of Geosynthetics in Ports, Waterways and Coasts organized for this purpose on 24th November 2018 were discussed and the Draft guidelines were released in the concluding ceremony. Followed by this introduction, a technical presentation on the applications of geosynthetics was given. Examples from Kolkata port trust, a school in Farakka located on the shore of the Ganges etc., were presented and discussed. Also detailed explanation about the understanding, the Designer must gain, and the understanding that the owner must gain was explained by way of a typical model study carried out in the laboratory on "Hydrodynamic characteristics of modular porous reef breakwaters". Few slides from the presentation are given below:



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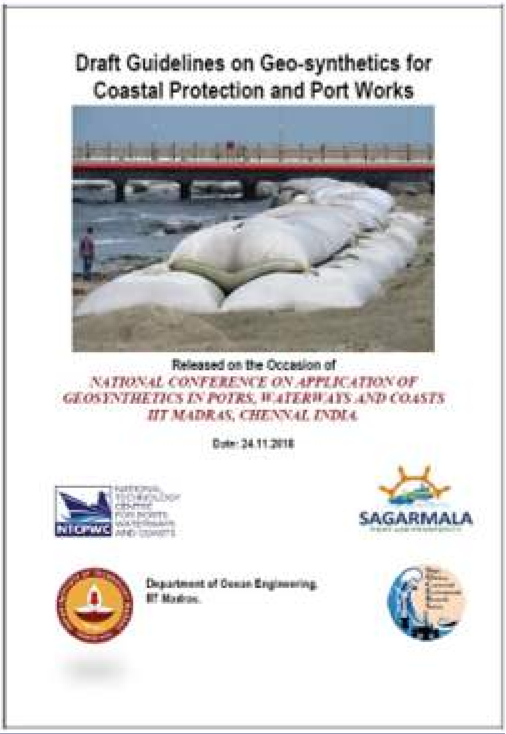
Geo-synthetics in Coastal Protection, Ports and Waterways

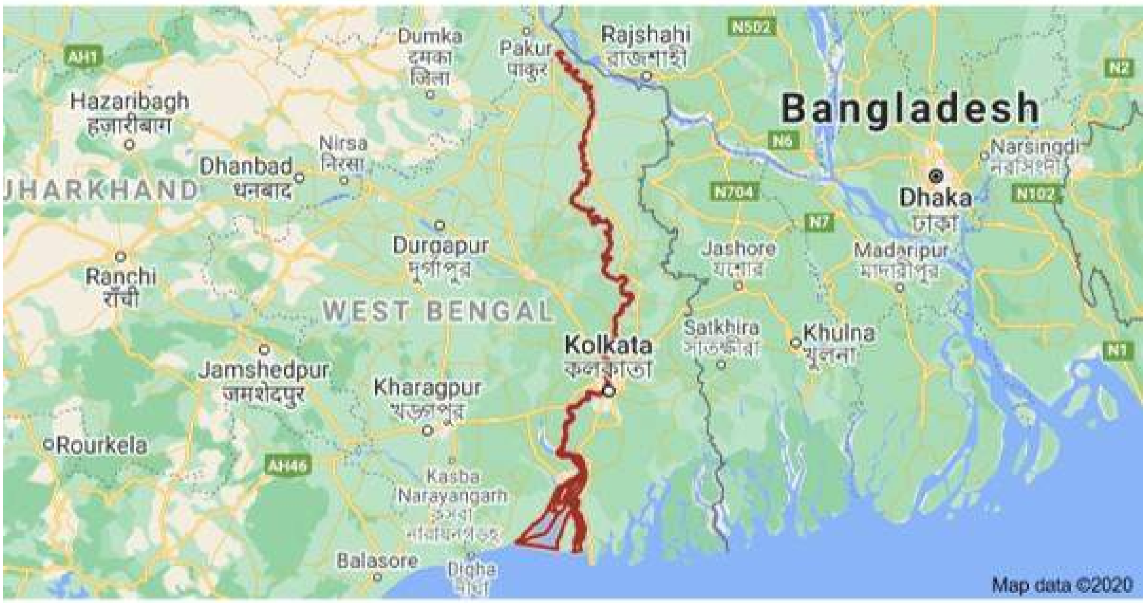
Background

- Initiation from MoPSW in July 2018 – in dialogue with Min of Textiles.
- NTCPWC, IITM entrusted with the responsibility of preparation of the guidelines.
- Draft Guidelines by 24 Nov. 2018.



Geo-synthetics in Coastal Protection, Ports and Waterways

Example – Farakka

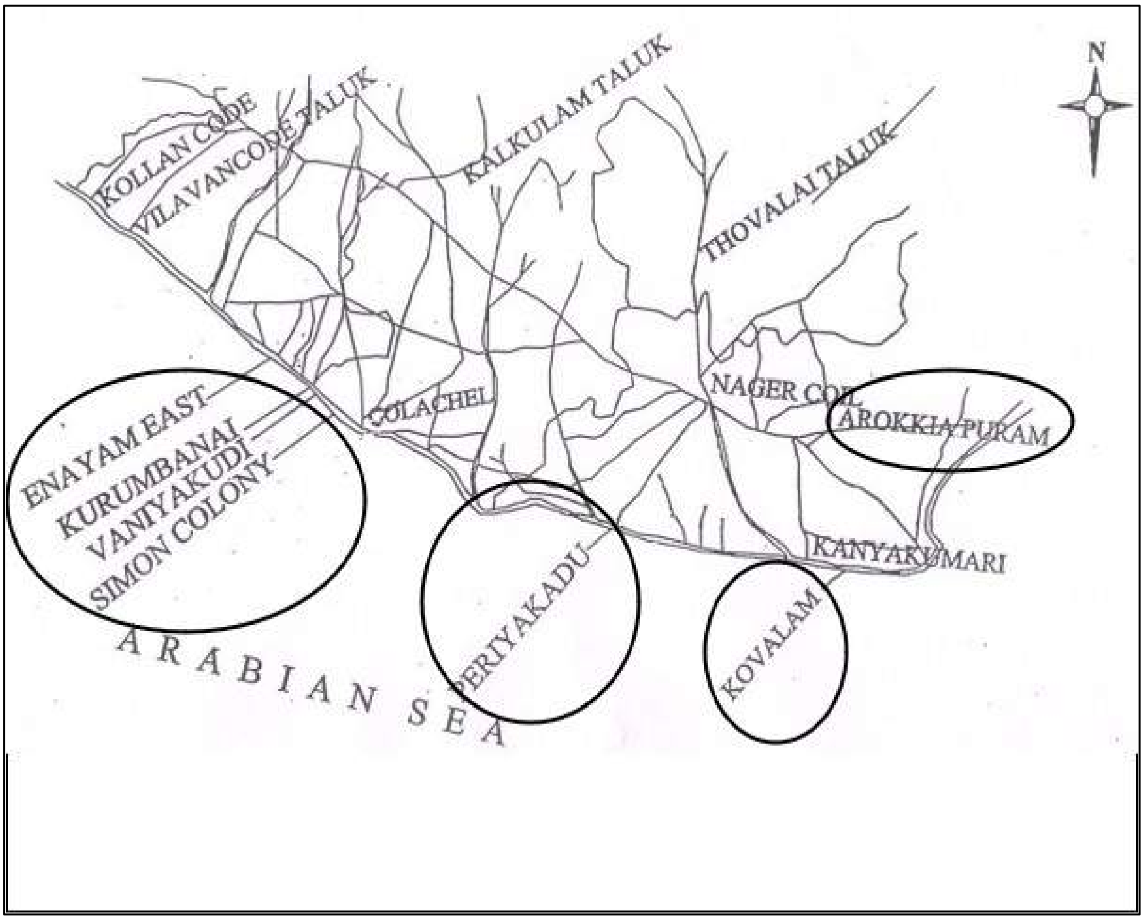


Prof.K.Murali
Department of Ocean Engineering, IIT Madras

7.2.2 Case Studies on: Coastal Erosion and Remedial Measures by Prof. S.R.Gandhi, SVNIT Surat

Professor S.R.Gandhi, Director, SVNIT Surat, delivered the Inaugural address for the webinar. Dr. S.R.Gandhi thanked the Ministry of shipping and the ministry of Textiles for giving this project to the Department of Ocean Engineering, IITM. A technical presentation was given on the Case studies on **Coastal erosion and Remedial Measures**. Some of the case studies were from post Tsunami at the southern side of Tamil Nadu. The benefits of Geosynthetic solution with Gabions in the reclamation process were highlighted. Case studies on constructing Groins at Kovalam and Arokiapuram were presented. The beach formation at Enayam after the receding of the Tsunami was discussed. The advantages of using Gabion walls over Reinforced wall construction were explained in detail and the construction process was explained by referring to various case studies.

Few slides from the presentation are given below:



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Groins at Arokiapuram

Beach formation at Enayam after receding of Tsunami

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7.2.3 Geosynthetics for Coastal Protection and Port works by Mr. Guru Vittel, Chief Scientist, CRRI

Mr. Guru Vittel, Chief Scientist, CRRI, gave his presentation on Geosynthetics for Coastal Protection and Port works. Coastal management with respect to coastal erosion – structural and non-structural measures was presented. Strategies for Coastal management were explained in detail. Alternate material which can be used without depleting the natural resources was discussed. Geosynthetic is one such material that can be utilised in various ways. The basics of geosynthetics, its classification, advantages, functions, raw material used in its manufacturing were explained in detail. Exposure was given on Geotextiles, Geogrids, Geonets, Geoties, Geomembranes, Geocells, Geocomposite, Band drains etc. Case studies on laying geotextile for constructing road over Clayey subgrade were presented. In this case study, Geotextile was used as a drainage layer and a separator between clayey subgrade and road surface. The Okhla Flyover Approach Embankment, which was the first geogrid reinforced fly ash approach embankment constructed in the country in 1996 was explained in detail. Band drain installation in Vizag was presented. Inputs were given on "Guidelines for use of Geosynthetics in Road Pavements and associated works" – IRC SP 59-2019. The various BIS Codes on Geosynthetics were collected by the presenter and displayed to the audience. The following suggestions for incorporation of Geosynthetics in IS 4651were also provided:

- Instead of geosynthetics being a part of IS 4651 – Part 3, Geosynthetics should be a separate part.
- Present draft deals with material specifications and brief construction aspects, the design aspect needs to be incorporated in the design guidelines.
- Compendium on case studies may be included.

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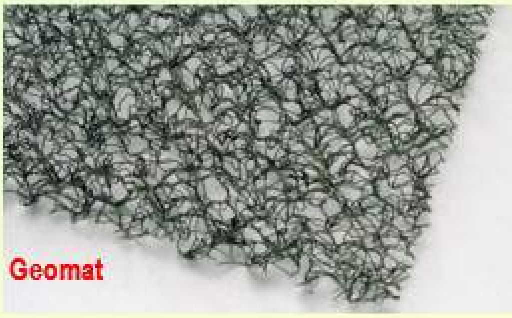
Geosynthetics

- Synthetic products used in various geotechnical applications for enhancing the serviceability / performance of an engineering structure
- Classification – Based on manufacturing process, materials used or function
  - Geotextiles
  - Geogrids
  - Geonets
  - Geoties
  - Geomembranes
  - Geocells, Geocomposite, Band drains, etc

Geosynthetics

Geonet



Geomat

Pavement Reinforcement Products

Ref. IRC SP:59

7.2.4 Geosynthetics Part 2 of IS 4651 and Sea wall at Pentha by Prof. R. Sundaravadivelu, IITM

Prof. R. Sundaravadivelu, IITM, discussed about Geosynthetics given in Part 2 of IS 4651 and Sea wall at Pentha.

The presentation started with the Sea wall at Pentha. It was highlighted that many literatures are available on the type of Sea walls constructed at Pentha which can be referred for research purposes. Design of geotube embankment should be done considering the poor soil condition and the wave force. To study its performance, only Numerical model studies are available on the geo technical stability for integrated saline embankment. There are two parts of this problem, one is Hydrodynamic stability, and another is Geotechnical stability. Geotubes were explained in detail with their advantages, properties, characteristics, behaviour etc. A case study on Sea wall at Pentha was discussed in detail. Design of Geotextile tube in Saline embankment consisting of the design of filter and scour apron, design of toe mound, design of core, design of gabion layer, gabion layer thickness, design strength of geotextile tube, was discussed in detail. Analysis for hydrodynamic coefficients using a model created in the laboratory was explained. Numerical modelling was done to analyse the physical dimensions of geotube configuration such as 4+3+2+1 vs 3+2+1. Embankment tubes of 9 m and 15 m circumference were considered, and its effects were studied using numerical study. It is found that the performance of 4+3+2+1 was much better when compared to 3+2+1. Conclusions drawn were.

- The present numerical study, patterning to geotube embankment, reveals good geotechnical stability against the scour.
- Hydrodynamic performance shows that gabion has good wave energy dissipation characters.
- More number of tubes with lesser diameter is preferred when compared to a smaller number of tubes with larger diameter.
- Sheet pile with rubble toe protection is recommended where wave energy concentration is observed.
- Sand accretion is observed after construction due to energy dissipation of Gabion box.
- Scour protection is essential.
- The geotextile bedding layer sandwiched with 500mm thick sand is distributing the load effectively to the subsoil.

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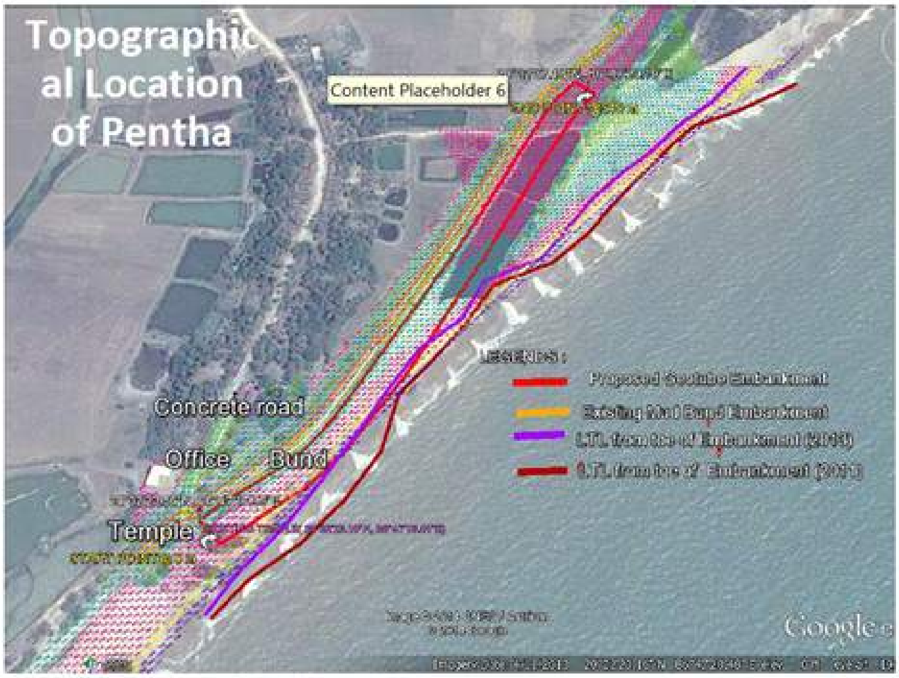
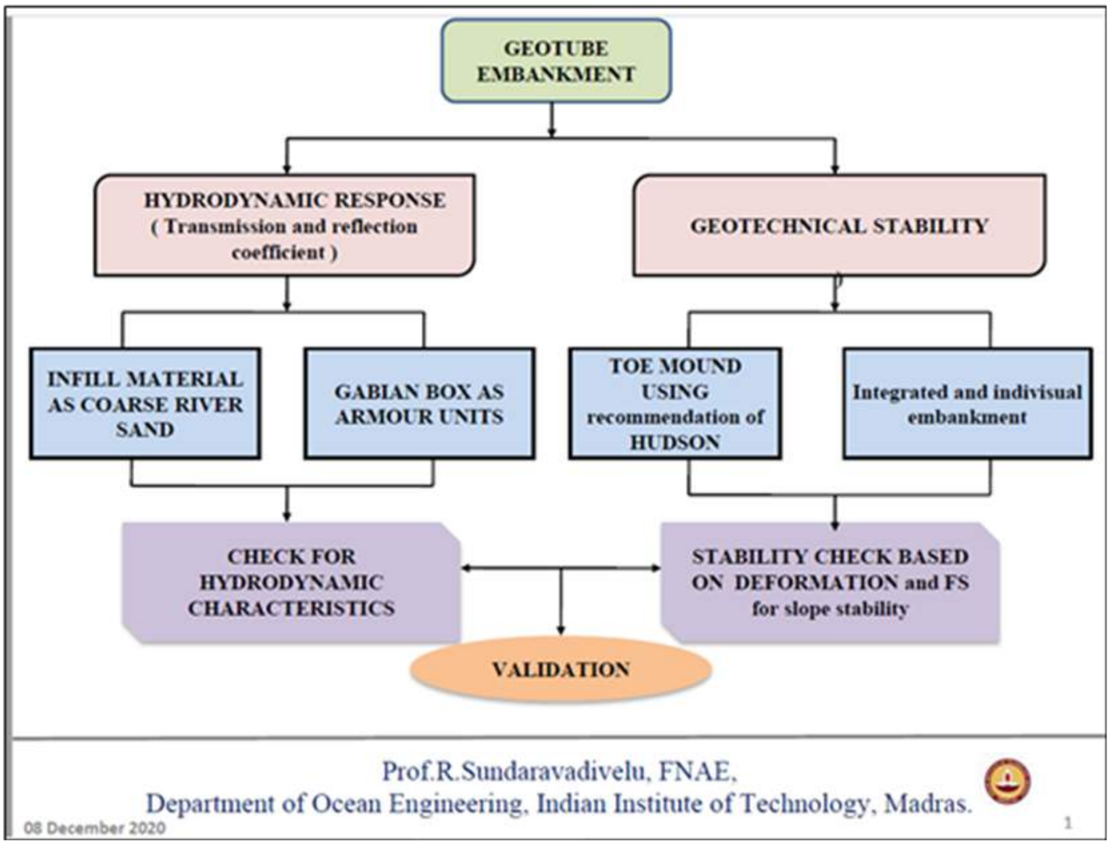
OUTLINE

- Introduction
- Literature Review
- Scope
- Objective
- Design of Geo-tube embankment
- Physical Model study on reflection and transmission of Geo-tube embankment with and without gabion
- Numerical Model on Geo-technical Stability for integrated and individual saline embankment
- Results & Discussion
- Conclusion
- Visible Outputs
- References

Prof.R.Sundaravadivelu, FNAE,
Department of Ocean Engineering, Indian Institute of Technology, Madras.
08 December 2020



Prof.R.Sundaravadivelu, FNAE,
Department of Ocean Engineering, Indian Institute of Technology, Madras.
08 December 2020



Sea Wall Embankment at Pentha, Odisha

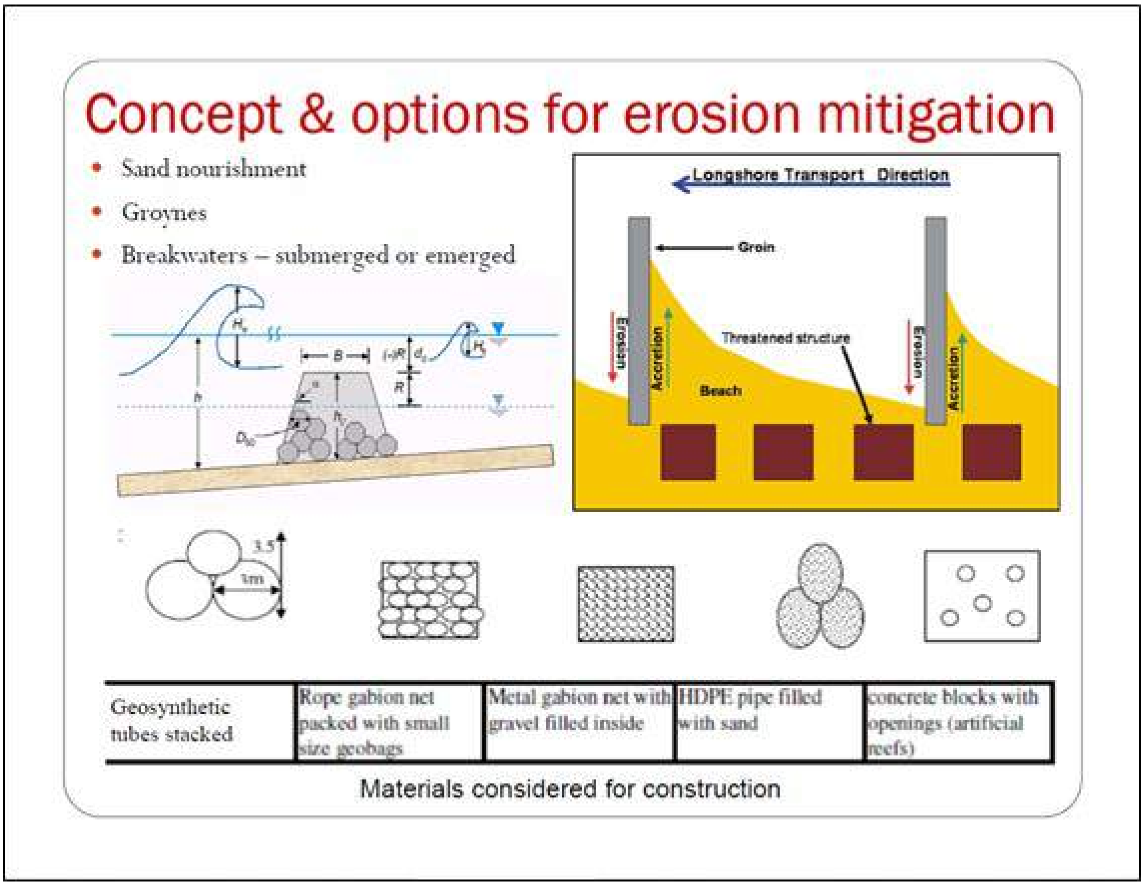
Current Status

- The installation of the sea wall embankment was completed on 10th June, 2016.
- Being a pivotal project, it was featured in the Intergovernmental Panel on Climate Change (IPCC) as India's initiative on the environmental change.

Prof.R.Sundaravadivelu, FNAE,
Department of Ocean Engineering, Indian Institute of Technology, Madras.
08 December 2020

7.2.5 Design guidelines for geotextile applications underwater by Dr. Vijaya Ravichandran, NIOT

The presentation started with a brief introduction about 'Shoreline erosion'. A short explanation about the concept and options for erosion mitigation, to stop the high-Intensity waves from hitting the shores was presented. The site, PALAR RIVER SHORELINE and its characteristics were explained. Satellite imagery analysis of the shoreline behaviour, showing erosion of 10 meters in the years 1990-2013 was depicted. The solution for Segmented Submerged Breakwater was proposed along with the construction aspects and features for local fishermen. The aspects of the plan in reference to the **Haldia** project were discussed in detail. Geosynthetic tubes were used for the project, as they provide options for changes in the future. The design criteria with parameters such as Pumping pressure and lateral earth pressures were discussed for the Geosynthetic tubes. After briefly explaining all design optimizations, deployment techniques that can be used were explained.



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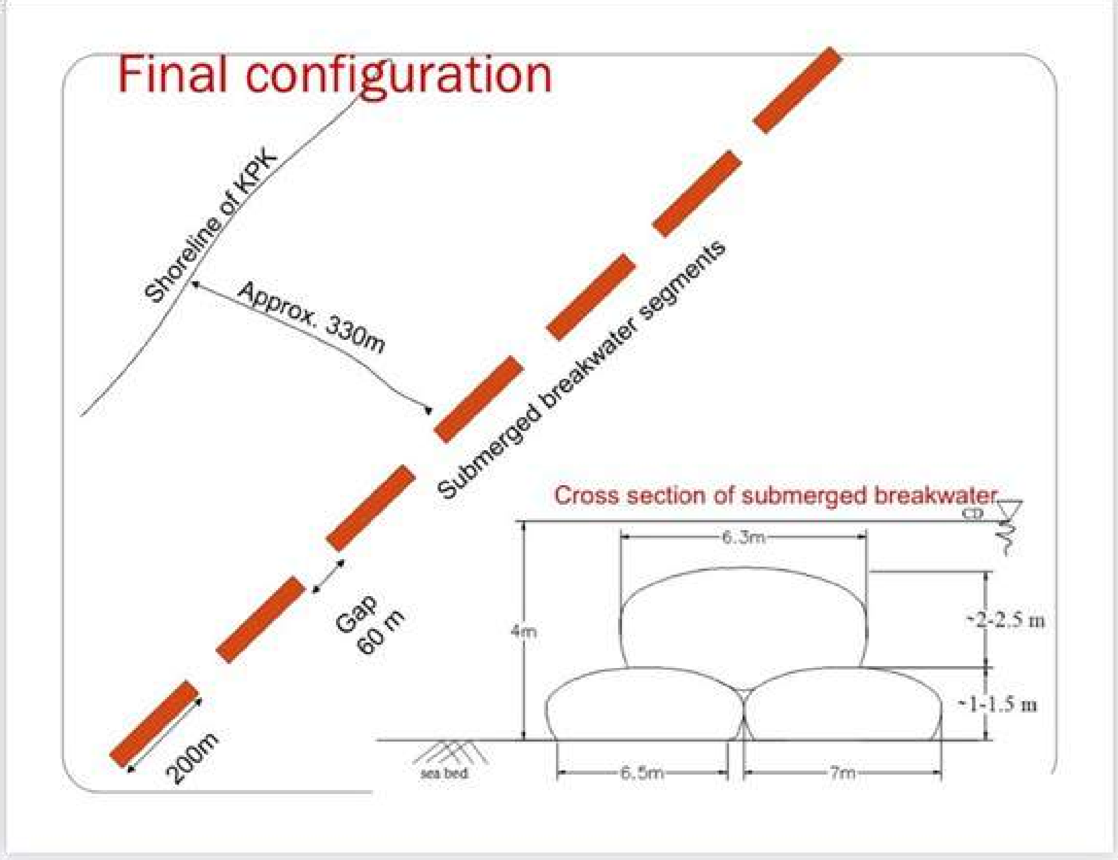
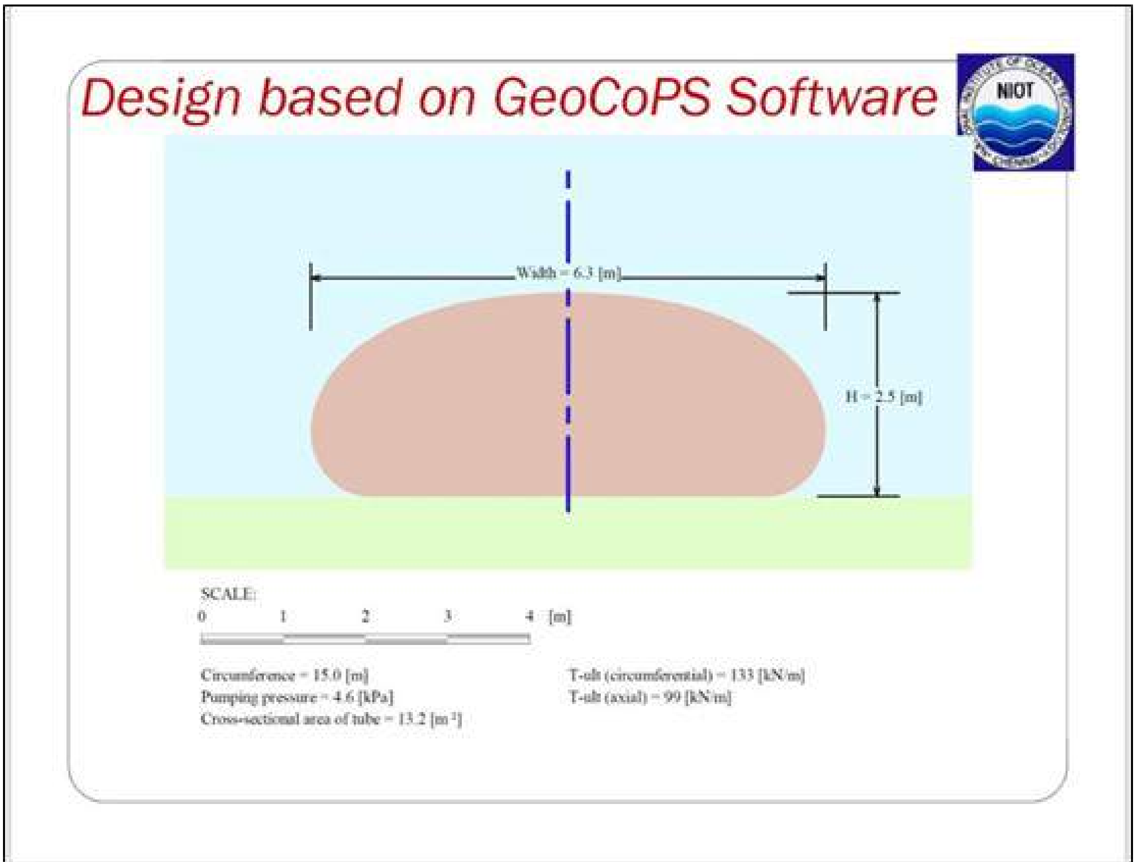

Final configuration
Shoreline of KPK
Approx. 330m
Submerged breakwater segments
Gap 60 m
200m
Cross section of submerged breakwater
CD
6.3m
4m
~2-2.5 m
~1-1.5 m
sea bed
6.5m
7m


Design based on GeoCoPS Software
NIOT
Width = 6.3 [m]
H = 2.5 [m]
SCALE:
0 1 2 3 4 [m]
Circumference = 15.0 [m]
Pumping pressure = 4.6 [kPa]
Cross-sectional area of tube = 13.2 [m²]
T-ult (circumferential) = 133 [kN/m]
T-ult (axial) = 99 [kN/m]

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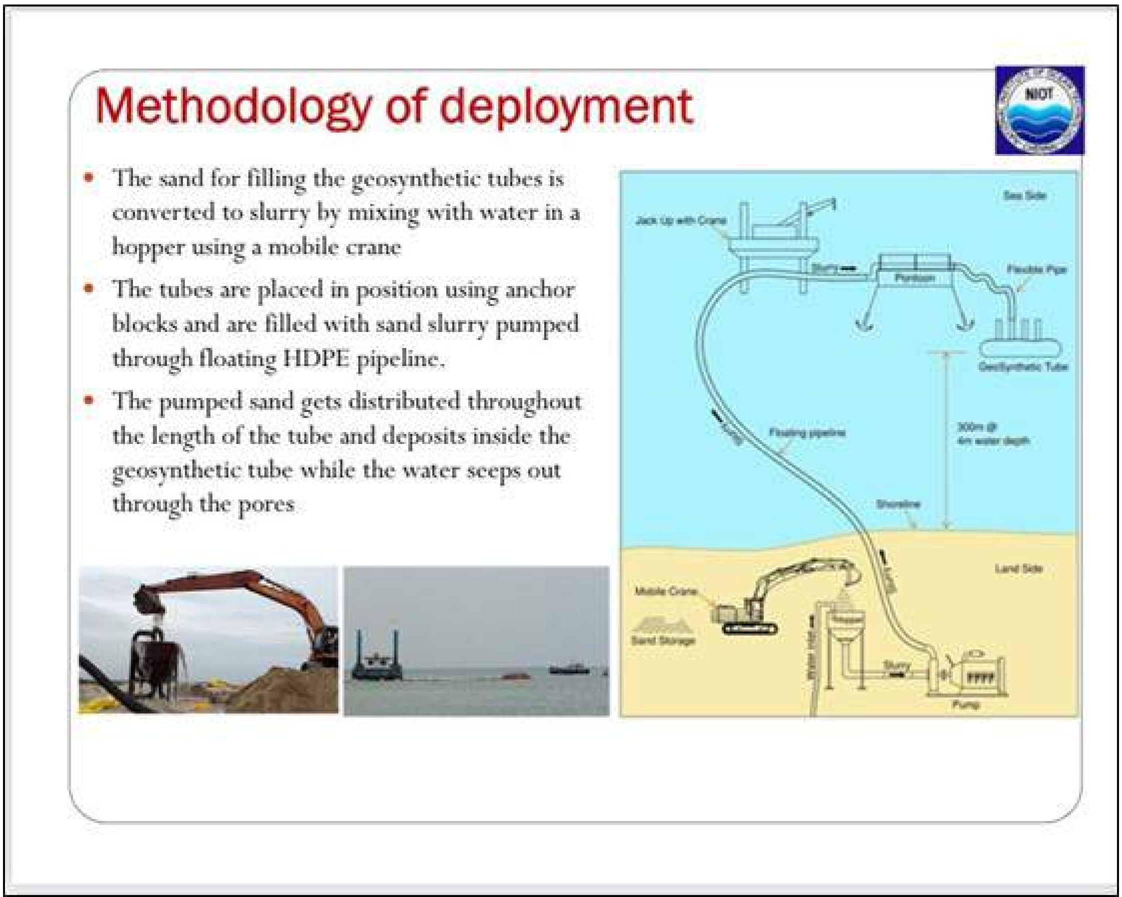

7.2.6 Geosynthetics for ground improvement by Prof. K. Ilamparuthi Anna Univ (Retd)

Prof. K. Ilamparuthi Anna Univ (Retd) gave a presentation on guidelines of Geosynthetics for ground improvement. Prof. K. Ilamparuthi discussed the need for ground improvement, ground improvement options, ground improvement using vertical drains and ground improvement using stone columns. The concept of band drains and embankment construction on band drains stabilized ground were explained. Guidelines for band drains, their scope, the material used, properties and specifications were given. Tests on core and sleeve (jacket) as per IS standard or ASTM/ISO were explained. Specifications as per section 704 of MORTH (Ministry of Road Transport & Highways) were provided. Band drains design concept was explained with the equations. The installation procedure and specifications were also explained in detail. Finally, Guidelines for band drain installation were given. Advantages of stone column, the mechanism by which stone column behaves as a short or long column, design parameters of stone columns, the capacity of a stone column as per IS 15284 PART 1 and column material specification as per IS 15284 (part 1 & 2) and MOST were presented.

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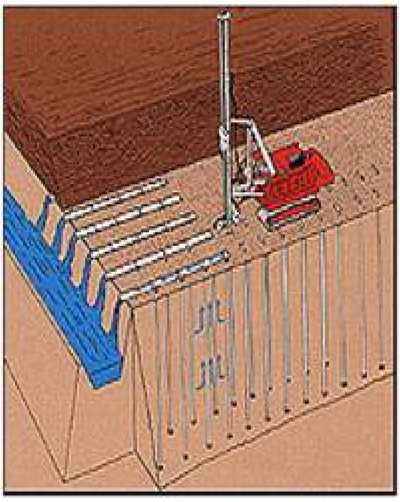
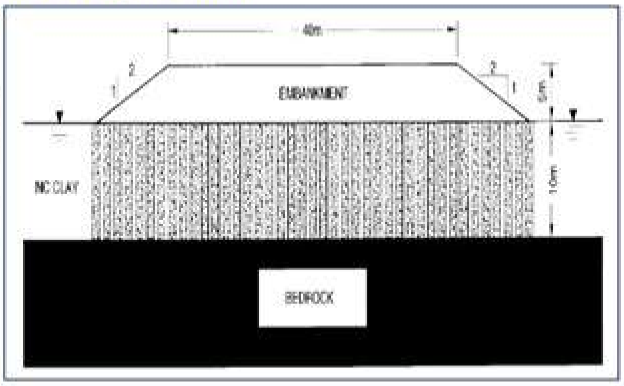
GUIDELINES ON GEOSYNTHETICS FOR COASTAL PROTECTION AND PORT WORKS

Ground Improvement



3D View of Band Drain Stabilised Bed



Embankment on Stone column stabilized bed

By
Dr. K.ILAMPARUTHI
Professor & CHAIRMAN (Retd.), Faculty of Civil Engineering
College of Engineering Guindy, Anna University.

BAND DRAINS - GUIDELINES

BAND DRAINS – Properties & Specifications

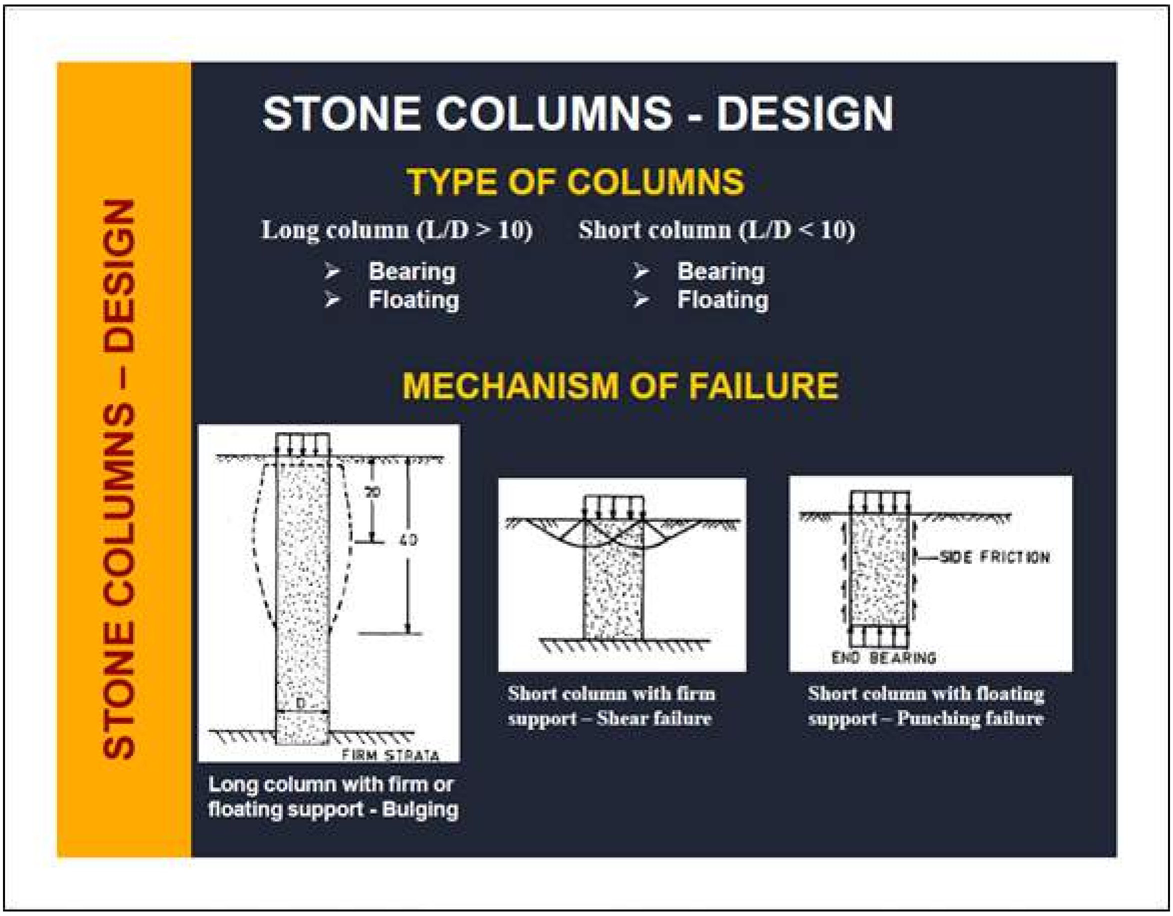
Properties:

i. Discharge capacity of composite drain
ii. Filter properties of jacket / sleeve (AOS)
Permeability and Resistance to clogging
iii. Tensile strength of composite drain (Core and filter).

TESTS ON CORE & SLEEVE (Jacket)
(As per IS standards or ASTM / ISO)
Compatibility and Quality control

SPECIFICATIONS AS PER SECTION 704 OF MORTH

S. No	Property		Test Method	Value
A	Composite Drain			
1)	Width			≥100 mm
2)	Thickness		ASTM D5199	≥4 mm
3)	Tensile strength		ASTM D4595	>2.00 kN
4)	Elongation at break			~36%
5)	Discharge capacity	i = 1.0 at, 300 kPa pressure	ASTM D4716	>1.5 x 10[illegible] m^3/s



7.2.7 Design of bunds for river bank protection by Prof. Nilanjan Saha, IITM

Prof. Nilanjan Saha, IITM has presented a Case study on Gabion mattress revetment behind Jetty at Hazira port.

- Reliance Industries Limited (RIL) commissioned their Hazira Manufacturing Division (HMD) on the banks of river Tapi.
- Due to tidal variations and floods during the monsoon, around 940m has been observed to be under constant erosion.
- A necessary and suitable shore protection is to be adopted to prevent erosion and restrict ingress of floodwater into HMD plant area.
- The novel construction methodology (as per site requirements) using gabion mattress revetments was proposed.
- Challenge was to execute the revetment behind two operational jetties

Site layout and causes for failure of existing shore protection works were projected with pictures from the site for clear understanding. Meteorological data, geotechnical data and Bathymetric survey carried out were explained. The revetment thickness considered; Scour

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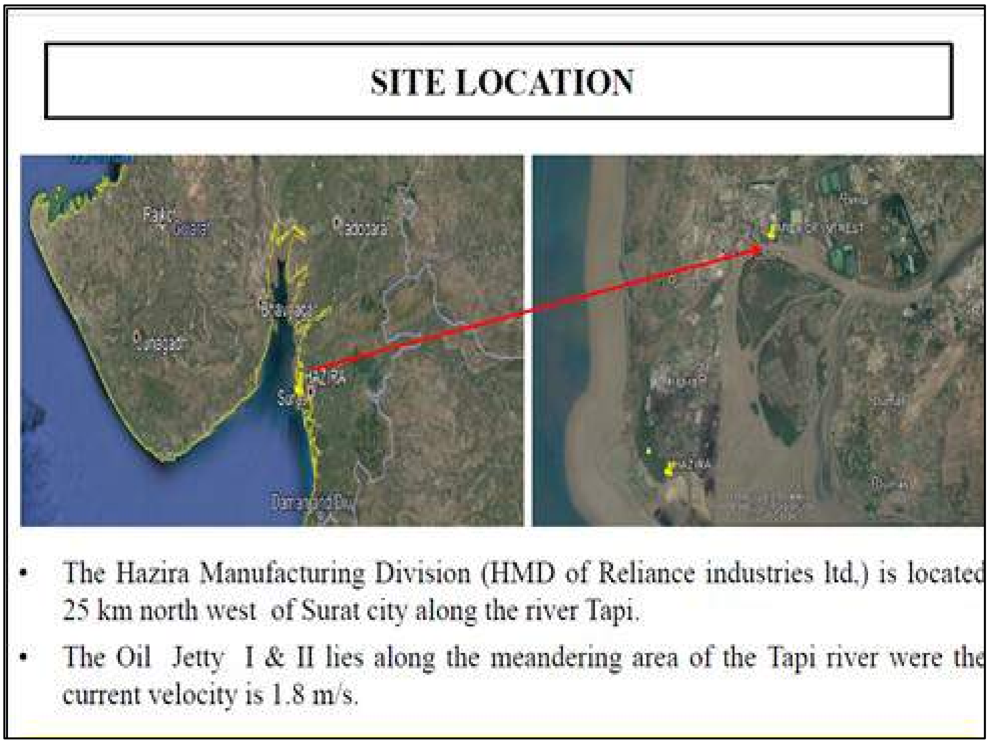
depth arrived using Lacey's formula considering the discharge capacity of Tapi river were explained. The methodology adopted in the reclamation process was as follows:

1. Preparation of exiting sloping ground and debris.
2. Filling of locally sources soil
3. Spreading of Geo textile and ground anchoring
4. Stacking of Gabions
5. Zinc and PVC coated gabion box for above water application and PP gabion box for below water application were installed.

The following conclusions were arrived:

- The traditional way of bank protection, methods and materials were ineffective over this particular site.
- Site Specific remedial measure was proposed and constructed for the erosion prone zone.
- The river bund was reinforced with geo-textile and protected and gabion protection to arrest the erosion due to river discharge.
- The proposed and designed cross-section was analyzed numerically for slope stability using PLAXIS 2D.
- The slope stability analysis for designed revetment indicates the factor of safety is more than 1.5 against the minimum factor of safety of 1.4.



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CAUSES FOR FAILURE OF EXISTING SHORE PROTECTION WORKS

- Non engineered shore protection was carried out using construction debris over in irregular slope.
- The shore is continuously exposed to tidal variation and storm surge.
- The Top surface of failed embankment was constructed using locally available soil which is silty clay.
- The traditional way of embankment protection is in-effective due to the drain cuts and river flow.
- No proper road access due to presence of LPG pipelines and hence periodic maintenance could not be done.

Problems at site

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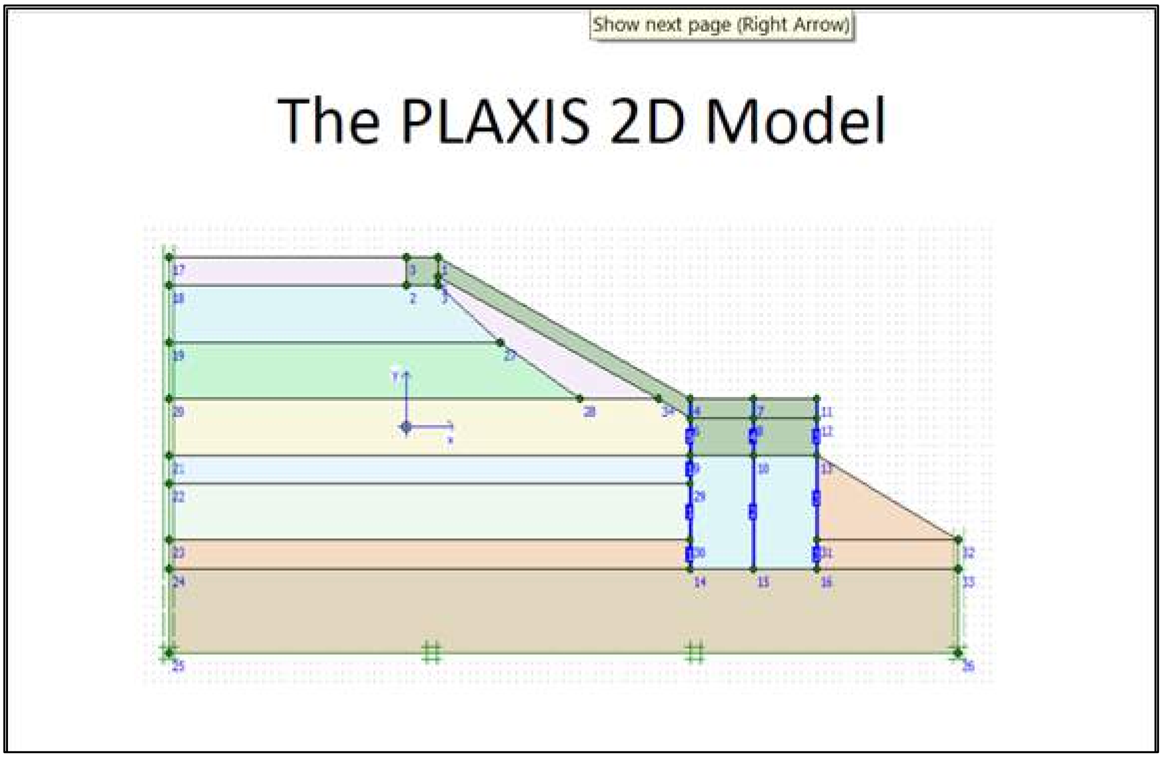

7.2.8 Geosynthetics in Pavements of port areas by Prof. G. L. Sivakumar Babu, IISC

Prof G L Sivakumar Babu presented on Pavement structures. The presentation started with an introduction to Flexible pavements and its components. Later, the types of pavement failures and its reasons such as damage due to drainage and confinement, Fatigue cracking principles, rutting mechanism for asphalt Layer were discussed in detail. Application of Geotextiles and Geogrids and its effect in pavement durability were explained briefly. The material properties of sand subgrade, base aggregates, and methods of testing along with results were discussed elaborately. Effect of Geo grids in pavement structures and various studies conducted to evaluate the use of Geogrids such as permanent deformation studies, pressure distribution studies, pressure distribution in base layer were compiled and explained in detail. The field studies and aspects of the roads were discussed. The procedure of incorporating the Geogrids and Geotextiles in existing as well as new roads were depicted using pictures. Various test results and design charts were explained for the process. Pavement drainage design was given with factors such as risk of permeable layer, and assessment taken for Indian condition considered. The Presentation was summarized with the following points:

- From the studies it was observed that the geosynthetic reinforcement in the unbound granular layers effectively reduces the permanent deformation compared to the unreinforced section.
- For the same thickness, it was noted that the permanent deformation per cycle was much higher for unreinforced section compared to the reinforced section.
- The resilient deformation for the reinforced section was much lower when compared to the resilient deformation of the unreinforced section.
- Field studies clearly showed the effectiveness of geosynthetics in pavements.

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Pavement failures

Damages due to lack of drainage and confinement

There is a need for proper design and construction methods in this connection to address the above factors

Use of geosynthetics results in significant savings, improved performance and very good serviceability in both short term and long term
Geosynthetics have made it possible to construct roads and pavements in seemingly difficult situations such as marshy stretches, soft and organic deposits and in expansive soil areas

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Field Studies

PROJECT SITE

Materials And Methods

- Experiments were carried out on unreinforced, and road mesh aggregate systems to evaluate their performance under static and repeated loading conditions.
- The materials that were used in the plate load tests to represent the subgrade and base layer of pavements were:
  - Subgrade Soil - sand
  - Base layer - Aggregate

Plate load test setup with data acquisition system

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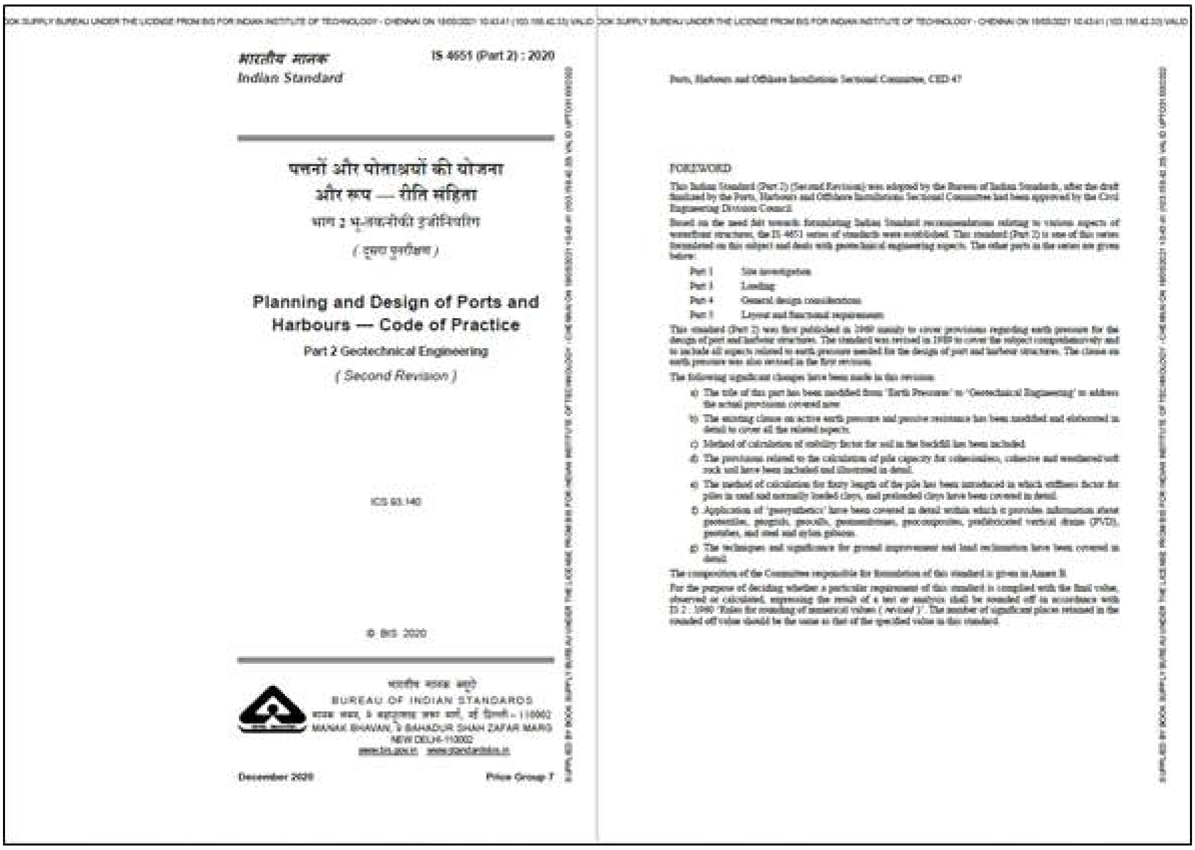
7.2.9 Geosynthetics Part 2 of IS 4651 and Slope stability analysis and design of Posonallah guide bund along Dibang river, Arunachal Pradesh by Prof. R. Sundaravadivelu, IITM

Prof. R. Sundaravadivelu, IITM, discussed bout the IS code 4651 part 2: Ports and Harbours – Planning and Design - Code of practice – Part 2 Geotechnical Engineering. The entire content present in the IS code was explained and discussed in the forum. The participants were requested to send Case studies if available with them to include in the Draft Guidelines present. The draft guideline was presented to the delegates and individual chapters were discussed.



भारतीय मानक
Indian Standard

IS 4651 (Part 2) : 2020

पत्तनों और पोताश्रयों की योजना और रूप — रीति संहिता

भाग 2 भू-तकनीकी इंजीनियरिंग

(दूसरा पुनरीक्षण)

Planning and Design of Ports and Harbours — Code of Practice

Part 2 Geotechnical Engineering

(Second Revision)

ICS 93.140

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भारतीय मानक ब्यूरो
BUREAU OF INDIAN STANDARDS
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NEW DELHI-110002
www.bis.gov.in www.standardsbis.in

December 2020 Price Group 7

Ports, Harbours and Offshore Installations Sectional Committee, CED 47

FOREWORD

This Indian Standard (Part 2) (Second Revision) was adopted by the Bureau of Indian Standards, after the draft finalized by the Ports, Harbours and Offshore Installations Sectional Committee had been approved by the Civil Engineering Division Council.

Based on the need felt towards formulating Indian Standard recommendations relating to various aspects of waterfront structures, the IS 4651 series of standards were established. This standard (Part 2) is one of this series formulated on this subject and deals with geotechnical engineering aspects. The other parts in the series are given below:

Part 1 Site investigation
Part 3 Loading
Part 4 General design considerations
Part 5 Layout and functional requirements

This standard (Part 2) was first published in 1969 mainly to cover provisions regarding earth pressure for the design of port and harbour structures. The standard was revised in 1989 to cover the subject comprehensively and to include all aspects related to earth pressure needed for the design of port and harbour structures. The clause on earth pressure was also revised in the first revision.

The following significant changes have been made in this revision:

a) The title of this part has been modified from 'Earth Pressure' to 'Geotechnical Engineering' to address the actual provisions covered now.
b) The existing clause on active earth pressure and passive resistance has been modified and elaborated in detail to cover all the related aspects.
c) Method of calculation of stability factor for soil in the backfill has been included.
d) The provisions related to the calculation of pile capacity for cohesionless, cohesive and weathered soft rock soil have been included and illustrated in detail.
e) The method of calculation for fixity length of the pile has been introduced in which stiffness factor for piles in sand and normally loaded clays, and preloaded clays have been covered in detail.
f) Application of 'geosynthetics' have been covered in detail within which it provides information about geotextiles, geogrids, geocells, geomembranes, geocomposites, prefabricated vertical drains (PVD), geotubes, and steel and nylon gabions.
g) The techniques and significance for ground improvement and land reclamation have been covered in detail.

The composition of the Committee responsible for formulation of this standard is given in Annex B.

For the purpose of deciding whether a particular requirement of this standard is complied with the final value, observed or calculated, expressing the result of a test or analysis shall be rounded off in accordance with IS 2 : 1960 'Rules for rounding off numerical values (revised)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

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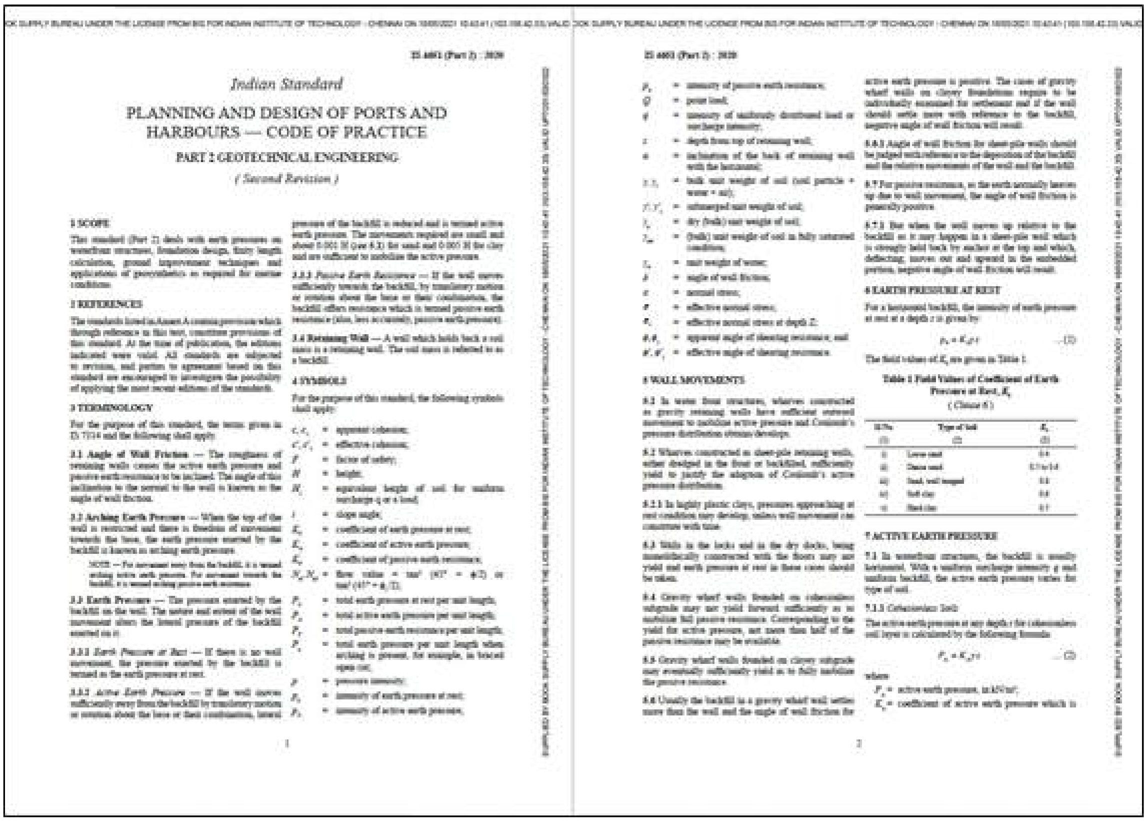


Indian Standard

PLANNING AND DESIGN OF PORTS AND HARBOURS — CODE OF PRACTICE

PART 2 GEOTECHNICAL ENGINEERING

(Second Revision)

1 SCOPE

2 REFERENCES

3 TERMINOLOGY

4 SYMBOLS

5 WALL MOVEMENTS

6 EARTH PRESSURE AT REST

7 ACTIVE EARTH PRESSURE

Prof. R. Sundaravadivelu, IITM presented a case study on Slope stability analysis and design of Posonallah guide bund along Dibang river, Arunachal Pradesh. The scope of work, was to reclaim a damaged guide bund along a stretch of Dibang river. Review of Hydrological study and Satellite Data Analysis were presented. The design data consisted of Design discharge, design high flood level, design velocity and geotechnical data, and general arrangement of the site were presented with typical cross section and plan view for better understanding. Design of guide bund with gabion boxes was explained consisting of revetment thickness, scour depth calculation for HFL (High Flood Level) and LWL (Low Water Level), and the launching apron for protection of toe on river side, . Slope stability analysis was carried out by using PLAXIS 2D model analysis and the all the properties that are arrived for giving input to the model was explained in detail. A detailed cost estimate was presented. IITM recommendation for Repelling spur, design of repelling spur and the proposed detailed plan and cross section of the repelling spur adopted was given

Scope of Work

Phase-I (Preparation of Preliminary Project Report)

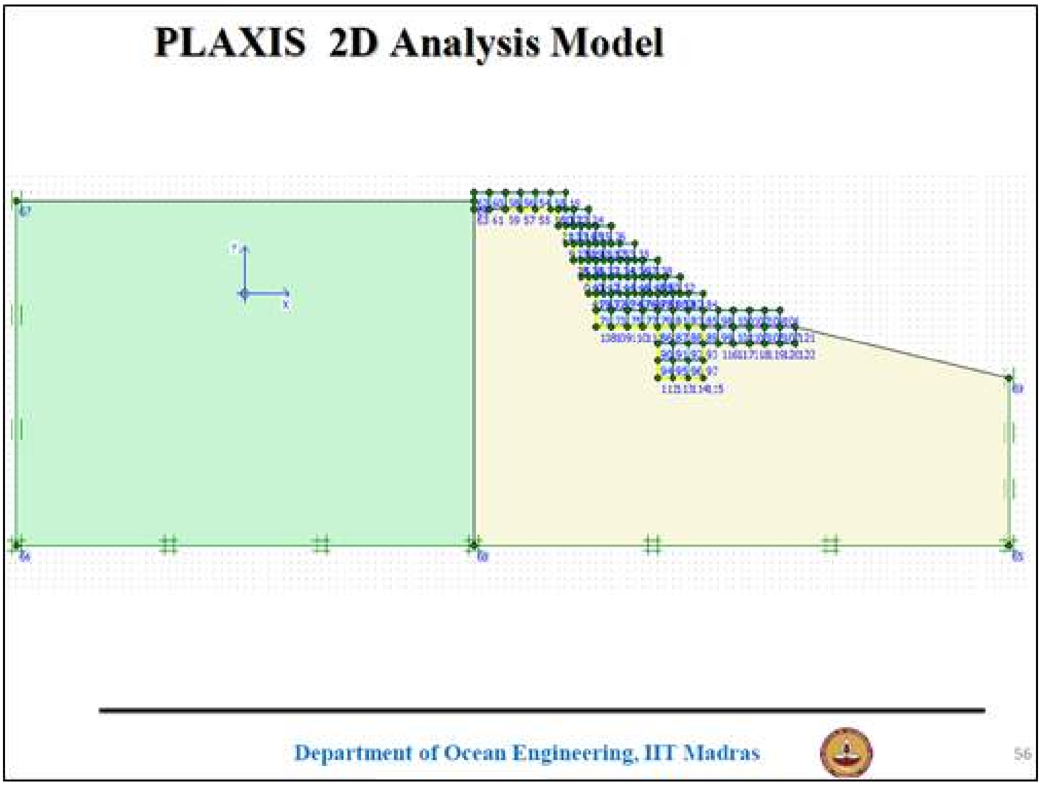
i. To carry out review of desktop studies on hydrology of the Dibang river.
ii. To provide a suitable design and shore protection methodology using gabion boxes for construction of the guide bund.
iii. To perform numerical analysis using PLAXIS 2D for the proposed shore protection methodology and check stability against sliding for the proposed model.
iv. Submission of preliminary project report including the details of the analysis and shore protection methodology.

Phase-II (Preparation of Detailed Project Report)

i. The detailed engineering of the shore protection methodology selected in the PPR stage.
ii. List of drawings and list of designs.
iii. Methodology of construction and its sequence.
iv. Submission of Detailed Project Report including Good for Construction Design and Drawing, Bill of Quantity and Cost Estimate.

Department of Ocean Engineering, IIT Madras 4

Proposed Location of Guide Bund



7.2.10 Coastal erosion and role of geosynthetics by Prof. S. A. Sannasiraj IIT M

Prof. S. A. Sannasiraj, IIT M delivered a lecture on Coastal erosion and role of geosynthetics. The presentation was in two parts, the first part was on the necessity of using geosynthetics in coastal environment. The second part was on the issues that is to be given importance – based on the design aspect and scour aspect. In the coastal erosion part, apart from rubble stones or masonry/concrete blocks of different shape, it was highlighted that gabions, geosynthetic bags, containers, geotubes are being used now a days. Case study on protection work at Paduthomse Rasid, Kartanaka was explained in detail. Another case study presented in the session was about Lakshadweep Islands seawall which was constructed using tetrapods and concrete hollow blocks and mortar filled coir bags. Detailed presentation of those construction with pictures from the site was given. Geosynthetics for hydraulic structures are used since 1980s and its advantages were explained. Design principles for wave energy dissipation and filtration along with technical specifications were given. Here Physical Stability Criteria in terms of External stability and Internal Stability were given priority. Design of sections in the Marine Environment and their governing factors in terms of stability and settlements were addressed. Necessity of toe protection, types of toe protection, design aspects of toe layer with case studies from west Bengal were discussed. Installation of geotubes and problem associated with it along with their remedial measures were highlighted.

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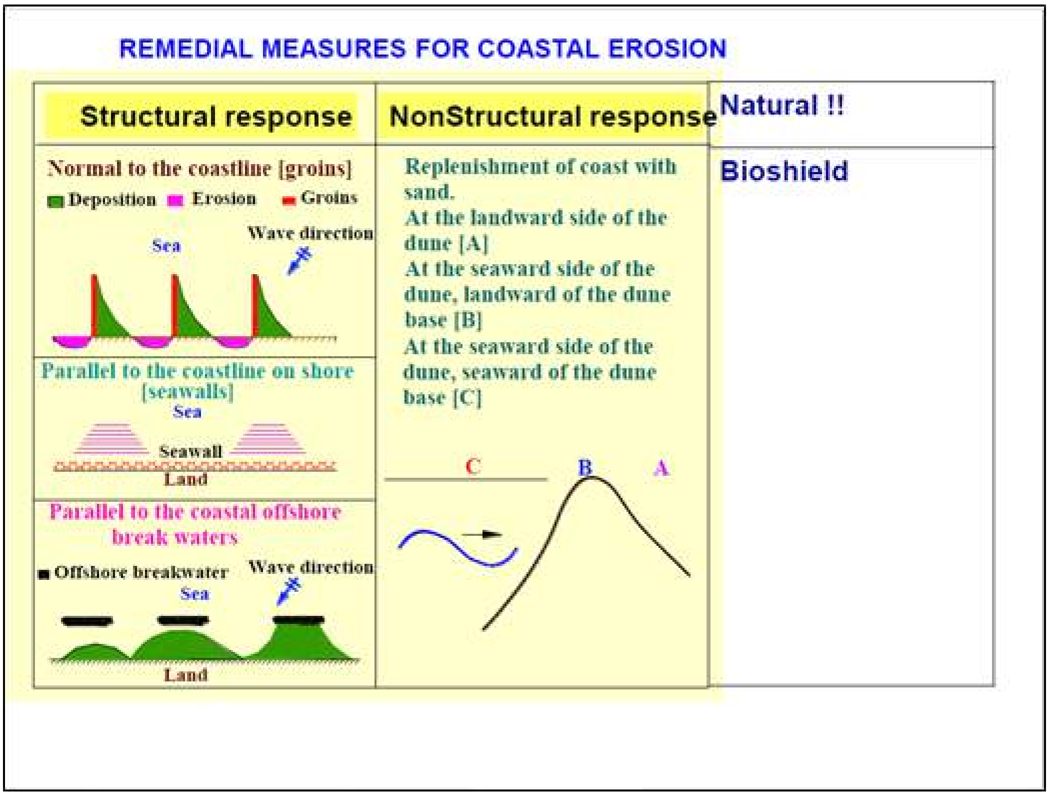
REMEDIAL MEASURES FOR COASTAL EROSION
Structural response
NonStructural response
Natural !!
Normal to the coastline [groins]
Deposition
Erosion
Groins
Wave direction
Sea
Parallel to the coastline on shore [seawalls]
Sea
Seawall
Land
Parallel to the coastal offshore break waters
Offshore breakwater
Wave direction
Sea
Land
Replenishment of coast with sand.
At the landward side of the dune [A]
At the seaward side of the dune, landward of the dune base [B]
At the seaward side of the dune, seaward of the dune base [C]
C
B
A
Bioshield


PROTECTION WORK AT PADUTHOMSE RASID, KARNATAKA
Paduthomse-Rasid

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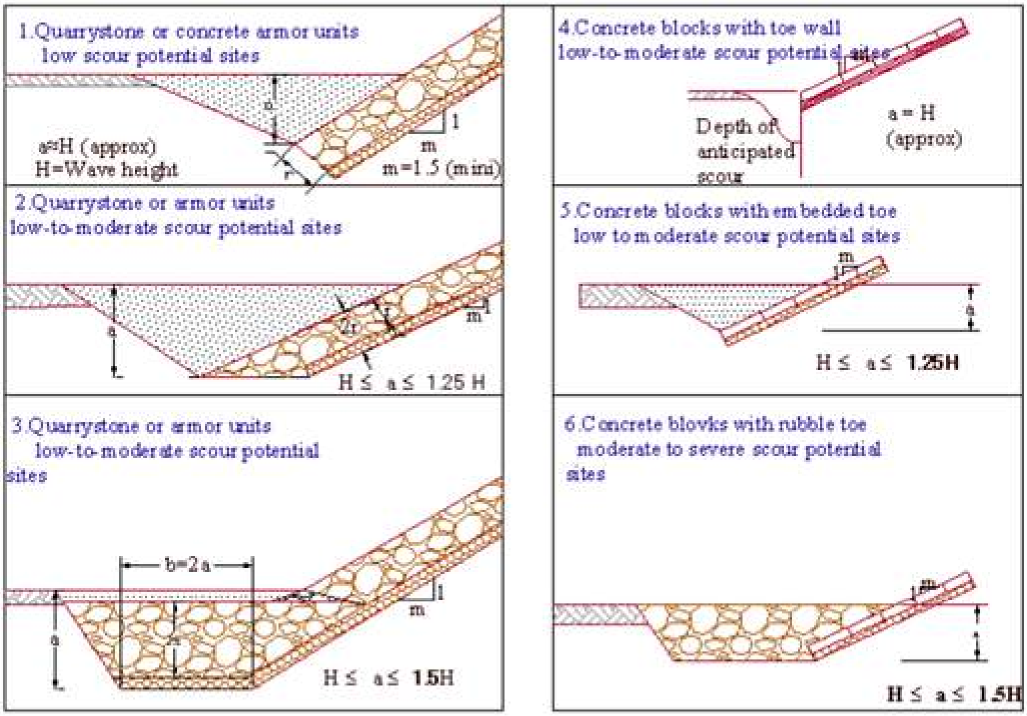
Toe Protection



WEST BENGAL
Shankarpur to Dhiga

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7.3 Annexure – Q & A

Mr.Krishnamoorthy, Chief Engineer & Administrator, Andaman Lakshadweep Harbour Works, Port Blair, gave the following inputs for incorporation into the Guidelines.

1. In Page No.9 it is not clearly mentioned about the usage of Geo-textiles in breakwater, however the same has been allowed in offshore breakwater.

2. There is a general tendency to replace armor units of breakwater with the same armor unit in case of any damages to the Breakwater structure. The provisions with Geo synthetic materials for rehabilitation & replacement of existing armour units of breakwater may also be included.

3. While to undertaking Tsunami, rehabilitation works at Little Andaman breakwater it has been tried to keeping more Nos. of tetrapods in gabion box and it was quite successful in temporary restoration. The stability aspects of such restoration work may be included in the guidelines.

4. In page No. 36 re-filled gabion box placed in water and the foundation of the gabion wall is not elaborated.

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5. It has been made a small building unit of 3 x 3 meters size by using plastic pet bottles filled with waste materials such plastic bottles were replaced holo-blocks like construction materials. Can such efforts be made using Geo synthetic materials while filling core of breakwater or shore protection works and any other such works is below 00 water level?

The following discussions were made after the presentation:

1. Polymer wire gabions are normally used in the coastal region. Can we use steel wire gabion in coastal areas?
   While using steel wire gabions, corrosion is the major issue. Also, the Flexibility problem is there with steel gabions. There will be uneven settlement while using steel wire gabions and polymer gabion can take care of that without any problem

2. Which type of material (woven geotextile) is used for Geo tube used in the case study by Dr.Vijaya? tape by tape or multifilament?
   Multifilament polypropylene was used. Also, multi-element woven Geotextile was used in Cuddalore - Periyakuppam project. In Haldia, a polyester woven Geosynthetic tube was used.
   Comment from the audience: Multifilament fabric to be considered for various uses can be provided with specifications in this draft.

3. What is the factor of safety for scour to be used in the coastal environment?
   Already 1.5 scour depth is considered for the coastal environment. The same can be used.
4. In the breakwater chapter, whether we can use geotextile? Can the existing armor unit be replaced using geotextile?
   The possibility of arriving at the Stability number for the Gabion wall filled with stones and the gabion wall filled with tetrapod will be looked into.

5. Instead of bricks, whether waste materials can be used for the construction in coastal areas?
   Geobag as a core material for construction is used in many places like Vyanjanam rani ports. Also, Dr.Vijaya from NIOT used Geo-bags for construction purpose.

6. In Gabions filled with stone as a core layer, three layers in the bottom, with 2 layers above it and one 1 layer at the top is normally used. Can this arrangement be replaced with a single layer?

   The main issue while using a single layer is permeability.

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Geo-synthetics in Coastal Protection, Ports and Waterways

K Murali

Professor

Department of Ocean Engineering

IIT Madras

Background

- **Initiation from MoPSW in July 2018 – in dialogue with Min of Textiles.**

Actionable Points **Annexure-III**

- Mandating upto 20% of Geosynthetics in new constructions and modernization of jetties, port roads, container goods, terminal shunting yard, shanty goods etc 2018, 40% by 2020, 60% till 2022, 80% till 2023, 100% till 2025.
- Inclusion of Geosynthetics in "Wearing Coat and Appurtenances" and "Repair and Replacement of Wearing Coat" in port roads. Phased implementation with 20% in 2018, 40% till 2020, 60% till 2022, 80% till 2023, 100% till 2025.
- Mandating use of Geogrids and Geocells for erosion control and issuing notification to incorporate these products in DPRs and tenders for replacing stone pitching in new constructions and modernization of jetties, port roads, container goods, terminal shunting yards, etc.
- Finalise and release guidelines for using Geogrids and Geocells pavements in new constructions and modernization of jetties, port roads, container goods, terminal shunting yards, pavements etc .
- In DBFOT/PPP and other such projects, Geogrids and Geocells use in constructions and modernization of jetties, port roads, container goods, terminal shunting yard, shanty goods etc pavements should be approved by Independent Engineers "IEs"
- A notification from the Ministry of Shipping, Road Transport and Highways (Department of Shipping) to use geogrids and geocells for container yard construction and to minimize the use of concrete in such pavements to be issued.

Background

- Initiation from MoPSW in July 2018 – in dialogue with Min of Textiles.

Some Production Figures of Geosynthetics

. lo	Company Name	Production in Sq. M/Annum					Installed Capacity in Sq. M/Annum
		Geogrid (Uniaxial)	Woven Geo textiles (Polyester /Polypropylene)	Nonwoven Geo textiles	Bi-axial Geogrid	Total	
1.	Texfab India Ltd	71428571	123216000	48000000	6857142.9	249501714	299402057
2.	Strata Geo systems	1000000	-	-	-	1000000	1200000
3.	Maruti Rub Plast	71428571	-	-	-	71428571	85714285
4.	Flextuff	-	144000000	48000000	-	192000000	230400000
5.	Sravya Textiles Plast	-	7200000	-	-	7200000	8640000
6.	Technofab	-	7200000	-	-	7200000	8640000
7.	Hi-Tech Speciaility	-	2400000	-	-	2400000	2880000
8.	Kusumgar Corporates	-	600000	-	-	600000	720000
9.	Jeevan Nonwovens	-	-	12000000	-	12000000	14400000
10	Manas GeoTech	-	-	11520000	-	11520000	13824000
11	Parishudh	-	-	12000000	-	12000000	14400000
12	Maccaferri	-	-	-	3600000	3600000	4320000
		14,38,57,142	28,46,16,000	13,15,20,000	1,04,57,143	57,84,50,285	68,45,40,342

rce: BTRA, Mumbai

Background

- **Initiation from MoPSW in July 2018 – in dialogue with Min of Textiles.**
- **NTCPWC, IITM entrusted with the responsibility of preparation of the guidelines.**

Background

- Initiation from MoPSW in July 2018 – in dialogue with Min of Textiles.
- NTCPWC, IITM entrusted with the responsibility of preparation of the guidelines.
- Draft Guidelines by 24 Nov. 2018.

Draft Guidelines on Geo-synthetics for Coastal Protection and Port Works

Released on the Occasion of

NATIONAL CONFERENCE ON APPLICATION OF GEOSYNTHETICS IN POTRS, WATERWAYS AND COASTS IIT MADRAS, CHENNAI, INDIA

Date: 24.11.2018

NATIONAL TECHNOLOGY CENTRE FOR PORTS WATERWAYS AND COASTS NTCPWC

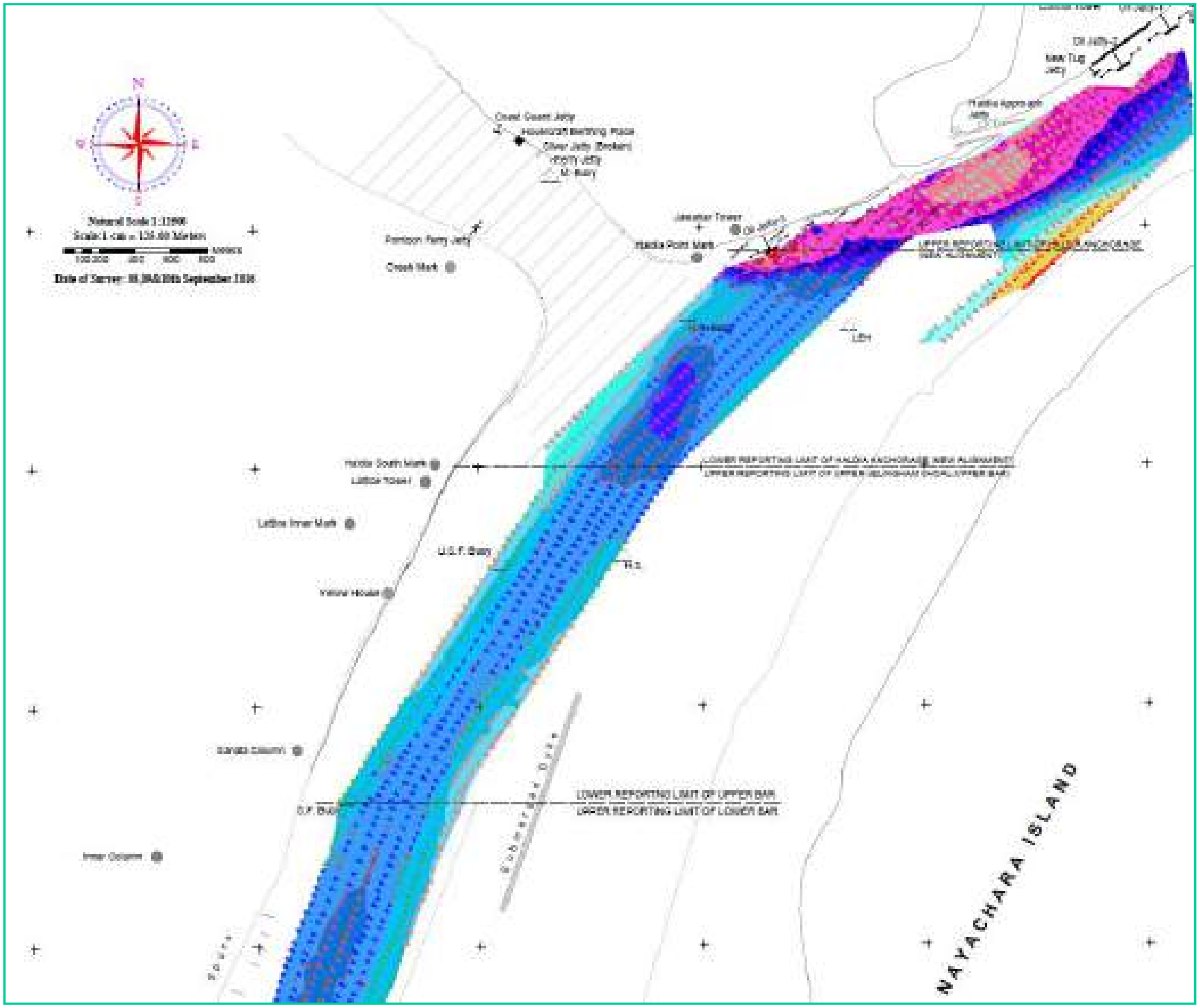
SAGARMALA

Department of Ocean Engineering, IIT Madras.

Major Application Areas

Products/ Applications	Geo-textiles	Geo-grids / Geonets	Geo-composites	Gabions	Geopipes	Geocells
Filtration	●		●			
Drainage	●		●		●	
Separation	●					
Reinforcement	●	●	●			●
Barrier	●					
Protection	●	●		●		●

Examples – SMP, Kolkata



Example – Farakka

Example – Farakka

Webinar on
Draft Guidelines on Geo-synthetics for Coastal Protection and Port Works
Case Studies on:
Coastal Erosion and Remedial Measures

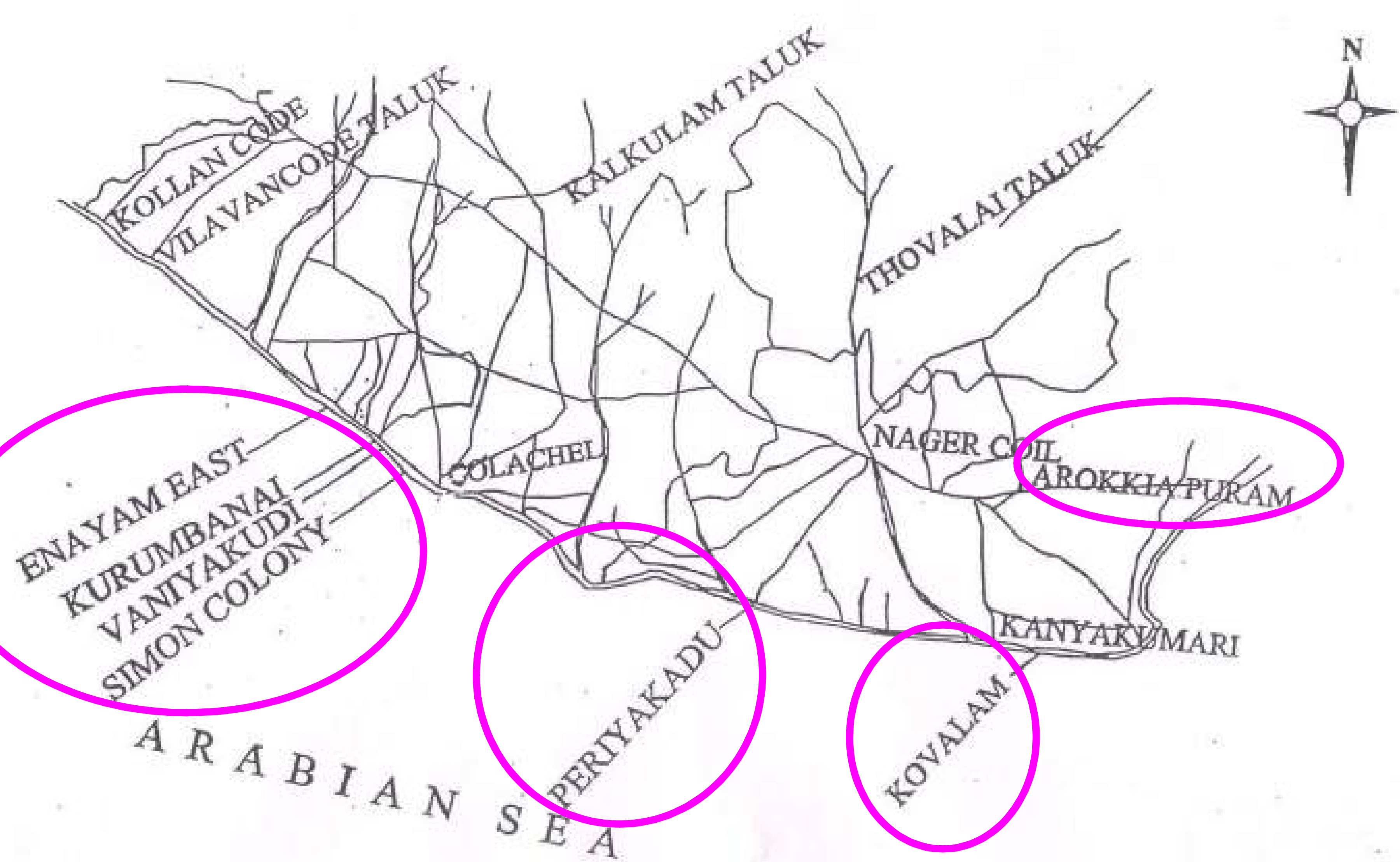
By

Prof. S. R. Gandhi

Director, SVNIT Surat

director@svnit.ac.in srgandhi@gmail.com

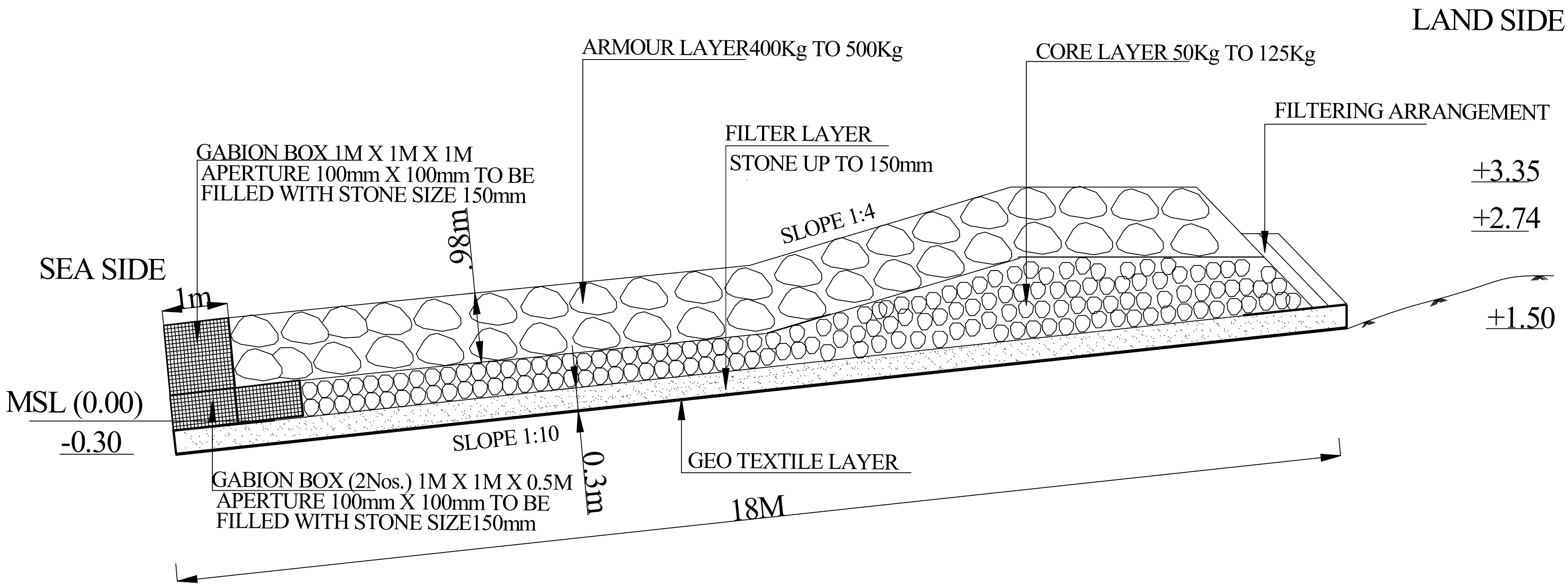
Prof. S.R.Gandhi, Director SVNIT Surat




GABION BOX FILLED WITH STONES
GEOTEXTILE LAYER

Gabion Box 1mx1mx1m Filled with Stones

Prof. S.R.Gandhi, Director SVNIT Surat

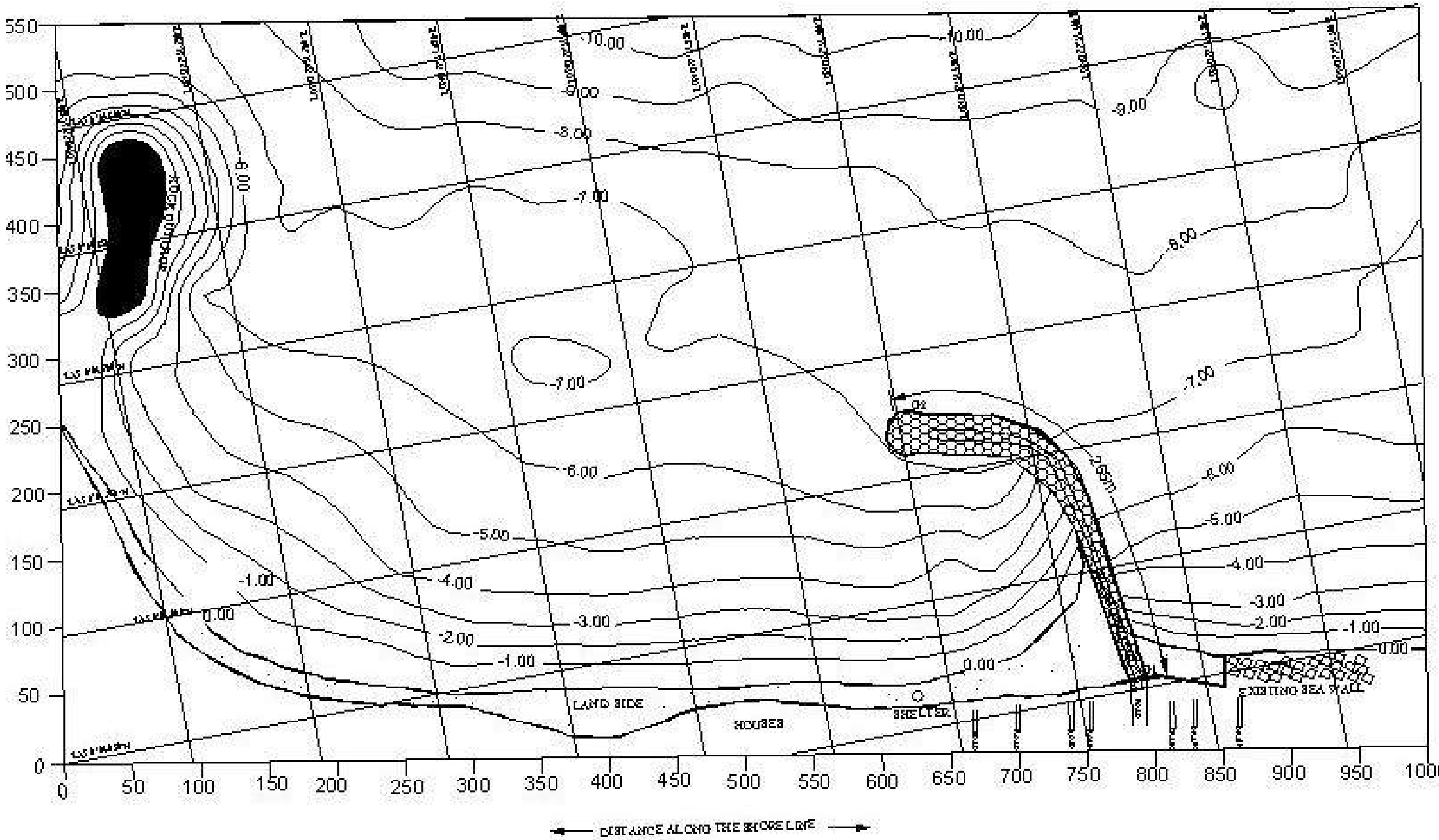


Typical cross section of rubble mound seawall

NATIONAL TECHNOLOGY CENTRE FOR PORTS WATERWAYS AND COASTS
NTCPWC

Webinar on Geo-synthetics for Coastal Protection and Port works
10th and 11th Dec 2020

Prof. S.R.Gandhi, Director SVNIT Surat



GROIN AT KOVALAM, KANYAKUMARI

Webinar on Geo-synthetics for Coastal Protection and Port works
10th and 11th Dec 2020

Groin at Kovalam

Groins at Arokiapuram

Prof. S.R.Gandhi, Director SVNIT Surat

Groin at Simon Colony

Groin at Vaniyakudi

Beach formation at Enayam after receding of Tsunami


Reinforced Wall at Mangalore Refinery

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Questions?

THANK YOU

Guidelines on Geosynthetics for Coastal Protection and Port Works

Introduction and Terminology

U.K.Guruvittal
Chief Scientist
CSIR - Central Road Research Institute
New Delhi – 110025

Coastal Management

Coastal zones constitute about 15% of land area but home to 45% of world's population - Economic and strategic importance of coastal areas, ports

Harbour and sea works – Docks, breakwaters, jetties, road and other infrastructure

Coastal Management – Defense against flooding and erosion, Techniques to check coastal erosion

Climate change hastening sea level rise

Protecting coastal areas from sea erosion – Structural and Non-structural measures

Ref: WIKIPEDIA

Structural Systems for Coastal Protection

Two parts (a) Outer revetment – Armour layer to absorb hydraulic energy (b) Inner filter layer

Revetment – Rip-rap blocks, prefabricated concrete elements, RCC/ stone masonry walls, etc

Filter layer – Prevent soil particles from being eroded, allow water to escape simultaneously

Filter – Well designed granular material in layers

Mr. Guru Vittel CRRI

CSIR CRRI

Systems for Coastal Protection

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10th and 11th Dec 2020

Geosynthetics

Synthetic products used in various geotechnical applications for **enhancing** the **serviceability / performance** of an engineering structure

Classification – Based on manufacturing process, materials used or function

- Geotextiles
- Geogrids
- Geonets
- Geoties
- Geomembranes
- Geocells, Geocomposite, Band drains, etc

Geosynthetics

Advantages – (a) Technical superiority with a variety of engineering products/ techniques (b) Faster construction (c) Flexibility of structures which perform well during earthquakes and (d) Better quality control of products

Functions – Reinforcement, stabilisation, Drainage, Separation, Filtration, Erosion Control, Moisture barrier

Raw material – Petro-based synthetic polymers such as polypropylene, polyester, polyvinyl chloride, polyamide, polyethylene, etc.

CSIR CRRI

Geotextiles

- Synthetic permeable textiles used with foundation, earth, rock, or any similar geotechnical engineering related materials as an integral part of a structure or infrastructure projects
- By suitably engineering geotextiles, they can be used both as revetment and as filter media
- Classification
  - Woven
  - Non woven
  - Knitted
  - Composite

Geogrids

Polymers formed into an open, grid configuration with apertures between individual ribs in transverse and longitudinal directions

Raw materials – Polypropylene (PP), Polyester (PET), High Density Polyethylene (HDPE), Glass

Classification

- Manufacturing process (woven, knitted, bonded, welded, extruded)
- Directional behaviour (uniaxial, biaxial, triaxial)
- bonding between ribs and
- Polymer used

Other types of Geosynthetics

Geostrips – Reinforcement strips made using high tenacity polyester yarn which are encased in suitable polymer sheath

Geomembranes – Low permeability/ impervious synthetic membrane liner / barrier used to control fluid migration across the plane

Geocomposites – Drainage Composites made by combining a core of geonet with a geotextile cover

Geocells – Three-dimensional honeycombed cellular confinement system, filled with compacted soil or aggregate

Other types of Geosynthetics

Geonets – Integrally connected, parallel sets of ribs formed by continuous extrusion into a netlike configuration for in plane drainage of liquids

Geomats – Two or three dimensional mats, formed using multi-filaments and geogrids folded and knitted/ bonded, having openings which allow vegetation growth to check erosion of soil slopes

Bituminous Pavement Reinforcement Products for strengthening of bituminous pavement layer – Paving Fabric, Glass-Fibre Grid, Composite

Suggestions for Geosynthetics – IS 4651

- Instead of geosynthetics being a part of IS 4651 – Part 3, Geosynthetics should be separate part
- Present draft deals with material specifications and brief construction aspects
- Preparation of a design guidelines
- Compendium on case studies

Mr. Guru Vittel CRRI

Webinar on Geo-synthetics for Coastal Protection and Port works
10th and 11th Dec 2020

TWO DAY WEBINAR ON
DRAFT GUIDELINES ON GEO-SYNTHETICS FOR COASTAL PROTECTION AND PORT WORKS
10.12.2020 & 11.12.2020

GEOSYNTHETICS PART 2 OF IS 4651 AND SEA WALL AT PENTHA

PRESENTED BY :

PROF R. SUNDARAVADIVELU

DEPARTMENT OF OCEAN ENGINEERING,

IIT MADRAS.

WEBINAR CONDUCTED BY

Dept. of Ocean Engineering,
IIT Madras.

NATIONAL TECHNOLOGY CENTRE FOR PORTS WATERWAYS AND COASTS
NTCPWC

Ships, Offshore, Coastal and Environmental Research Society

OUTLINE

- Introduction
- Literature Review
- Scope
- Objective
- Design of Geo-tube embankment
- Physical Model study on reflection and transmission of Geo-tube embankment with and without gabion
- Numerical Model on Geo-technical Stability for integrated and individual saline embankment
- Results & Discussion
- Conclusion
- Visible Outputs
- References

Introduction

➢Geotextile tubes are made of synthetic fibers which are sustainable, and permeable textile fibers that can contain, filter, and reinforce soil..

➢ Even though there may be a differential settlement, the geotextile tube will adjust with soil bed profile.

➢The Geo-tube embankment with infill material as coarse river sand will

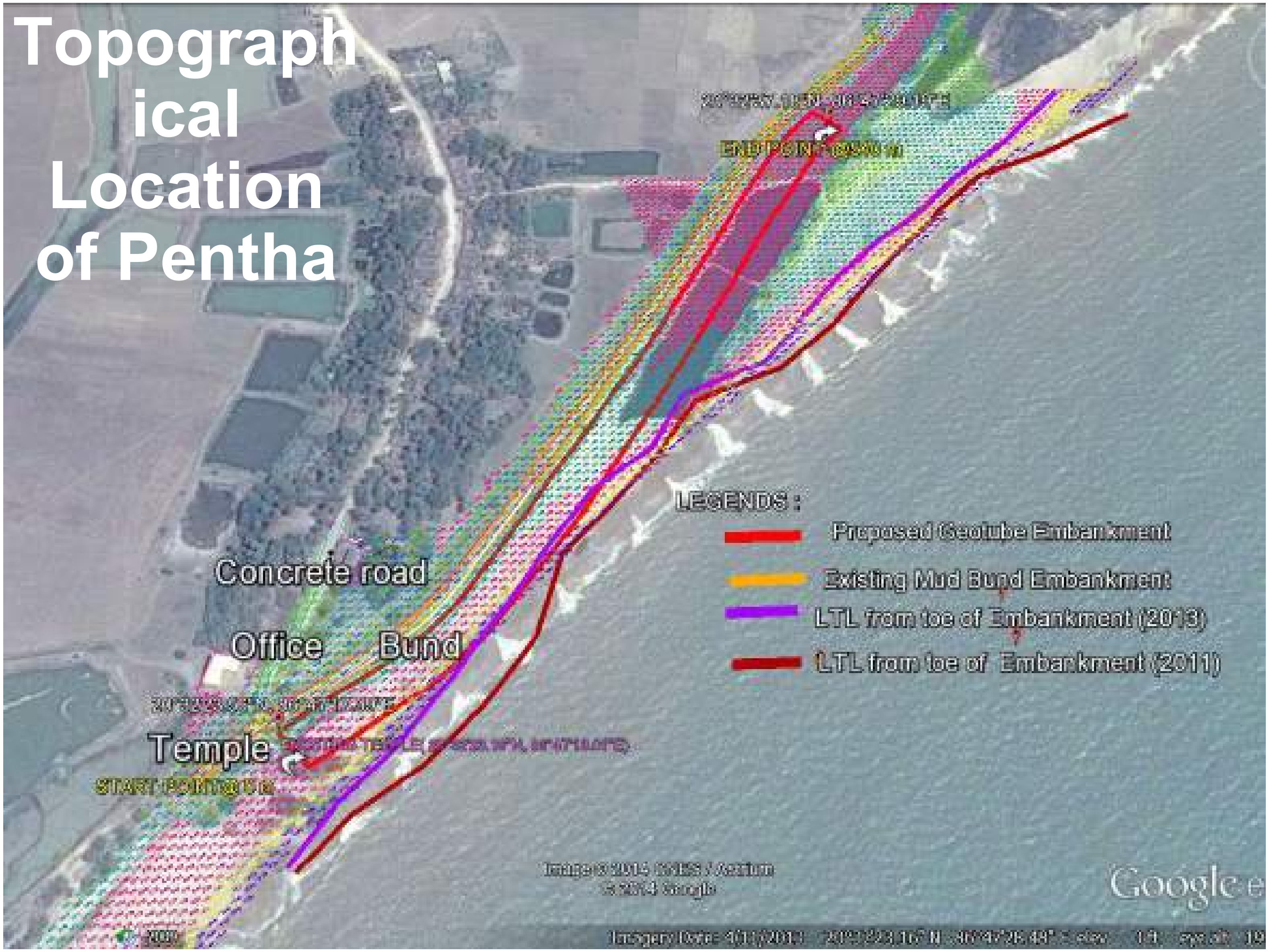
a. Allow free drainage

b. prevent the scouring action

c. Dissipate the wave energy

d. Development of pore water pressure will be avoided

e. Be used in soft soil

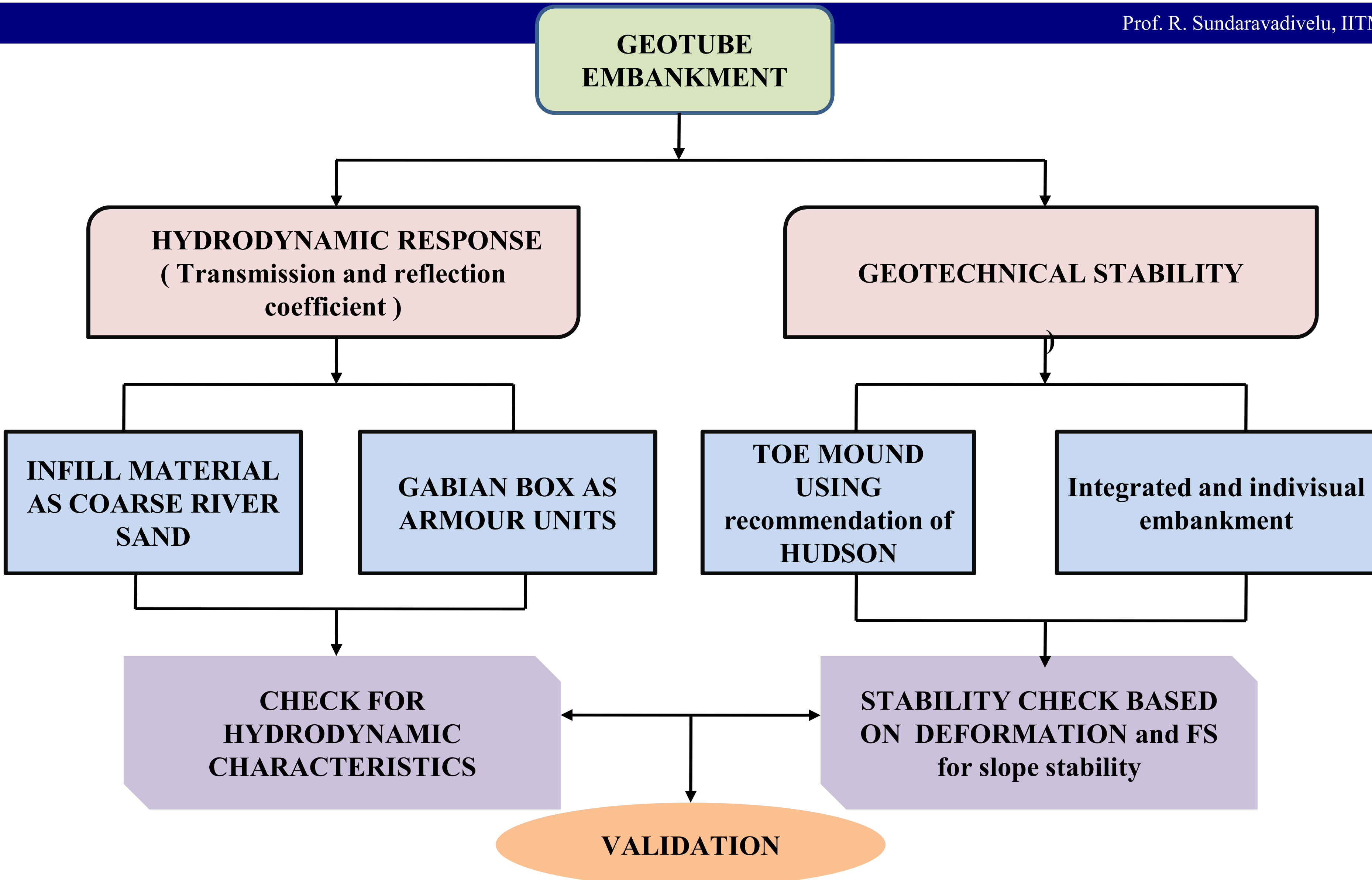


Topographical Location of Pentha
LEGENDS :
Proposed Geotube Embankment
Existing Mud Bund Embankment
LTL from toe of Embankment (2013)
LTL from toe of Embankment (2011)
Concrete road
Office
Bund
Temple
START POINT@0 m
Google earth

Causes for Erosion and Geotextile tube Construction

- ❖ Low tide level -0.5m.
- ❖ High tide level +3.6 m.
- ❖ Storm Surge + 1.4 m
- ❖ Sea Encroached 500 m distance since 1975.
- ❖ Erosion about 10 to 15 m per annum.
- ❖ 21st Nov 2009 shore at 50m from retarded Embankment.
- ❖ 23rd Oct 2011 shore at 33 m from retarded Embankment.
- ❖ Beach slope is very Gentle.
- ❖ Storm surge waves of high intensity break on the Embankment

Prof. R. Sundaravadivelu, IITM



Design of Geotextile Tube Saline Embankment

- **Filler Material**

Fill material for filling the geotextile tubes for coastal and hydraulic applications will normally consist of fine sand and will contain not more than 15% of fines by weight to minimize subsidence of the tubes after filling.

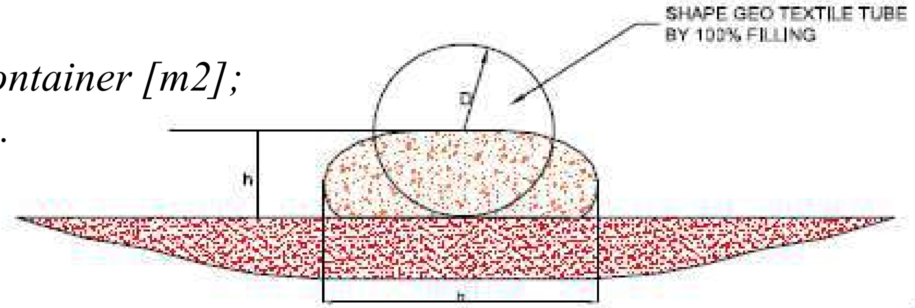
- **Filter**

Where

O_{90} - Pore size of the geotextile tube [µm];
D_{10} - Diameter corresponding to 10% finer [µm];
Cu - Coefficient of uniformity infill soil [-].

- **Degree of filling**

f = degree of filling [-];
A = filled cross-sectional area of the geotextile container [m2];
S = circumference of the geotextile container [m].



➢Design of filter and scour apron

Two layers of Geotextile filter filled with about *450mm* sand on the land side and *750mm* sand on the sea side placed on the *1:60* bed slopes which ensures good drainage and increases bearing capacity. Scour by breaking waves on sloping structure was calculated using the following relation.

$$\frac{S_m}{H_s} = 0.01\left(\frac{T_p\sqrt{g H_s}}{h}\right)^{3/2}$$

S_m = maximum scour depth from bed level
T_p = peak wave period
H_s = significant wave height
h = water depth in front of sea wall

➢Design of toe mound

The design criteria assumed for the toe protection was Hudson's formula, In which Width should be maximum of twice the significant wave height are else *0.4* times design water depth, height of the mound should be *50%* of the width.

➢Design of core

Geotube filled with granular coarse material at uniform filling ratio has to be maintained to have a uniform height and width, which helps in mobilizing uniform level of crest which relays in proper alignment and assurance for stability.

➢Design of gabion layer

Gabion box weight was assumed based on Hudson's formula and corresponding gabion dimension where selected.

$$W = \frac{W_r H^3}{k_D (S_r - 1)^3 \cot\theta}$$

➢ Gabion Layer Thickness

t = Effective layer thickness to be protected [m];
n = number of equivalent armour [-];
=Layer coefficient [-];
= Equivalent Diameter of Gabion box [m];
= Required design Weight of Gabion[tones];
= Unit weight of Gabion box[t/m^3].

➢ Design Strength of Geotextile Tube

Where,

= Ultimate required strength for the geotextile [kN/m];
T_m= Maximum tensile loads on the geotextile [kN/m];
F = Overall safety factor [-].
= Factor of safety for creep [-].
= Factor of safety for seam strength [-].
= Factor of safety for installation damage [-].
= Factor of safety for chemical degradation [-].
= Factor of safety for biological degradation [-].

PHYSICAL DIMENSIONS OF GEO-TUBE EMBANKMENT

Type of Structure	Prototype	Model (Scaling factor 1:10)
Geotube Circumference	9m	0.9m
Geotube Diameter	3m	0.3m
Gabion Box Dimension	2mx1mx1m	0.2mx0.1mx0.1m
Slope	Sea Side-1:1	Sea Side-1:1
	Land Side-1:2	Land Side-1:2
Water Level At Sea Side	4m, 5m, 6m, 7m	0.4m, 0.5m, 0.6m, 0.7m
Maximum Height Of Water Depth (D_{max})=Htl+Strom Surge	4.8m	0.48m
Breaking Wave Height (H_b)	3.8m	0.38m
Significant Wave Height (Hs)	2.8m	0.28m

Wave Parameters

- Wave Height : 5cm, 10cm,15cm, 20cm, 25cm, 30cm, 35cm
- Time Period : 1.58 sec, 1.89 sec, 2.21 sec, 2.52 sec, 2.84 sec,3.16 sec, 3.47 sec, 3.79 sec, 4.74 sec
- Water depth : 0.4 m, 0.5 m, 0.6 m, 0.7 m

Side view of the Geo tube & Gabion Structure

- Based on conservation Principles
  $K_R^2+K_T^2+K_L^2=1$
- **Reflection Coefficient $K_R^2 = H_R/H_I$**
- **Transmission Coefficient $K_T^2 = H_T/H_I$**
- **Loss Coefficient $K_L^2 = \sqrt{(1- K_T^2 - K_R^2)}$**

- Where

H_R is the height of the transmitted waves on the seaward side of the structure

H_T is the height of the transmitted waves on the landward side of the structure,

H_I is the height of the incident waves on the seaward side of the structure.

Analysis For Hydrodynamic Coefficients

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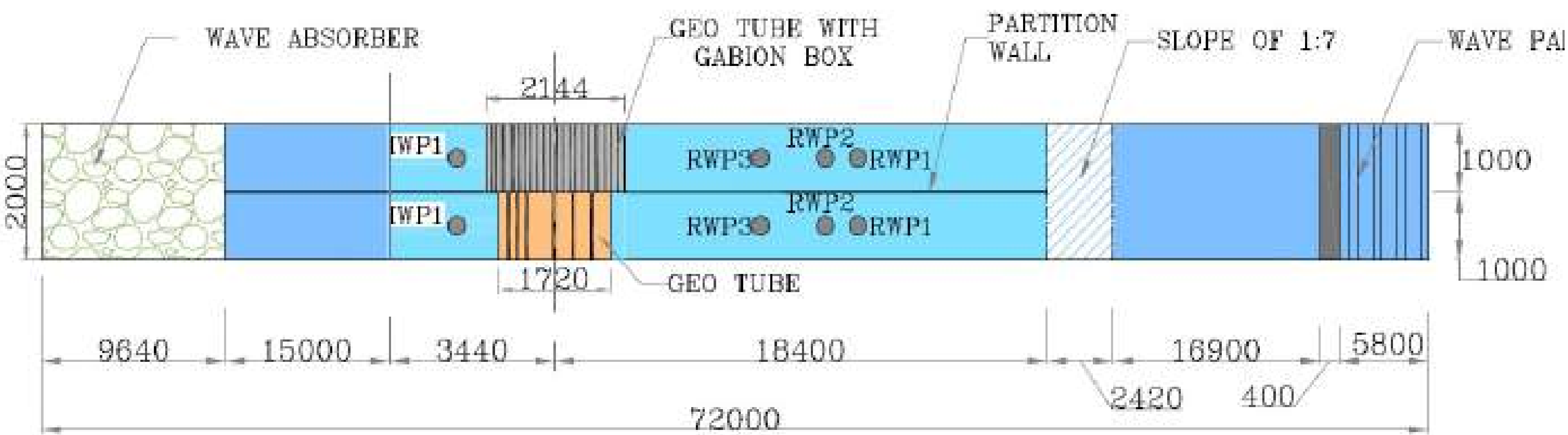
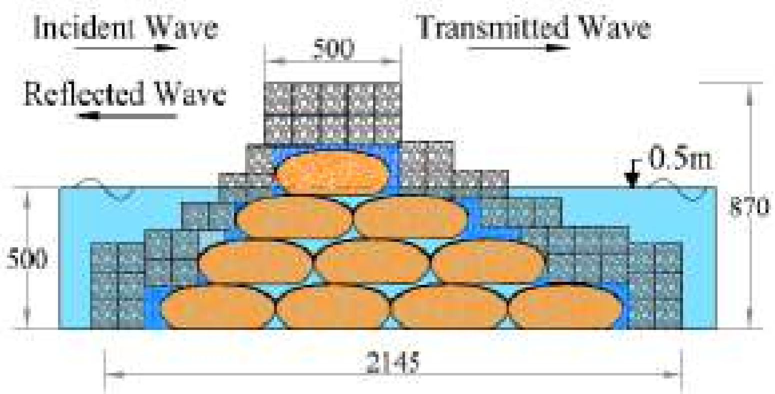
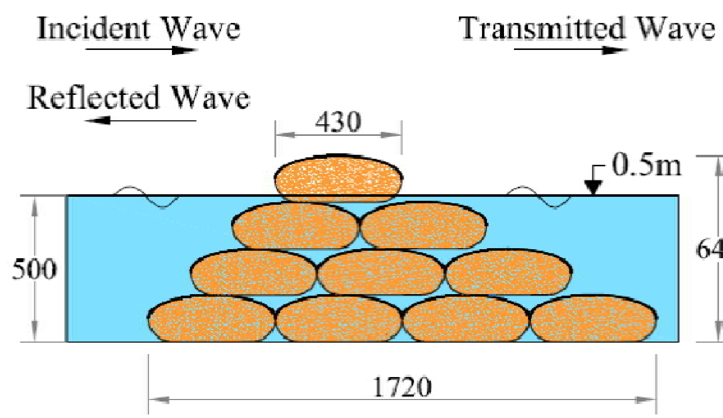
Plan view of flume arrangement



Typical cross section of geotube with gabion



0.5m WATER DEPTH



0.5m WATER DEPTH

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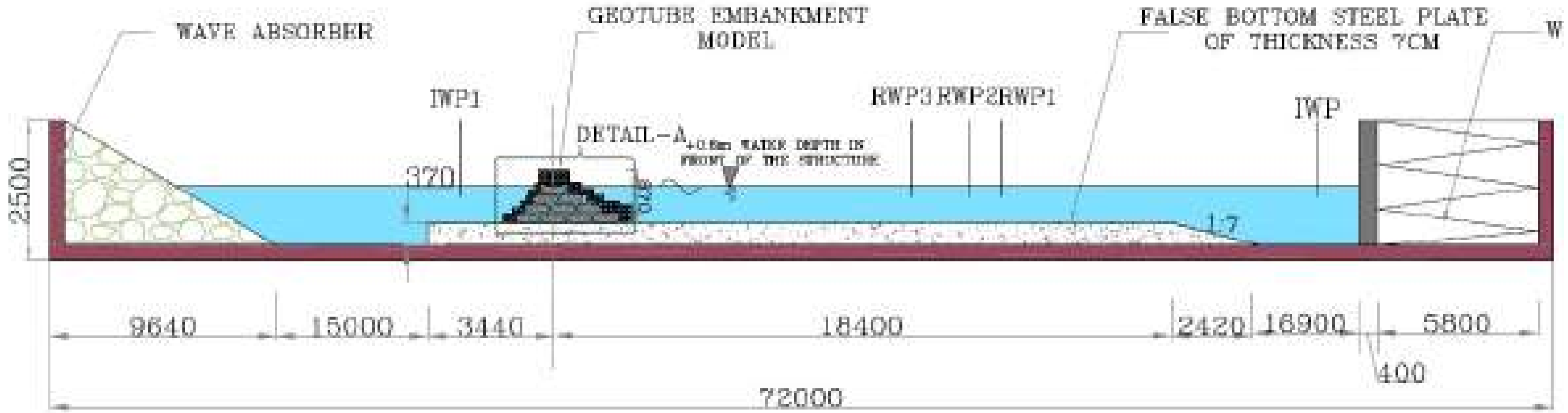
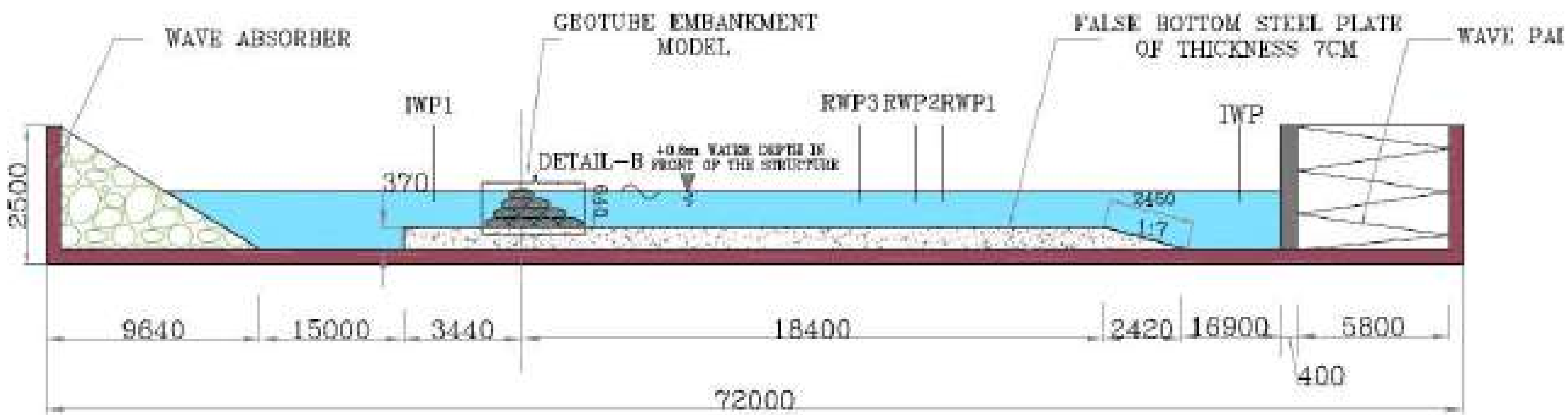
Wave flume cross section of separated section geotube with gabion

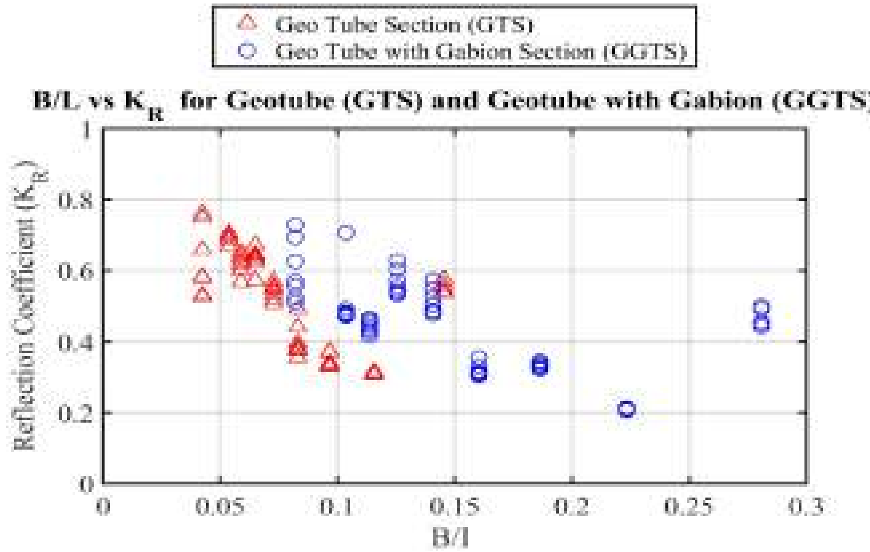


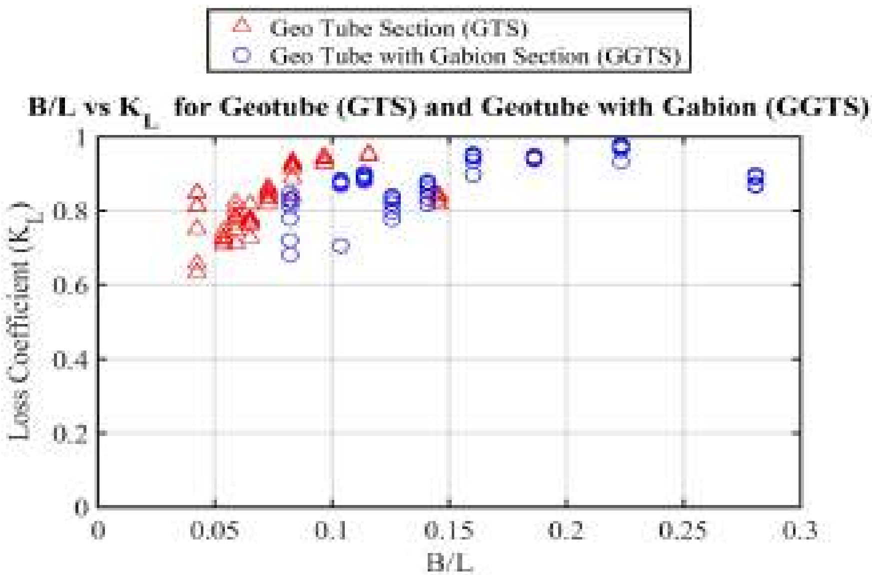
Wave flume cross section of separated section with only geotube

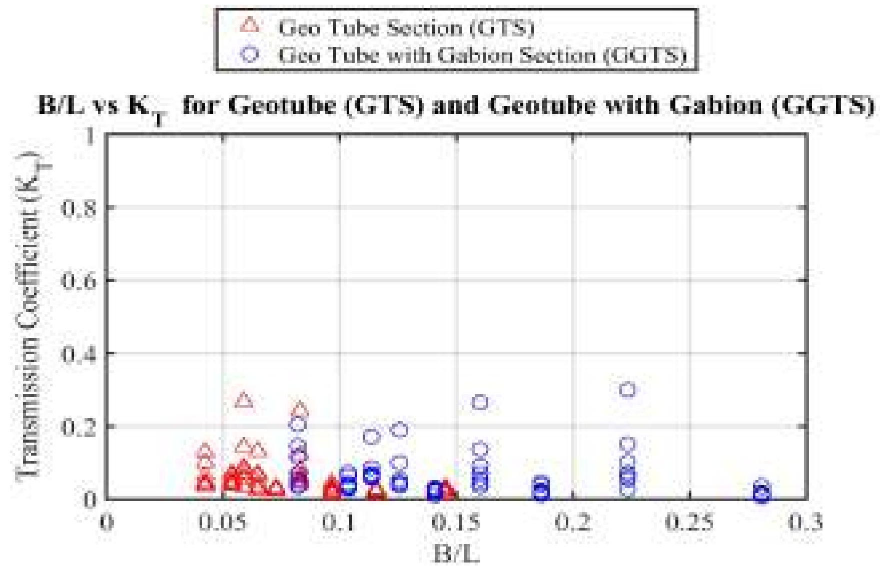


Effect of Relative Crest (B/L) on Hydrodynamic Coefficients

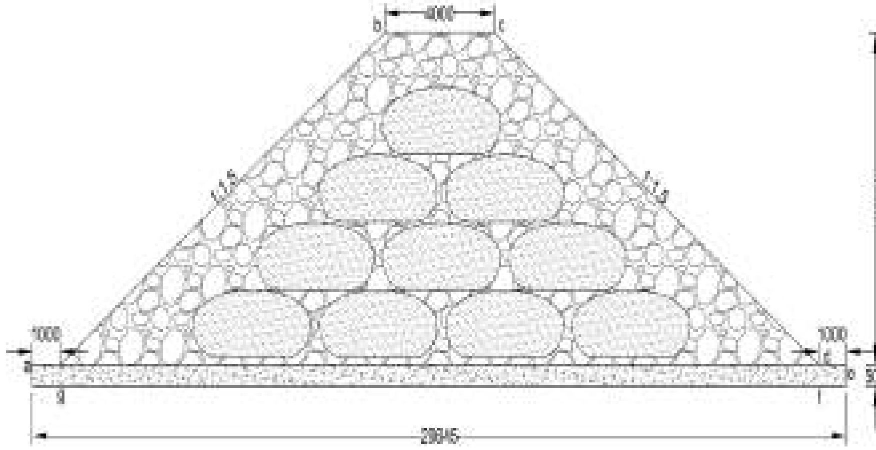
Prof. R. Sundaravadivelu, IITM

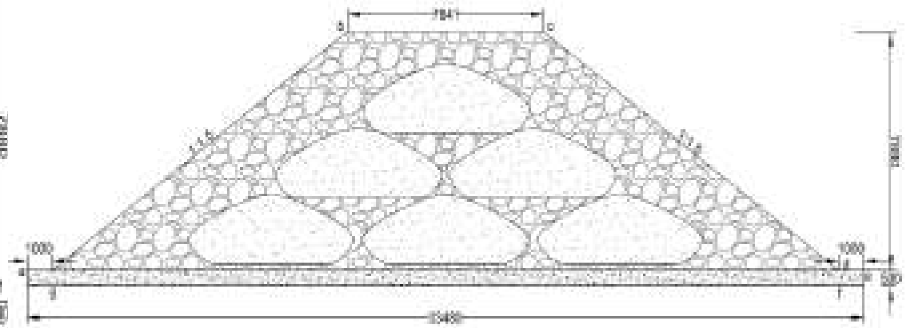


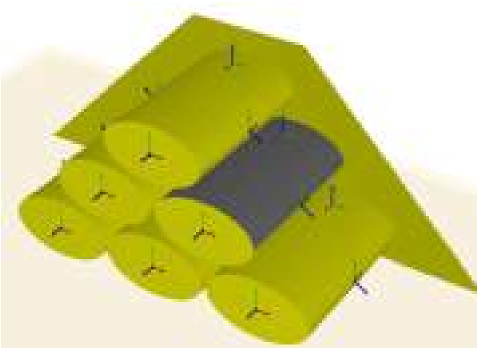


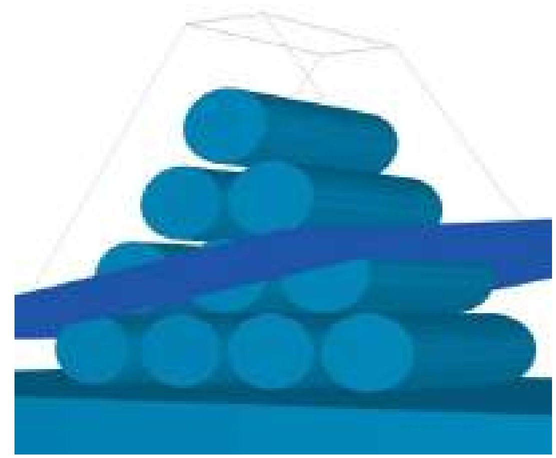


Physical Dimensions of Geotube Configurations



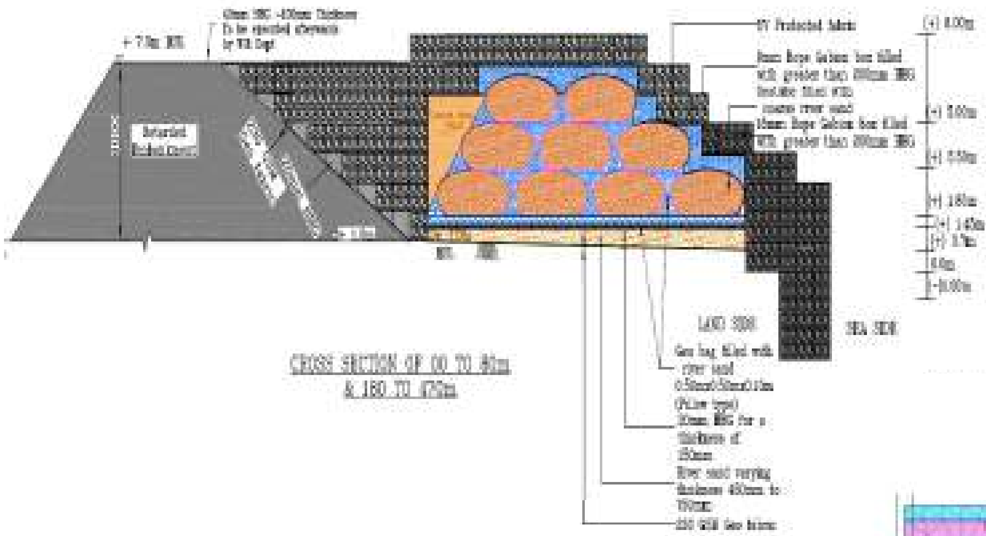


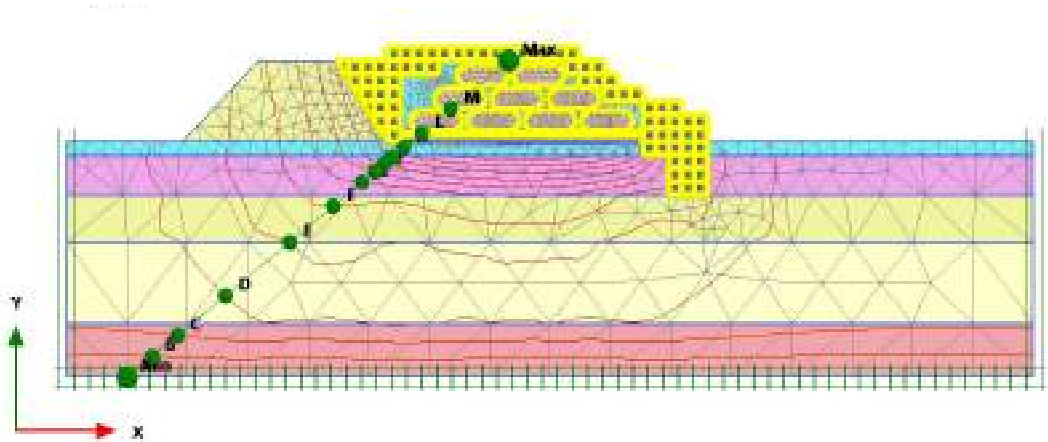




Prof. R. Sundaravadivelu, IITM

Typical Cross Section for integrated Embankment with scour apron and Numerical output



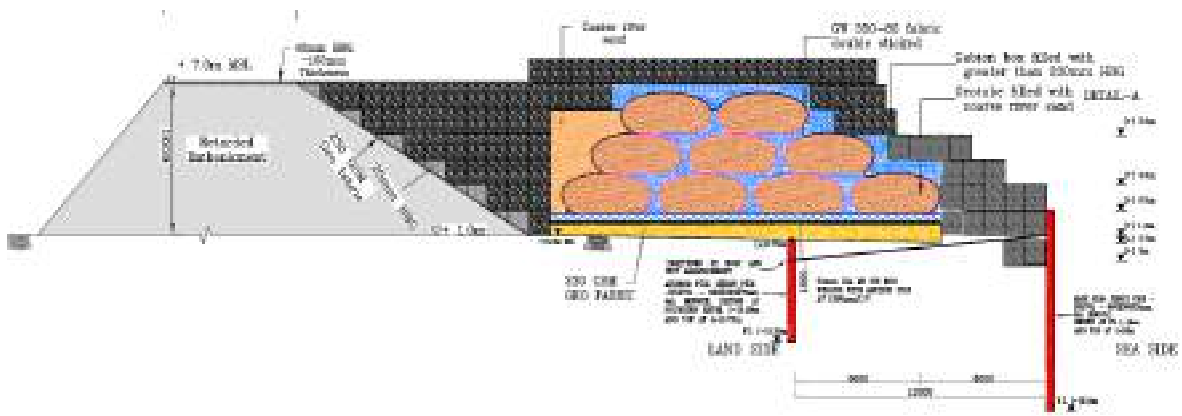


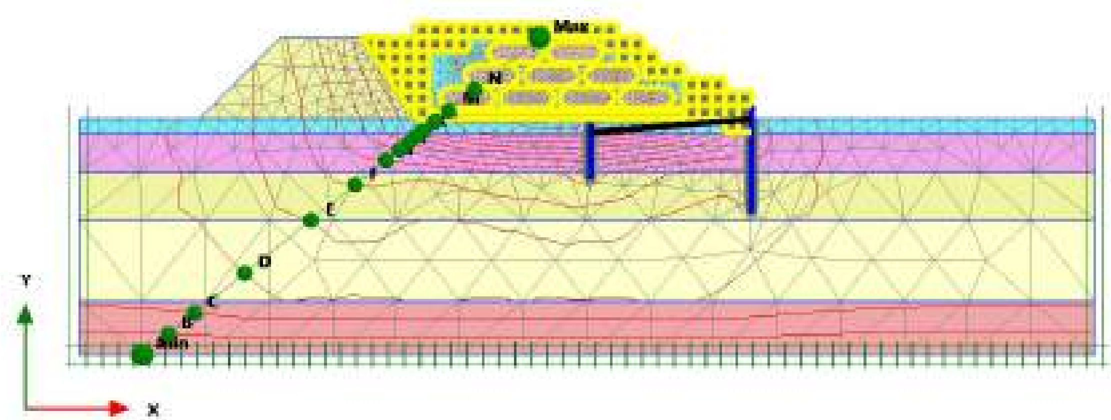
Total displacements |u|

Maximum value = 0.2563 m (Element 22 at Node 473)

Prof. R. Sundaravadivelu, IITM

Typical Cross Section for Integrated Embankment with scour apron and sheet pile toe

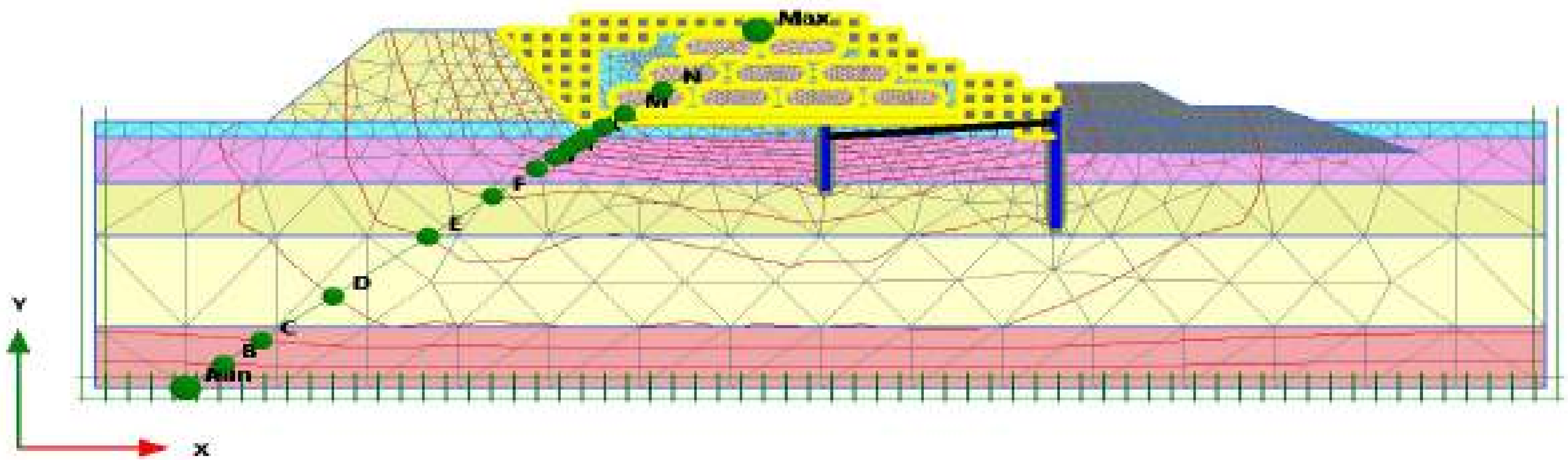




Total displacements |u|

Maximum value = 0.2792 m (Element 22 at Node 26408)

Integrated Geo-tube Embankment (Sheet Pile Wall with Rubble Mound Toe)



Total displacements |u|

Maximum value = 0.2786 m (Element 22 at Node 28563)

Summary and Conclusion

➢The present numerical study, patterning to geo tube embankment, reveals good geotechnical stability against the scour.

➢ Hydrodynamic performance shows that gabion has good wave energy dissipation characters.

➢10 tubes with 4.3 m Geotube is better

➢Sheet pile with rubble toe protection is recommended where wave energy concentration is observed

➢Sand accretion is observed after construction due to energy dissipation of Gabion box

➢Scour protection is essential

➢The geotextile bedding layer sandwiched with 500mm thick sand is distributing the load effectively to the subsoil

Sea Wall Embankment at Pentha, Odisha

- Commonly adopted structures to impede the actions of waves :

  Seawall, Spurs, Dykes, Groynes,

  Groynes, Shore revetments

- Conventional structures :

Material → Masonry, Concrete } Cost Prohibitive

Performance → Undue Distress → Settlement, Large Wave Action

DEPARTMENT OF WATER RESOURCES, GoO,

Problem faced: contd…

- In 2008, shoreline was 20m away from existing old embankment.
- Traditional method of protection using wooden stump piling and Big Resin PVC sand bag dumping was done every year.
- In the Year 2009-10, a retard embankment was constructed on the backside of the old embankment at a distance of 60 metre
- In 2011-12, the shoreline over-ran the old embankment for a length of 400 metre & washed away the embankment for 350 metre length.

Prof. R. Sundaravadivelu, IITM

DEPARTMENT OF WATER RESOURCES, GoO, PEA

Prof. R. Sundaravadivelu, IITM

DEPARTMENT OF WATER RESOURCES, GoO,

Pictures of the site: (Dt. 20.10.2011)

400 metre length of embankment as protected earlier by sand bags are fully washed by high surge

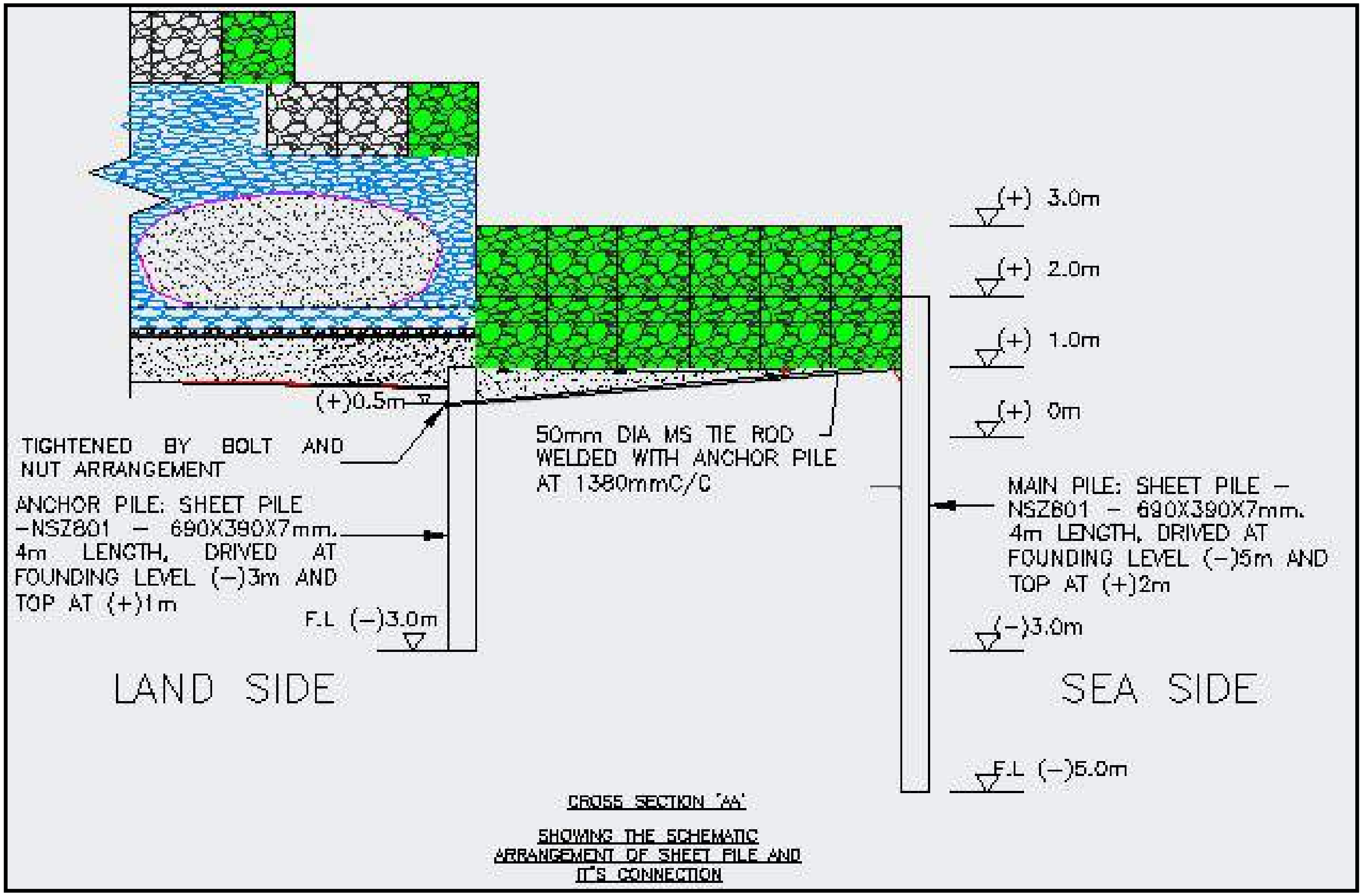
Installation of pumps on a barge to supply water for sand slurry

Geo tube laying starts after preparation of bed

Filling of Geo Tube

Prof. R. Sundaravadivelu, IITM

Two row sheet piles installed in cut off zone for the most critical length of 100 mtr (RD 80 to 180 m)



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Sheet pile being installed by using vibro hammers

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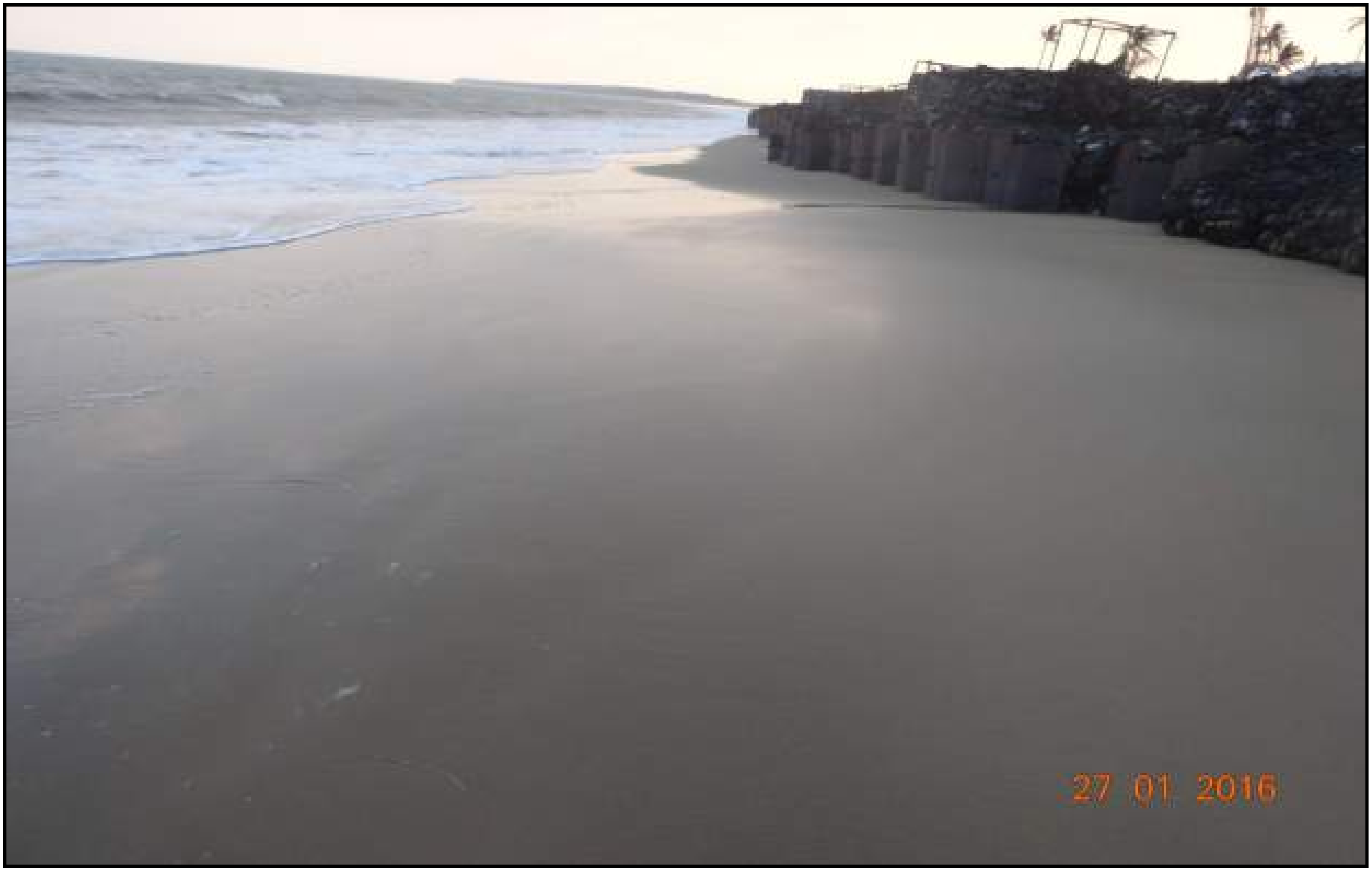
Sheet pile driving in progress

Sea Wall Embankment at Pentha,

- The installation of the sea wall embankment was completed on 10th June, 2016.
- Being a pivotal project, it was featured in the Intergovernmental Panel on Climate Change (IPCC) as India's initiative on the environmental change.

Conclusion : Accretion of sand has started in front of Geo Tube



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Accretion of sand has started in front of Geo Tube

Thank you

Dr.

Use of Geosynthetics in Marine Environment

Webinar on Geo-synthetics for Coastal Protection and Port works

10th and 11th Dec 2020

eobags – A Case Study of Usage in Earth etention

MODULAR FABRICATION FACILITY, KATTUPALLI PORT

BERTHS 14 & 16, KANDLA PORT

Port berths, JSW Jaigarh port

Dr.

Modular Fabrication Facility, Kattupalli Por

First Completed: 2010

Structural system Adopted: BCI Contiguous Pile in front with concrete de

- **First Rehabilitation: 2012**
  - **Solution: Extension of Jetty behind the existing jetty and close all sides and shorelines using Gabion Blocks**
- **Final Rehabilitation: 2016**
  - **Solution: Contiguous Piles with Geobags in Front with sides closed by Contiguous piles with geobags on the North & Gabion Blocks on the South.**

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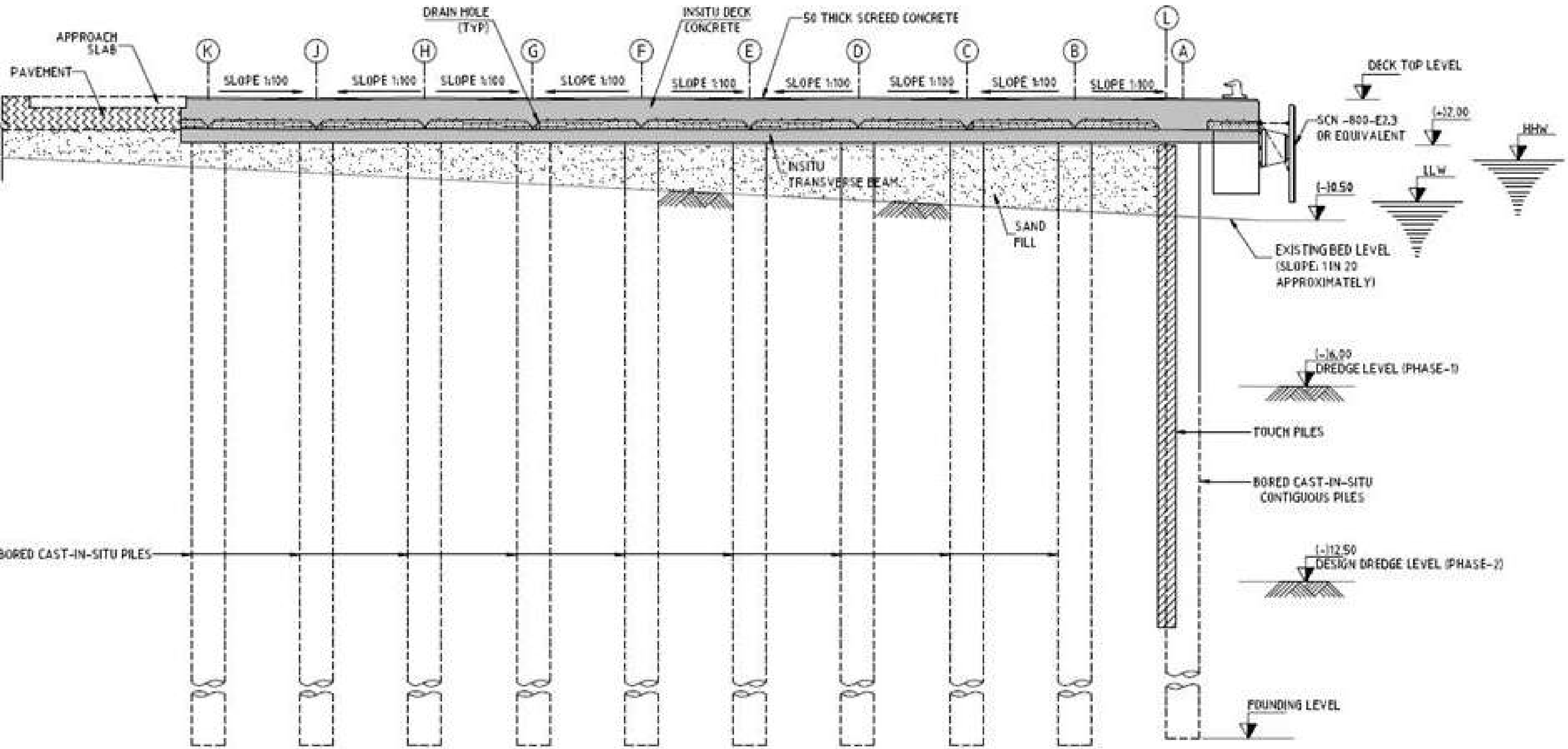
Dr.



Final Rehabilitation: 2016

- **Solution: Contiguous Piles with Geobags in Front with sides closed by Contiguous piles with geobag on the North & Gabion Blocks on the South.**

odular Fabrication Facility, Kattupalli Port

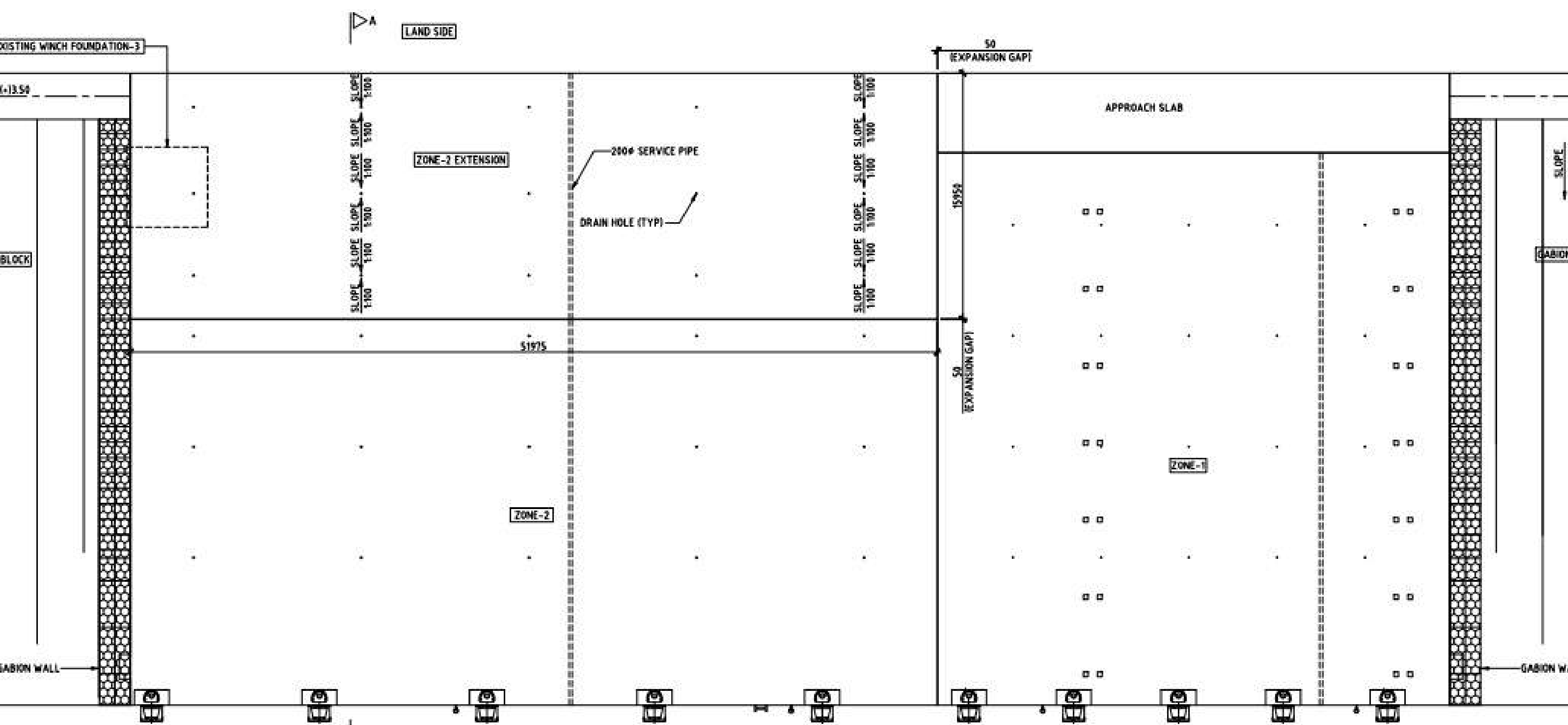
e Issues:

- dged soil used for backfilling has high ne sand content
- nstructed retaining system of contiguous iles has some gaps which has lead to vater intrusion and thus continuous soil rosion and loss of back fill
- ange in pile fixity and Loss of pile apacities – in strength & serviceability

Proposed Options

- Diaphragm wall in front of existing wall
- Sheet pile wall in front of existing wall
- Extension of jetty behind the structure
- Combi-wall in front of existing wall
- Diaphragm wall behind structure
- **Contiguous piles with geobags**

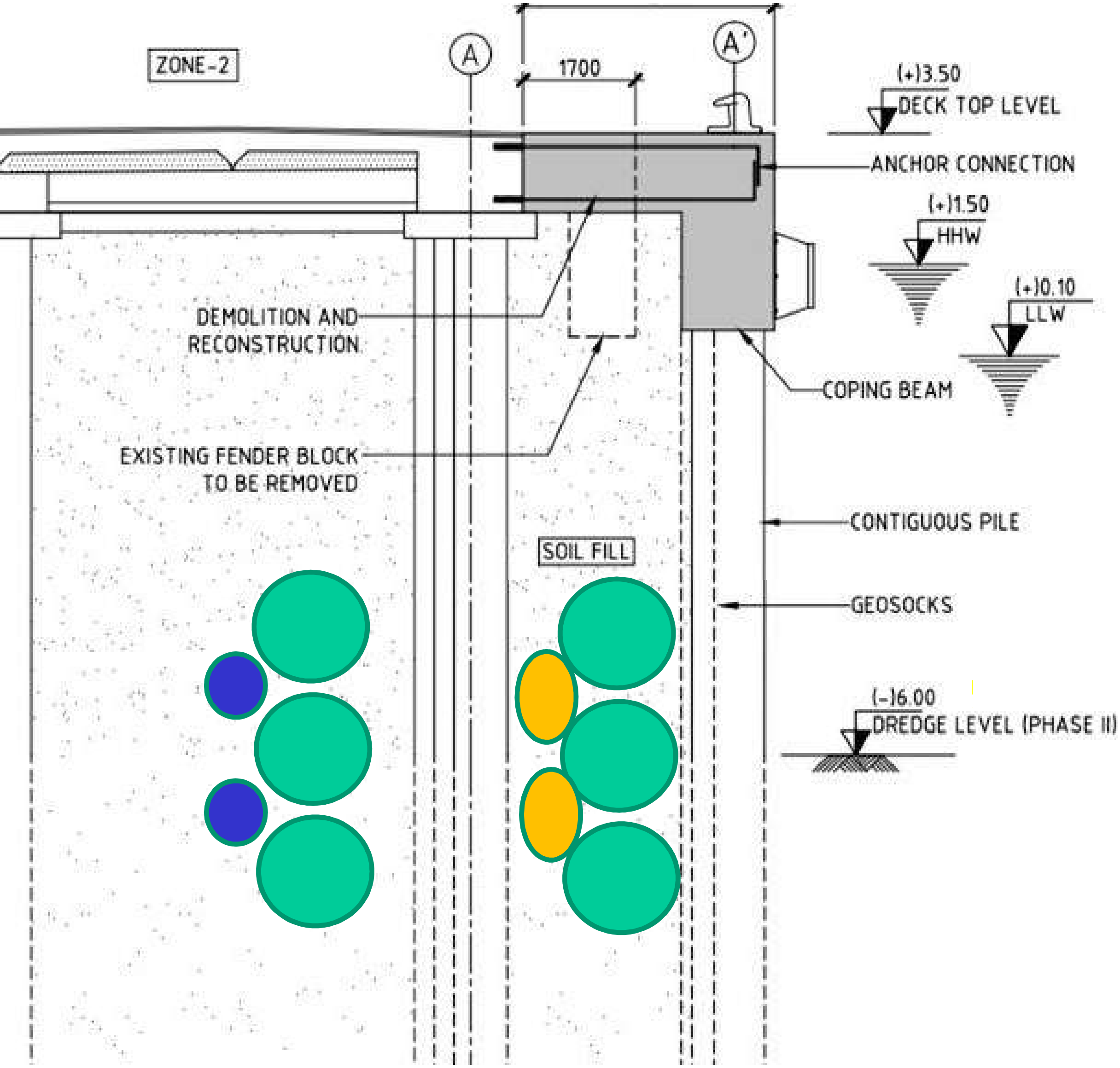
Dr.

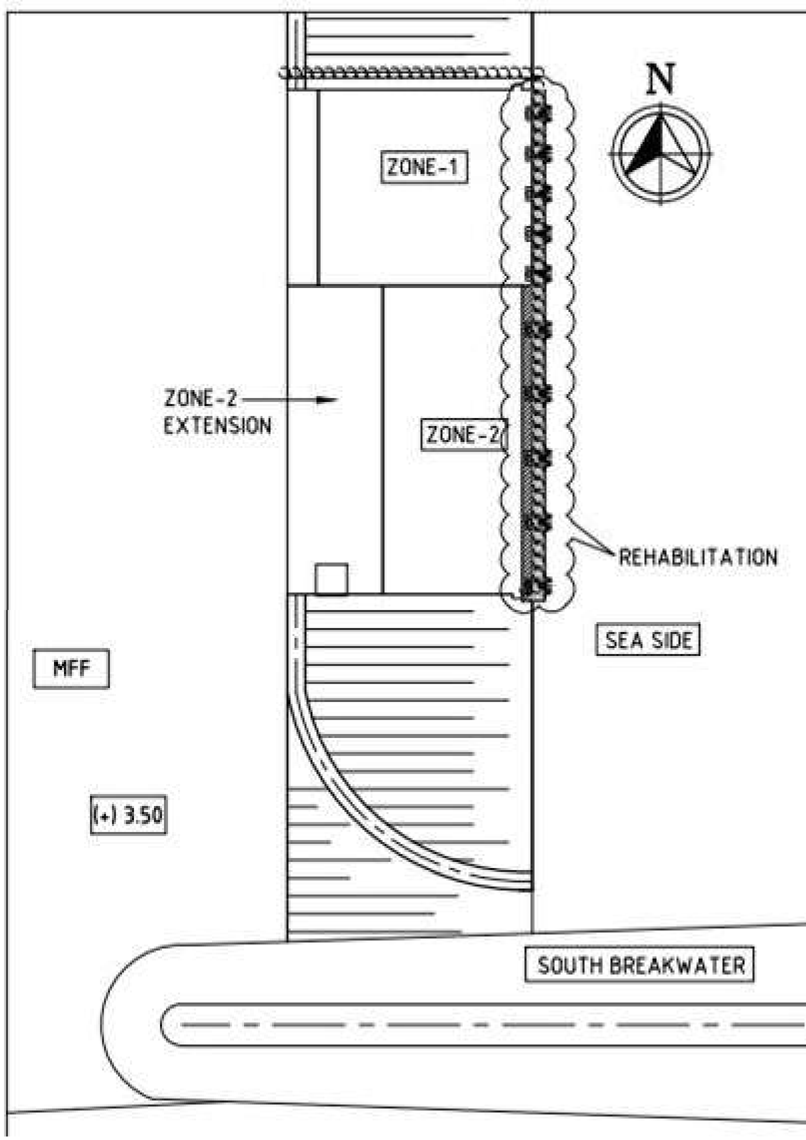


First Rehabilitation: 2012

- **Solution: Extension of Jetty behind the existing jetty and close all sides and shorelines using Gabion Blocks**

Dr.





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10th and 11th Dec 2020

Rehabilitation Steps

- Construct Contiguous Piles
- Demolish front cantilever of the Je
- Install Geobags by means of tying Pile top
- Fill the Geobags & behind the pile
- Construct capping beam & connec existing jetty – close top

Rehabilitation Works

Contiguous Piles with geosack collar

Dr.

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10th and 11th Dec 2020

Dr.

Container/ Bulk Berths, JSW Jaigarh Port

Designed in 2012-13

Constructed in 2016-17

ues at Site

Bund Unstable when loaded for surcharge (50 kN/sqm)

Area reclaimed with dredged Material

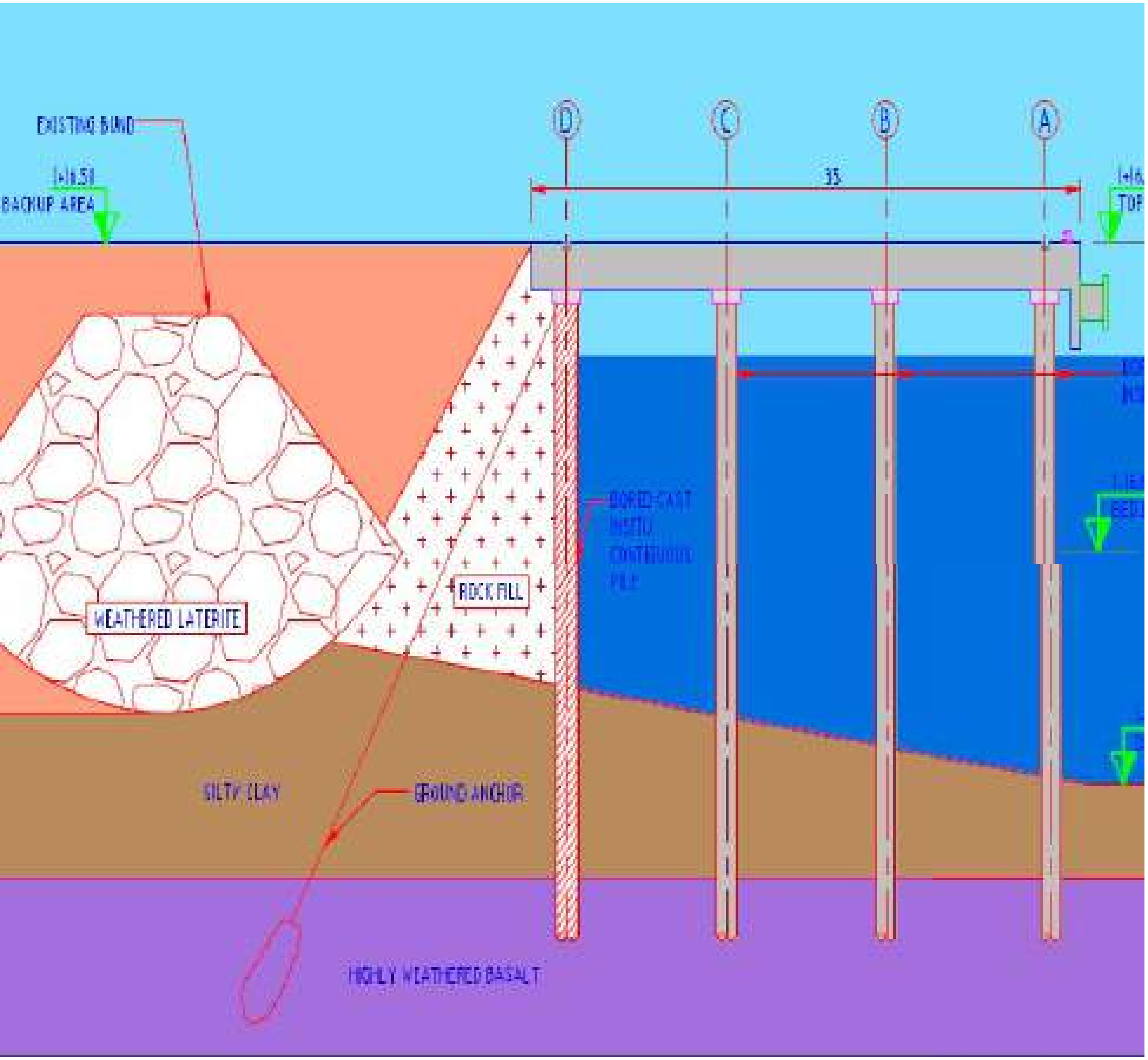
Berth Pocket already dredged till (-) 13.5 m CD

A Bund constructed on Soft Clay

omparison of Various Options

e options were checked for feasibility under various parameters such as Suitability for Site Operations, for Immediate Backup Area, for Soil Type, onstruction ease etc and tabulated in Table 1: Comparison Table

Dr.



Option 2- Contiguou Pile at Rea tied to Ro Anchor

Under constructi images

View from ex
rock bund
initial stage

Under constructi images

View from ex
rock bund
Partially
constructe

Under constructi images

Rock Anchor drilling fror partially constructe berth

Complete
Bulk Bert
Unit

View on the side

Dr.

Site plan

Dr.

TH AND 16TH LTIPURPOSE RTH AT KANDLA, JARAT

14th and 16th have the same structural uration. The length of the berth is about 300m and 55m.Two rows of 1400mm diameter (Pile grids A nd six rows of 1300mm diameter piles (pile grids) are proposed to support the RCC deck slab the berth direction

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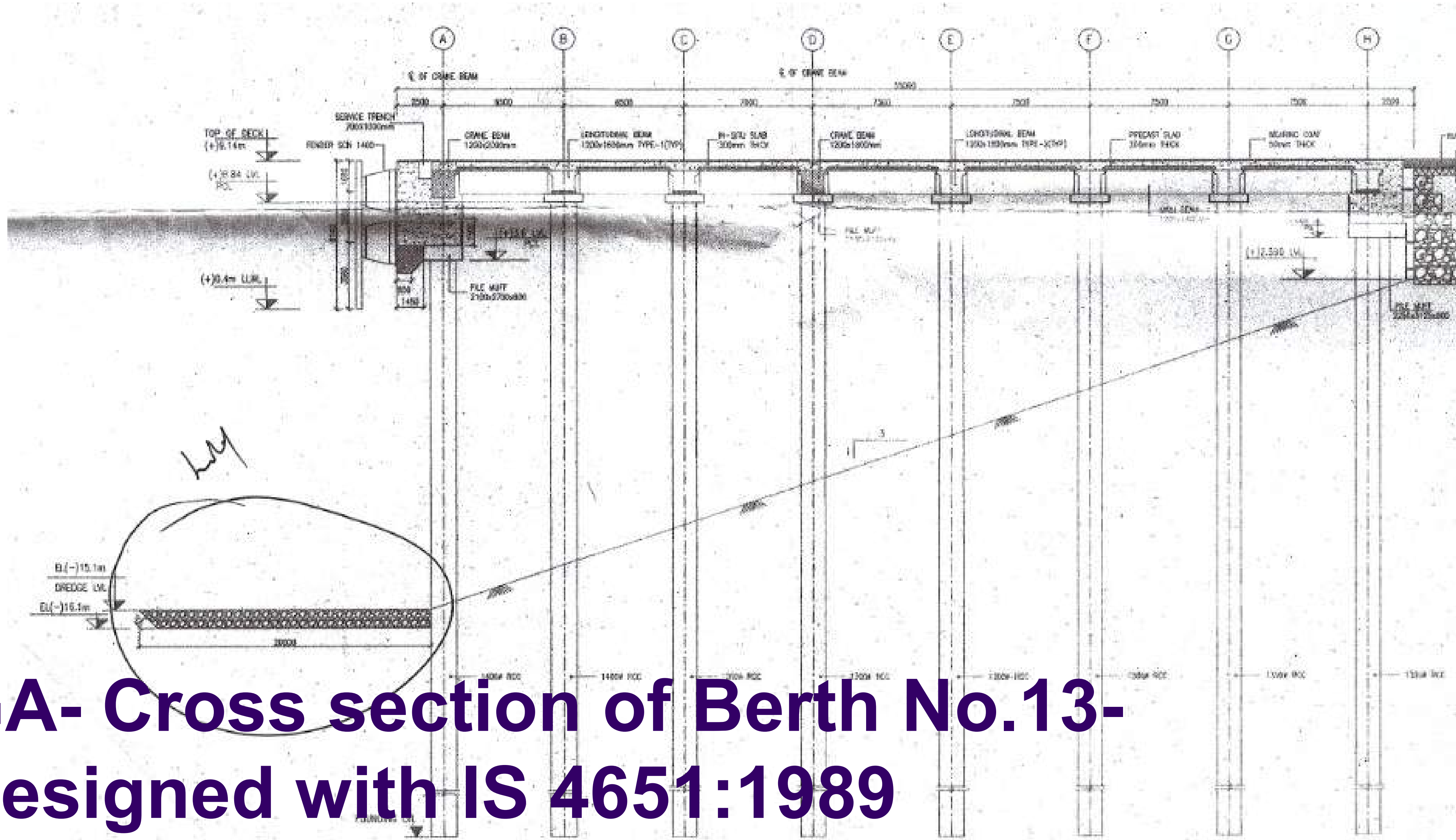
10th and 11th Dec 2020

The Problem

- It was informed that the cost estimate and BOQ for the proposed berths were prepared based on the existing 13th berth details.
- The berth 13 was designed as per old code of IS 4651.
- As the present design is done based on the revised code i.e. IS 4651-2014, the provisions of the same need to be satisfied, the new crackwidth criteria is 0.1mm compared to 0.3mm as per earlier code.
- In order to retain, the diameter of piles proposed during tender stage, earth pressure on the berth from the backup area should be avoided.
- Hence, a study on alternate options was taken up in place of the proposed cutoff wall.

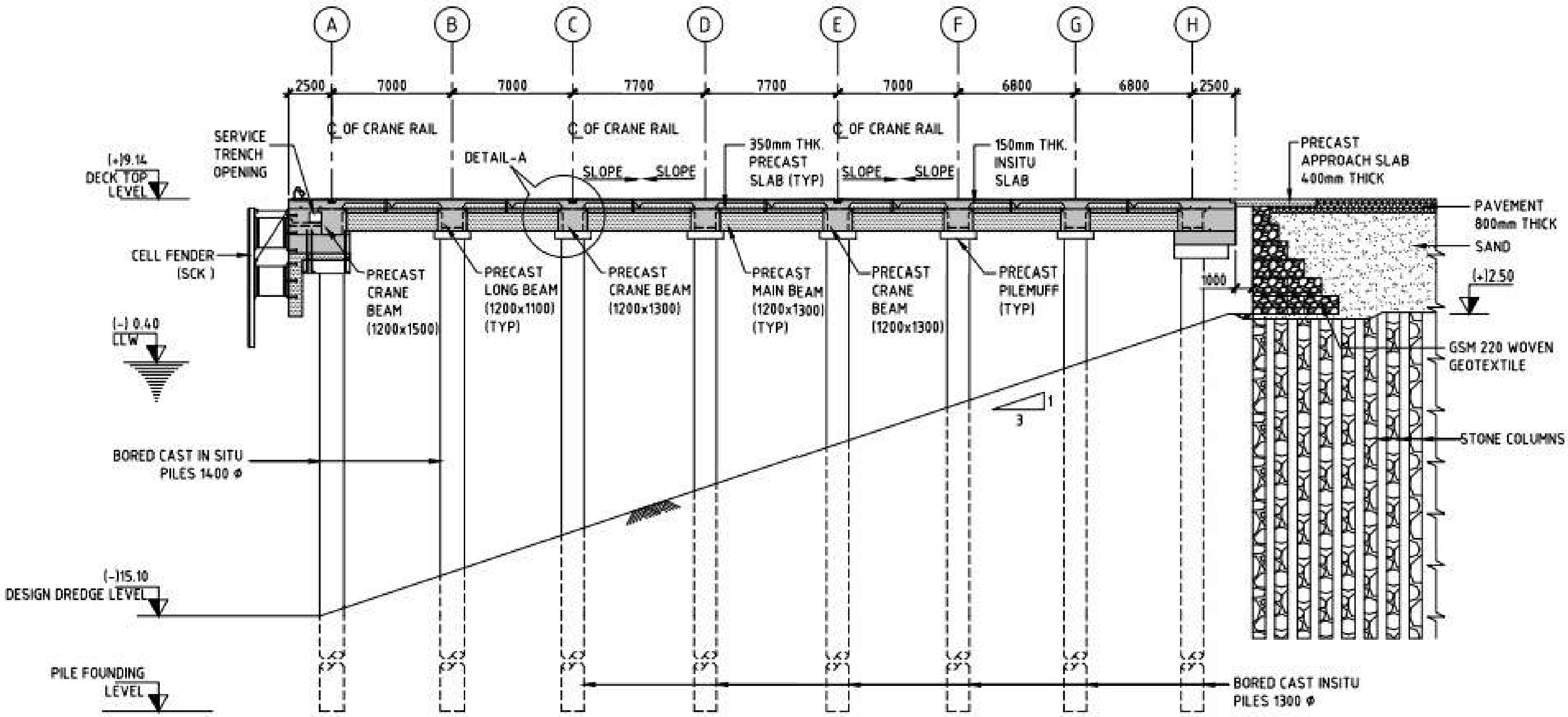
Dr.



GA- Cross section of Berth No.13- Designed with IS 4651:1989

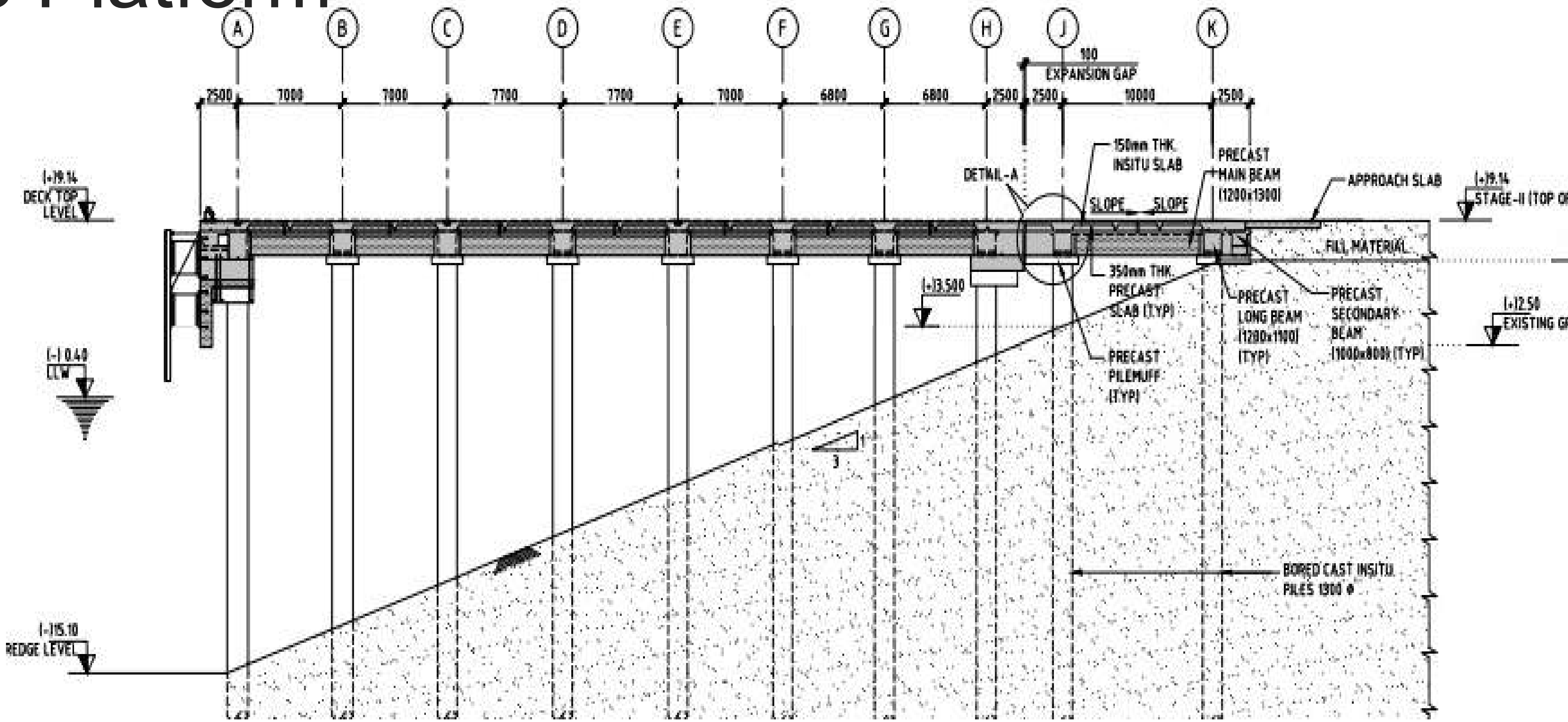
Dr.

GA- Cross section of Berth Option 1 -Stone Column with Gabion wall



Dr.

A- Cross section of Berth Option 2 –Extended Pile Platform



Dr.

Comparison of options

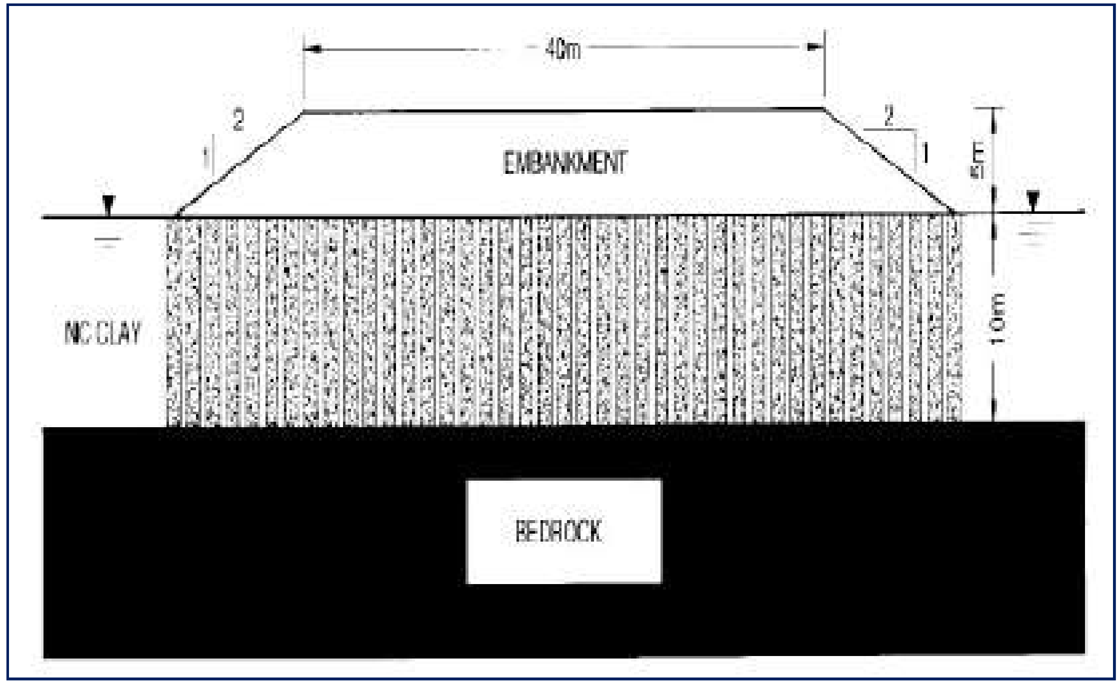
Description	Gabion wall	Extended Platform	Sheet pile	Contiguous Pile	Diaphragm wall
Constructability	Needs stone columns for stability of gabion wall	Normal construction	Needs specialised equipment's in marine mode	High number of piles	Construction in marine mode not possible, filling required
Items of work in line with existing contract	New Items	Existing items	New Items	Only Touch piles are new items	New Items
Cost in Cr	70	65	147	163	183


Thank You… Any Queries?

GUIDELINES ON GEOSYNTHETICS FOR COASTAL PROTECTION AND PORT WORKS

Ground Improvement

3D View of Band Drain Stabilised Bed



Embankment on Stone column stabilized bed

By
Dr. K.ILAMPARUTHI
Professor & CHAIRMAN (Retd.), Faculty of Civil Engineering
College of Engineering Guindy, Anna University.

GUIDELINES

CONTENTS OF PRESENTATION

- NEED FOR IMPROVEMENT
- GROUND IMPROVEMENT OPTIONS
- GROUND IMPROVEMENT USING VERTICAL DRAINS
- GROUND IMPROVEMENT USING STONE COLUMNS

GUIDELINES

NEED FOR GROUND IMPROVEMENT IN PORT AREAS

- Dearth of Land
- Poor Soil Condition
- High Intensity of Load
- Stringent Design Conditions
- Speedy Construction

Purpose

PURPOSE

- Increase Bearing Capacity
- Reduce Compressibility
- Increase Rate of Consolidation
- Improve Dynamic Load Response
- Reduce the Risk of Liquefaction

Purpose

OPTIONS

- Alternate Alignment
- Pile Foundation
- Pre-compression by Surcharge Load
- Pre-compression by Drains & Surcharge Load
- Modification by Stabilization

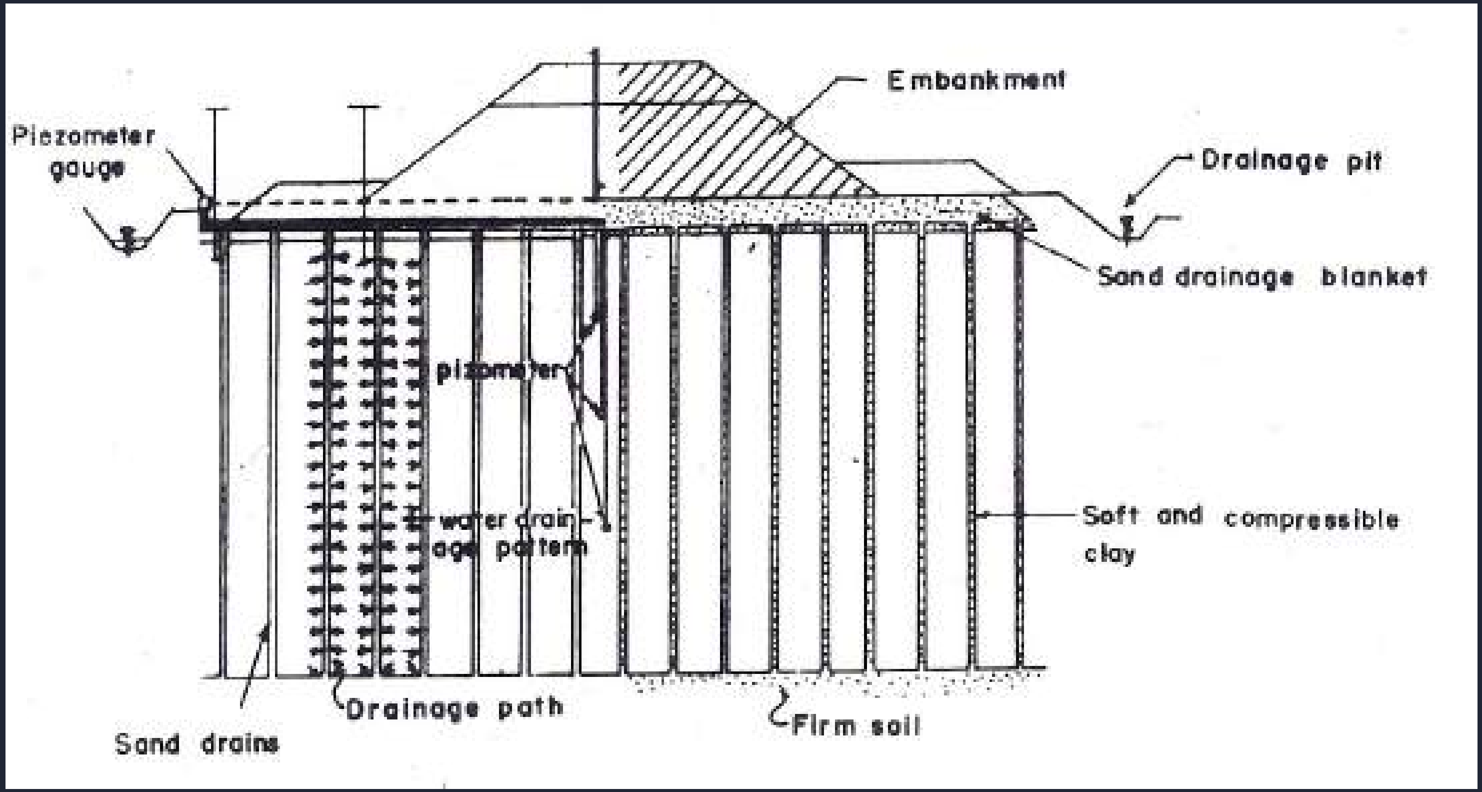
Necessary Data

NECESSARY DATA- Subsoil & Structure

- Subsoil profiles and their depth
- Properties of each layer
- Penetration Resistance
- Type of structures
- Intensity of Load
- Serviceability Limits

BAND DRAINS

EMBANKMENT ON BAND DRAINS STABILZED GROUND – General Arrangement



BAND DRANS - GUDELINES

BAND DRAINS

- Scope
- Materials
- Design
- Method of Installation

BAND DRAINS - GUIDELINES

BAND DRAINS – Scope & Materials

SCOPE: :

Stabilization of foundation soil by inclusion of band drains as per design and surcharging to achieve 90% to 95 % of consolidation within a predetermined time.

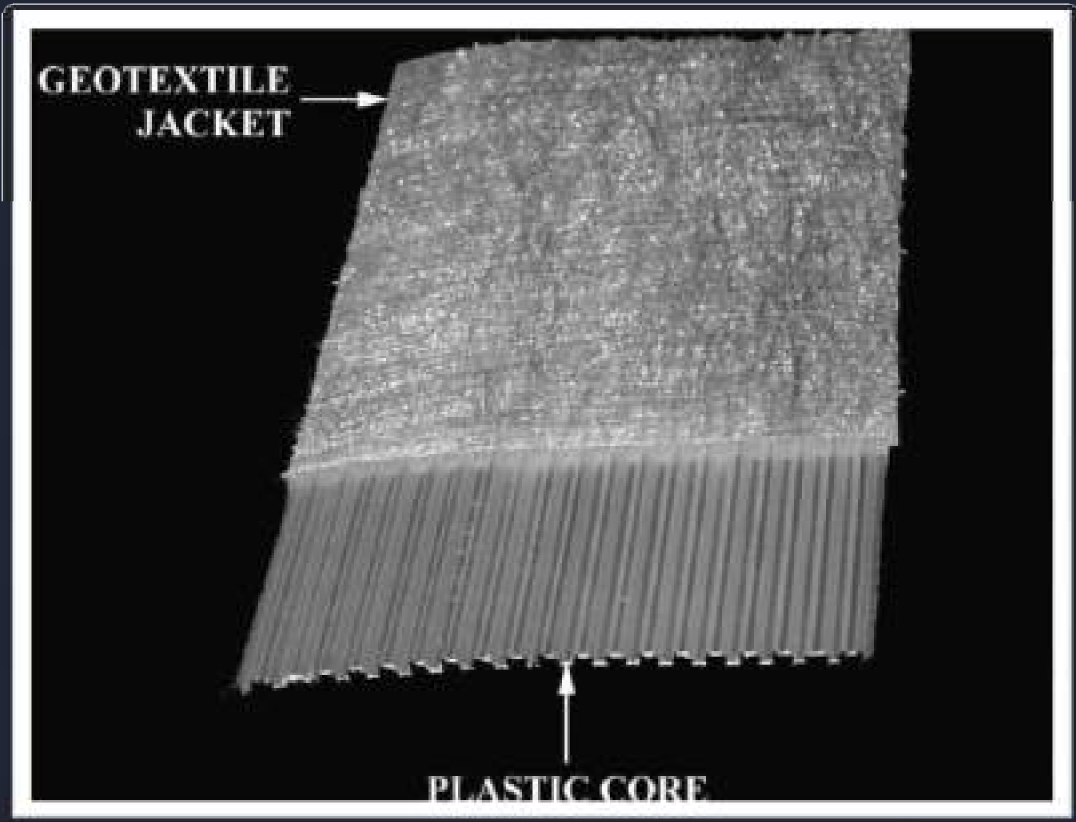
BAND (PVD) Drain

- **Plastic / polymeric core**
- **Thin geotextile filter jacket**

Size :

Width: 100mm (Minimum),

Thickness: 3mm to 9mm.



BAND DRAINS - GUIDELINES

BAND DRAINS – Properties & Specifications

Properties: :

i. Discharge capacity of composite drain
ii. Filter properties of jacket / sleeve (AOS)
   Permeability and Resistance to clogging
iii. Tensile strength of composite drain (Core and filter).

TESTS ON CORE & SLEEVE (Jacket)
(As per IS standards or ASTM / ISO)
Compatibility and Quality control

SPECIFICATIONS AS PER SECTION 704 OF MORTH

S. No	Property	Test Method	Value
A	Composite Drain		
1)	Width		≥100 mm
2)	Thickness	ASTM D5199	≥4 mm
3)	Tensile strength	ASTM D4595	>2.00 kN
4)	Elongation at break		>35%
5)	Discharge capacity i = 1.0 at, 300 kPa pressure	ASTM D4716	>1.5 x 10^{-5} m^3/s

BAND DRAINS - GUIDELINES

BAND DRAINS – Properties & Specifications

TESTS ON CORE & SLEEVE (Jacket)
As per IS standards or ASTM / ISO

SPECIFICATIONS AS PER SECTION 704 OF MORTH

S. No	Property	Test Method	Value
B	**Core**		
1)	Material		Polypropylene/Polyethylene
2)	Configuration/structure		Corrugated, filament, dimpled, studded etc.
C	**Filter**		
1)	Material		Polyester/polypropylene
2)	Structure		Nonwoven
3)	Mass per unit area	ASTM D5261	>120 g/m²
4)	Tensile strength	ASTM D4632	>500 N
5)	Elongation at break		>45%
6)	Trapezoid tear strength	ASTM D4533	>150 N
7)	Permeability	ASTM D4491	>5 x 10^{-6} m/s
8)	Apparent opening size	ASTM D4751	

- Soil retention ability
  - $O_{95} \leq (2 \sim 3)\ D_{85}$
  - and
  - $O_{50} \leq (10 \text{ to } 12)\ D_{50}$
- Permeability
  - $k_f \geq 10\ k_s$ (normally not a problem)
- Clogging resistance
  - $n \geq 30\%$
  - $O_{95} \geq 3\ D_{15}$
  - $O_{15} \geq (2 \text{ to } 3)\ D_{10}$

BAND DRAINS - GUIDELINES

BAND DRAINS – DESIGN

Radial consolidation Theory

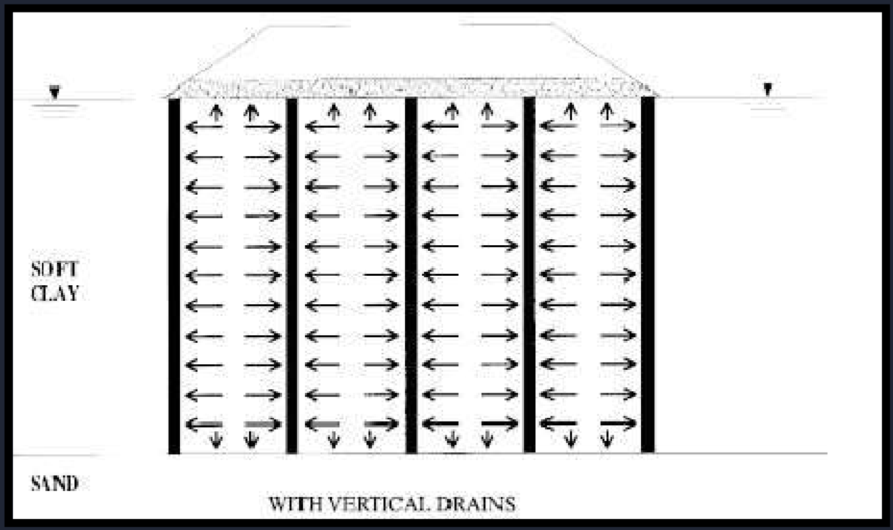
$$c_h\left(\frac{\partial^2 u}{\partial r^2}+\frac{1}{r}\frac{\partial u}{\partial r}\right)=\frac{\partial u}{\partial t} \qquad c_h=\frac{k_h}{m_v\gamma_w}$$

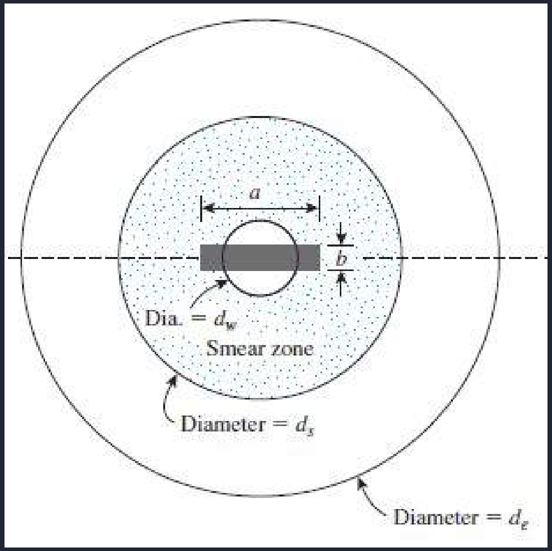
(C_h- Coefficient of horizontal consolidation)

Vertical & Radial consolidation Combined

$$c_v\frac{\partial^2 u}{\partial z^2}+c_h\left(\frac{\partial^2 u}{\partial r^2}+\frac{1}{r}\frac{\partial u}{\partial r}\right)=\frac{\partial u}{\partial t}$$

(C_h & C_v –Coefficients for Horizontal & Vertical consolidations)





UNIT CELL IDEALIZATION

BAND DRAINS-Design

BAND DRAINS-Equal Strain (Barron's Method-1948)

$$U_r = 1 - \exp\left(\frac{-8T'_r}{\alpha'}\right)$$

$$\alpha' = \left(\frac{n^2}{n^2 - S^2}\right)\ln\left(\frac{n}{S}\right) - \frac{3}{4} + \frac{S^2}{4n^2} + \frac{k_h}{k_s}\left(\frac{n^2 - S^2}{n^2}\right)\ln S$$

$T'_r = C_r\ t/d_w^{\,2}$

$$d_w = \frac{2(a+b)}{\pi}$$

U_r = radial degree of consolidation

U_v = vertical degree of consolidation

$T_r = C_r\ t/d_e^{\,2}$

$T_v = C_v\ t/\ H^2$

$n = d_e/d_w$

$S = r_s\ /r_w$

Determine the spacing "d" from effective diameter "d_e" from relation "(n)x(d_w)" for a degree of consolidation of predetermined time period

$$d = \frac{d_e}{1.05} \quad \text{(for triangular pattern)}$$

$$d = \frac{d_e}{1.128} \quad \text{(for square pattern)}$$

$$U = 1 - (1 - U_r)(1 - U_v)$$ ---- **(Carillo, 1942)**

BAND DRAINS - EQUIPMENT

BAND DRAINS INSTALATION

Equipment :

Use approved equipment to minimize the disturbance
Operation: Pushed, driven or vibrated

Crawler Excavator or Crane
Installation mast ("Stitcher")
Mandrel ($80cm^2$)

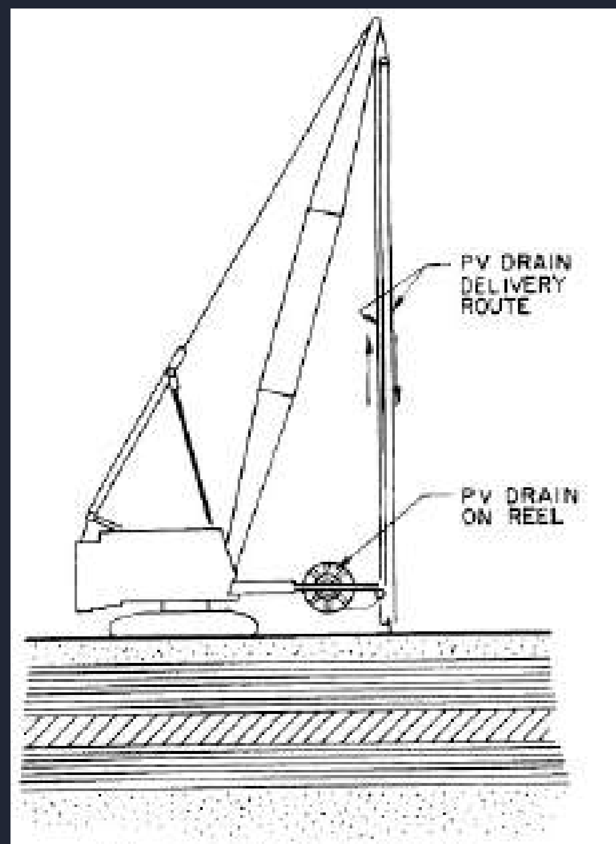
MANDREL

- Size and shape of mandrel shall be close to the size & shape of band drain.
- Length of Mandrel longer than the maximum installed length

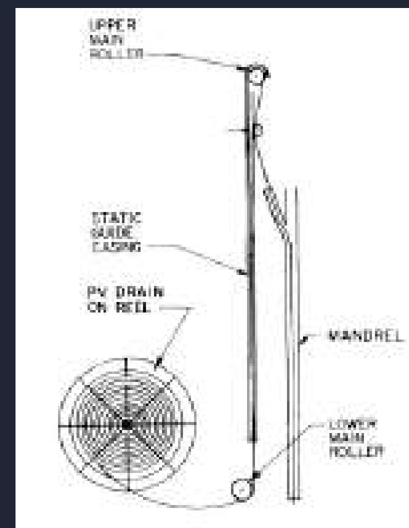
DRAIN DELIVERY ARRANGEMENT

ANCHOR

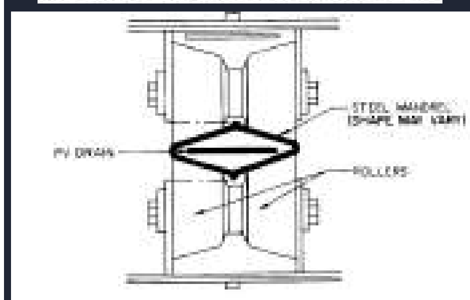
- Anchor plate to prevent entry soils and to anchor the tip of the drain at the required depth
- Anchor dimension shall confirm as closely as possible to mandrel dimension to minimize Disturbance



(A) INSTALLATION RIG



(B) DRAIN DELIVERY ARRANGEMENT



(C) CROSS SECTION OF MANDREL AND DRAIN

BAND DRAINS - INSTALLATION

BAND DRAINS - INSTALLATION SPECIFICATIONS

CONTRACTOR SHALL EXECUTE THE WORK BY SATISFYING THE SPECIFICATIONS

Men and Materials

- ❖ Furnish all necessary labour, equipment, and materials
- ❖ Perform all operations as per the details in the drawing
- ❖ Band drain (PVD) satisfying the specifications and compatible

Execution

- ❖ Engineer incharge of the project shall locate number and stake the locations of drains.
- ❖ Install the drains at marked locations by constant load or constant rate of penetration

Difficulty in Installation

- ❖ Vibration method can be adopted in case of difficulty in penetration.
- ❖ Installation of drain at any location can be abandoned if the rate of penetration is less than 150mm/s
- ❖ Abandon the installation if obstructions encountered
- ❖ Install another drain with in 500mm distance with the approval of engineer.
- ❖ Auguring or other means of loosening the stiff upper soil is permitted
- ❖ Engineer is authorized to vary the depth and spacing incase of difficulty in installation

Quantity

- ❖ Measurement of drains in liner meter including the lengths protruding above the ground to cover by sand blanket

BAND DRAINS - INSTALLATION

GUIDELINES FOR BAND DRAIN INSTALLATION

SETTING OUT, CLEARING AND GRUBBING

- Clearing & Clearing the area
- Setting out center line and boundary lines of area
- Installation of reference pillars
- Indicating initial levels of embankments, sand blanket and pre & post settlements
- Establish Temporary bench marks at five locations

LAYING THE BOTTOM LAYER OF EMBANKMENT

- To facilitate movement of Machinery and easy draining
- Thickness - 750mm
- Compacted in layers of 200mm thick to 95% MDD (MOST Specification)

BAND DRAINS - INSTALLATION

GUIDELINES FOR BAND DRAIN INSTALLATION (contd.)

INSTALLATION OF BAND DRAINS

- Marking band drain locations as per drawing and staked
- Rig along with hydraulic stitcher is kept in Position
- Band drain reel is attached to drain delivery arrangement
- Tip of the drain is attached to the mandrel and pushed down to the mouth and attached with anchor plate
- Rig is marched to exact location and positioned vertical
- Mandrel is pushed down to the ground along with the drain
- On reaching designed depth, the mandrel is pulled back after noting the depth of penetration
- AS soon as mouth of the mandrel pulled up cut of the drain at 0.20m (minimum) above the ground level using template
- Rig is moved to next location and installation cycle is repeated

BAND DRAINS - INSTALLATION

GUIDELINES FOR BAND DRAIN INSTALLATION (contd.)

LAYING OF SAND BLANKET

- Provide a sand blanket layer of suitable thickness (Minimum 400mm as per IS 15248 –Part 2).
- Use coarse sand free from fines (Zone-I sand with fines passing 200micron size < 5%)
- Hold drains vertical with suitable support
- Connect the open drain to sand blanket layer

LAYING OF GEOTEXTILE

- Cover the top surface of sand blanket using approved geotextile as detailed in the drawing
- Before laying ensure sand surface is smooth and free from sharp objects and debris
- Follow manufacturer’s instruction while laying the geotextile.

BAND DRAINS - INSTALLATION

GUIDELINES FOR BAND DRAIN INSTALLATION (contd.)

LAYING UPPER LAYER OF EMBANKMENT AND SUBGRADE

- Construct the embankment above the blanket layer to the required height layer by layer
- Compact each layer at least to relative compaction of 95% (follow MORTH specification clause 305)
- Conduct quality control test as per agreement
- Top 500 mm thick layer (sub-grade) is to be constructed with approved material
- Sub-grade shall be compacted layer by layer satisfying MORTH Specifications
- Check the levels to ensure they are within the limits of tolerance
- Maintain the record of quality control tests.

BAND DRAINS - INSTALLATION

GUIDELINES FOR BAND DRAIN INSTALLATION (contd.)

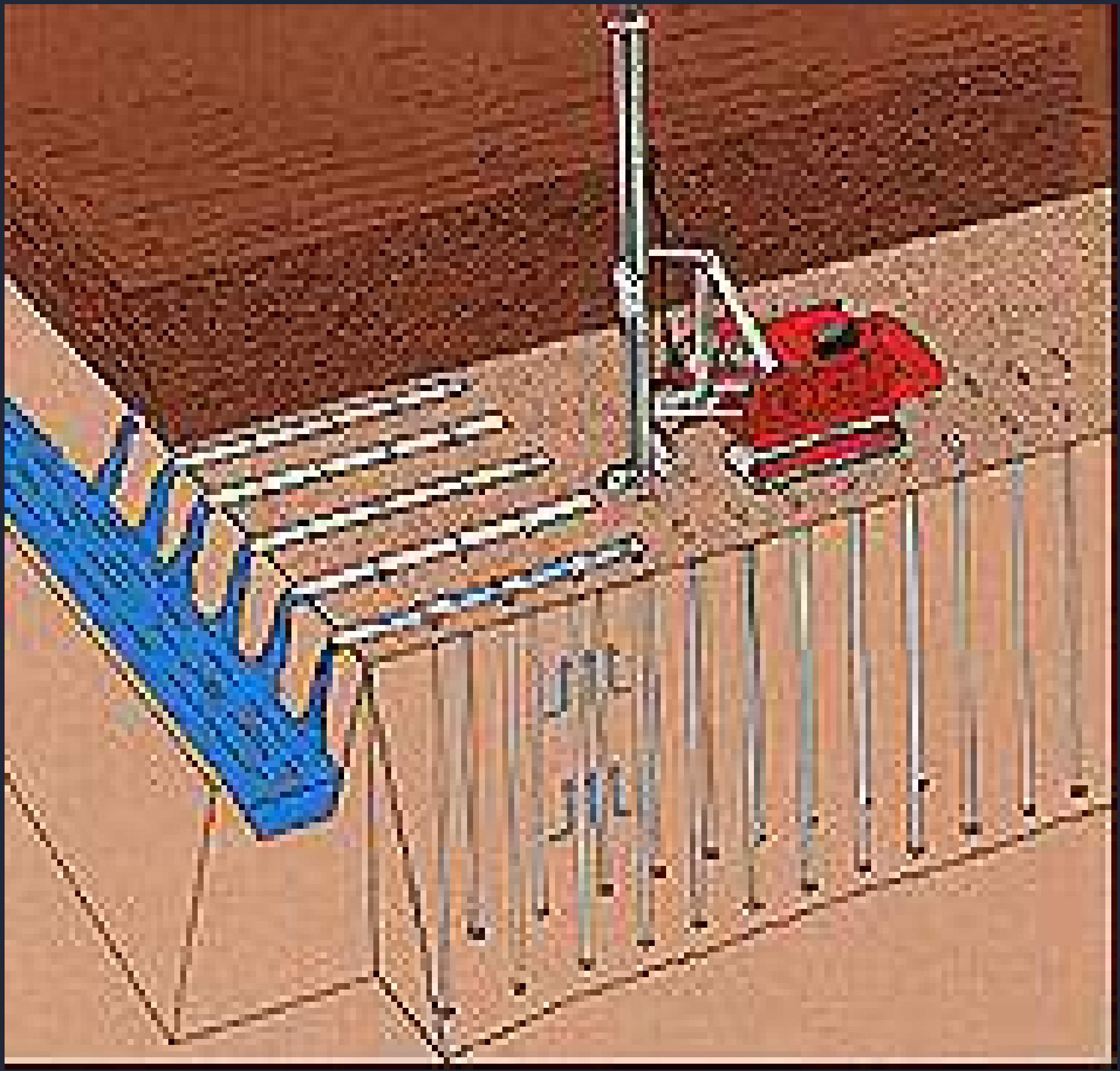
FINISHING OF FORMATION

- Finish the height, camber and slope after achieving desired degree of consolidation
- Construct the payment layers on certification from the Engineer

MONITORING

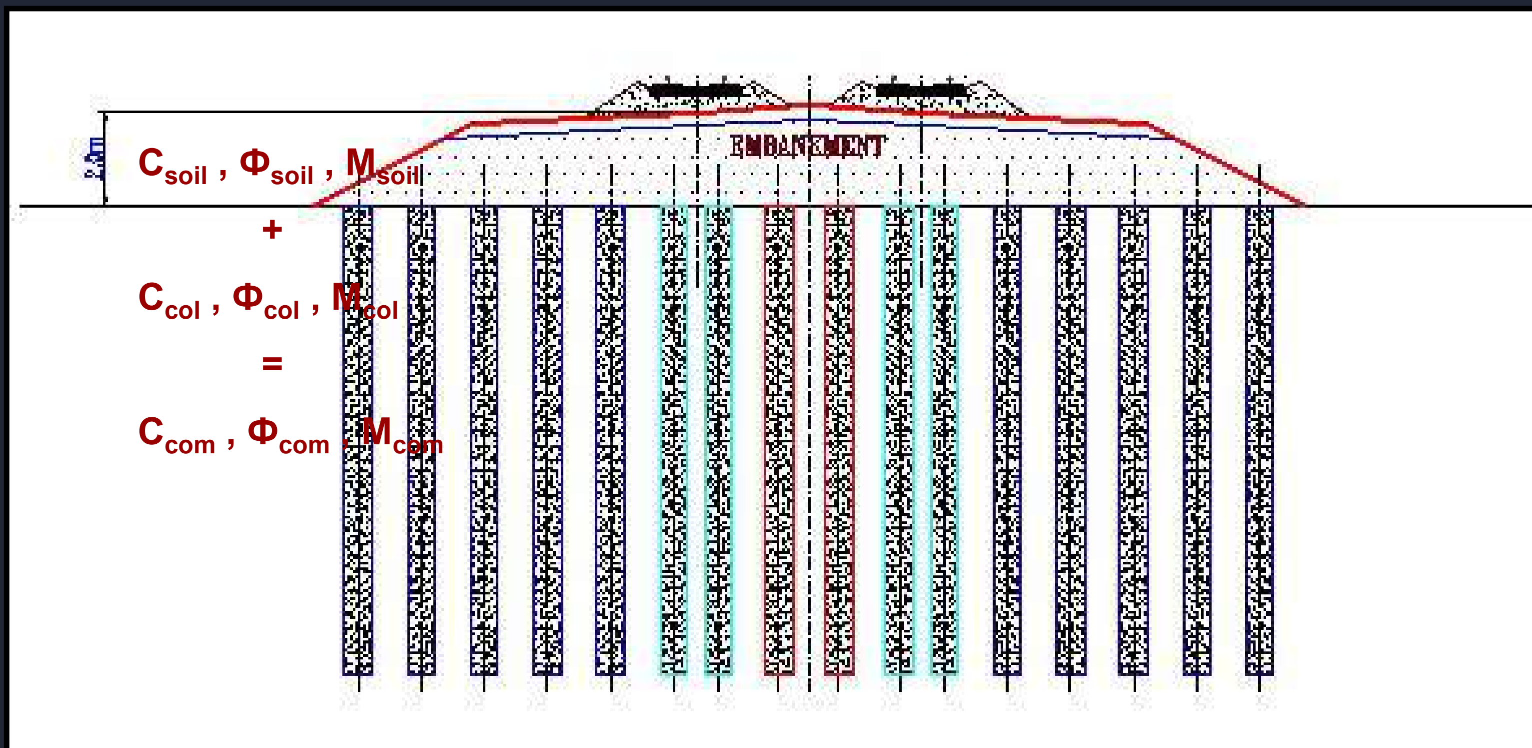
- Execute the "Monitoring Scheme" as approved
- Measure vertical settlement and lateral movement as per approved time schedule
- Measure pore water pressure as per time schedule

BAND DRAINS - COMPLETE VIEW



STONE COLUMNS- TYPOCAL LAYOUT

Embankment on Stone Column Stabilized Ground



Stone Columns

STONE COLUMNS

- Purpose
- Advantage
- Design
- Method of Installation

STONE COLUMNS - PURPOSE

STONE COLUMNS

PURPOSE

STONE COLUMNS IMPROVE THE BEARING SOILS IN FOLLOWING WAYS:

- **To Increase Bearing Capacity**
- **To Reduce Compressibility**
- **To Increase Rate of Consolidation**
- **To Increase Lateral Resistance**

STONE COLUMNS - ADVANTAGES

STONE COLUMNS

ADVANTAGES

- Stabilized ground is immediately ready to support the structure
- Suitable for structures sensitive to larger settlement
- Offers higher bearing resistance and lesser settlement than vertical drains

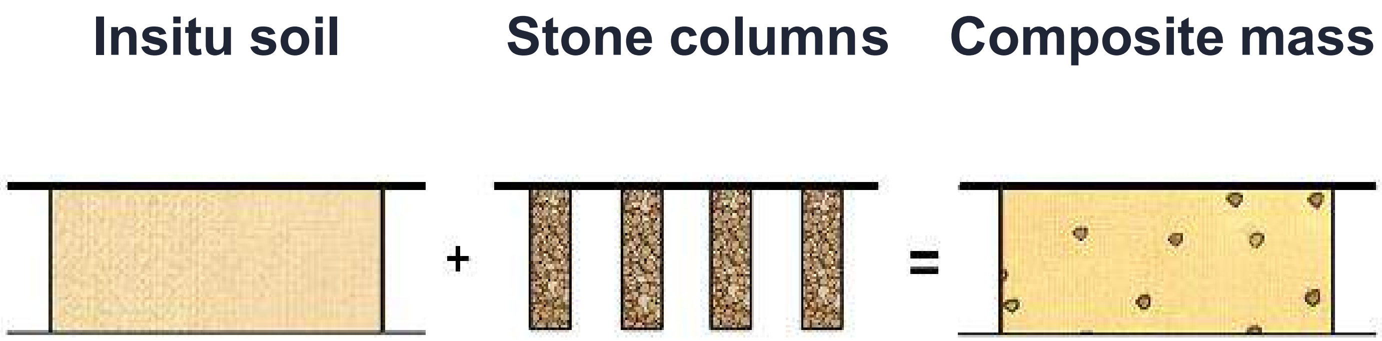
COMPARISON

COMPARISON OF ENGINEERING PRPERTIES IMPROVED

Type of granular column	Type of improvement: Increase ultimate bearing capacity	Reduce magnitude of settlement	Increase rate of settlement	Provide uplift resistance	Increase lateral resistance	Reduce liquefaction potential	Increase slope stability
Stone column	Significant	Significant	Significant	None	Moderate	Significant	Significant
Sand column	Significant	Significant	Significant	None	Some	Significant	Significant
Geopier	Significant	Significant	Significant	Significant	Significant	Significant	Significant
Gravel drain	Some	Some	Significant	None	Not applicable	Significant	Not applicable
Sand drain	Some	Some	Significant	None	Not applicable	Significant	Not applicable

STONE COLUMNS

Equivalent Soil Model



$D_s, C_u, \phi_s \quad + \quad D_c, \phi_c, d, s \quad = \quad D_e, C_{ue}, \phi_e$

Source: Keller Ground Engineering India Pvt. Ltd.

- **Results in a new composite soil having improved deformation and shear strength parameters**
- **Improved shear strength and deformation parameters are used in the stability and settlement analysis respectively**

STONE COLUMNS – DESIGN

STONE COLUMNS - DESIGN

TYPE OF COLUMNS

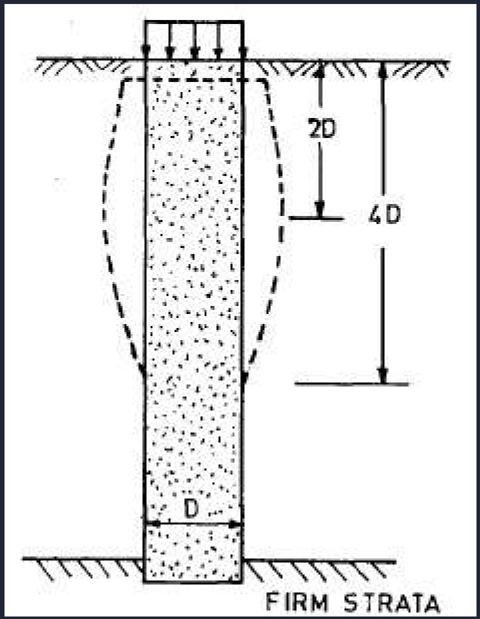
Long column (L/D > 10)

- Bearing
- Floating

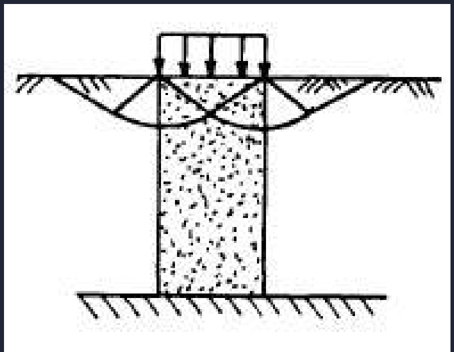
Short column (L/D < 10)

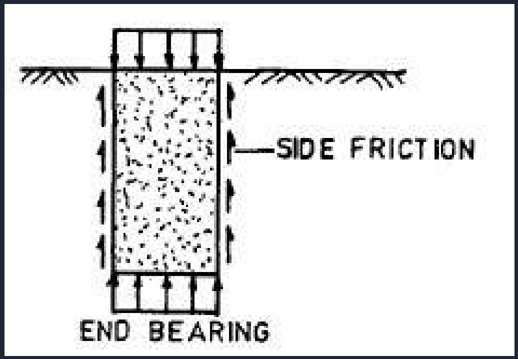
- Bearing
- Floating

MECHANISM OF FAILURE



Long column with firm or floating support - Bulging


Short column with firm support – Shear failure



Short column with floating support – Punching failure

Source: IS 15284 (Part 1): 2003

Vibro Stone Columns

Design Parameters of Stone Columns

- In situ soil
  - Cohesion
  - Friction angle
  - Compressibility
  - Depth
- Reinforcing material (stone)
  - Friction angle of material
  - Compressibility
- Column layout
  - Column diameter
  - Spacing
  - Length

CAPACITY OF STONE COLUMN

CAPACITY OF STONE COLUMN: IS 15284 (Part 1)

ESTIMATION OF LOAD CAPACITY OF A COLUMN

Load capacity of the treated ground may be obtained by summing up the contribution of each of the following components for wide spread loads, such as tankages and embankments:

a) Capacity of the stone column resulting from the resistance offered by the surrounding soil against its lateral deformation (bulging) under axial load,

b) Capacity of the stone column resulting from increase in resistance offered by the surrounding soil due to surcharge over it, and

c) Bearing support provided by the intervening soil between the columns.

CAPACITY OF STONE COLUMN

CAPACITY OF STONE COLUMN

1. Capacity based on bulging of column (IS 15284-Part 1)

$$Q_1 = \sigma_{rl} \, A_s / FS$$

where $\sigma_{rl} = k_{pcol} \, (\sigma_{r0} + 4 \, C_u)$

k_{pcol} = $\tan^2 (45 + (\phi'/2))$

ϕ' = $40°$

σ_{r0} = $k_o \, \sigma_v$

k_o = Average coefficient of lateral earth pressure for at rest condition (0.6 to 0.7)

C_u = Undrained shear strength of clay

A_{sc} = Area of stone column

S = Spacing between stone columns

CAPACITY OF STONE COLUMN

CAPACITY OF STONE COLUMN

2. Capacity Based on Surcharge Effect

$$Q_2 = (k_{pcol}\,\Delta\sigma_{r0} A_s)/FS$$

where $\Delta\sigma_{r0}$ = $q_{safe}(1+2k_0)/3$

q_{safe} = $c_u N_c/2.5$

N_c = Bearing capacity factor can be taken as 6 for circular section

A_c = $A - A_s$

A = $0.866\ S^2$ for triangular pattern

= S^2 for square pattern

A_s = Area of stone column ($\pi\ D^2/4$)

D = Diameter of stone column

CAPACITY OF STONE COLUMN

CAPACITY OF STONE COLUMN

3. Bearing support provided by the intervening soil,

$$Q_3 = q_s A_c$$

where	q_s	=	$C_u N_c$
	C_u	=	Undrained cohesion
	N_c	=	Bearing capacity factor can be taken as 6 for circular section
	A_c	=	$A - A_s$
	A	=	$0.866\ S^2$ for triangular pattern
		=	S^2 for square pattern
	A_s	=	Area of stone column ($\pi D^2/4$)
	D	=	Diameter of stone column

$$Q = (Q_1 + Q_2 + Q_3)$$

STONE COLUMNS – SPECIFICATIONS

STONE COLUMNS - SPECIFICATIONS

SPECIFICATIONS FOR

- Materials
- Construction
- Field Control
- Load Tests
- Monitoring
- Field Test

STONE COLUMNS – SPECIFICATIONS

STONE COLUMNS – Materials

MATERIALS FOR

- COLUMNS
- BLANKET

COLUMNS MATERIAL S SPECIFICATIONS (as per IS 15284 -1 & MOST)

- Crushed stone for column back fill
- Clean, hard, angular ,Chemically inert and resistance breakage
- Free from organic ,trash and deleterious materials
- Well-Graded ($C_u > 4$)
- Impact Value > 30%
- Satisfying following Gradation

Size of the Crushed Aggregate	% Passing
75 mm	90-100
50 mm	80-90
38 mm	55-75
20 mm	10-20
12 mm	5-13
2 mm	5

STONE COLUMNS – SPECIFICATIONS

STONE COLUMNS – Materials

RATING OF COLUMNS MATERIALS

SUITABILITY NUMBER

$$S_N = 1.7\sqrt{\frac{3}{(D_{50})^2} + \frac{1}{(D_{20})^2} + \frac{1}{(D_{10})^2}}$$

Range of S_N	Rating as backfill
0–10	Excellent
10–20	Good
20–30	Fair
30–50	Poor
>50	Unsuitable

BLANKET MATERIALS SPECIFICATIONS & LAYING (as per IS 15284 -1 & MOST)

Specifications :

- Gravel or Coarse sand
- Well drained
- Minimum thickness 500mm
- Relative density 75 to 80%

Laying :

- Remove deposited slush on the ground to a depth of 500mm (Minimum)
- Lay granular blanket and compacted as per specifications
- Blanket should exposed to atmosphere

STONE COLUMNS – CONSTRUCTION

STONE COLUMNS – Construction

METHODS OF IN STALLATION (IS 15284 –Part 1)

BORING AND RAMMING TECHNIQUE

Non-Displacement Method

- Indigenous Method
- Special equipment not required
- Requires bored piling equipment
- Rammer for compaction

VIBRATORY TECHNIQUE

Vibro – Displacement Method

Vibro- Replacement Method

MACHINERY

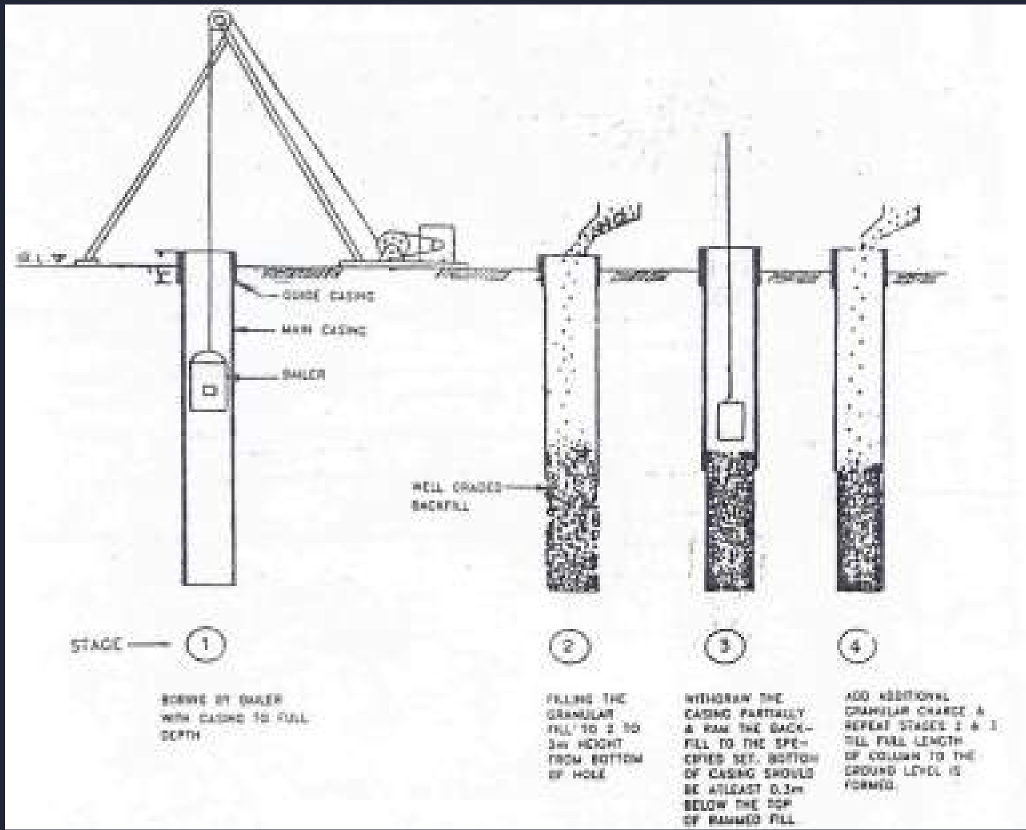
- Vibrofloat (Poker)
- Rig equipped with Crane power supply
- Water Pump
- Loader cum Dozer
- Compacting Roller

NON DISPLACEMENT METHOD

NON DISPLACEMENT METHOD

OPERATION

1. Boring by bailer and casing
2. Filling Gravel to 2-3m thick
3. Withdraw casing partially and ram the backfill to the specified set
4. Add additional granular charge and repeat step 3



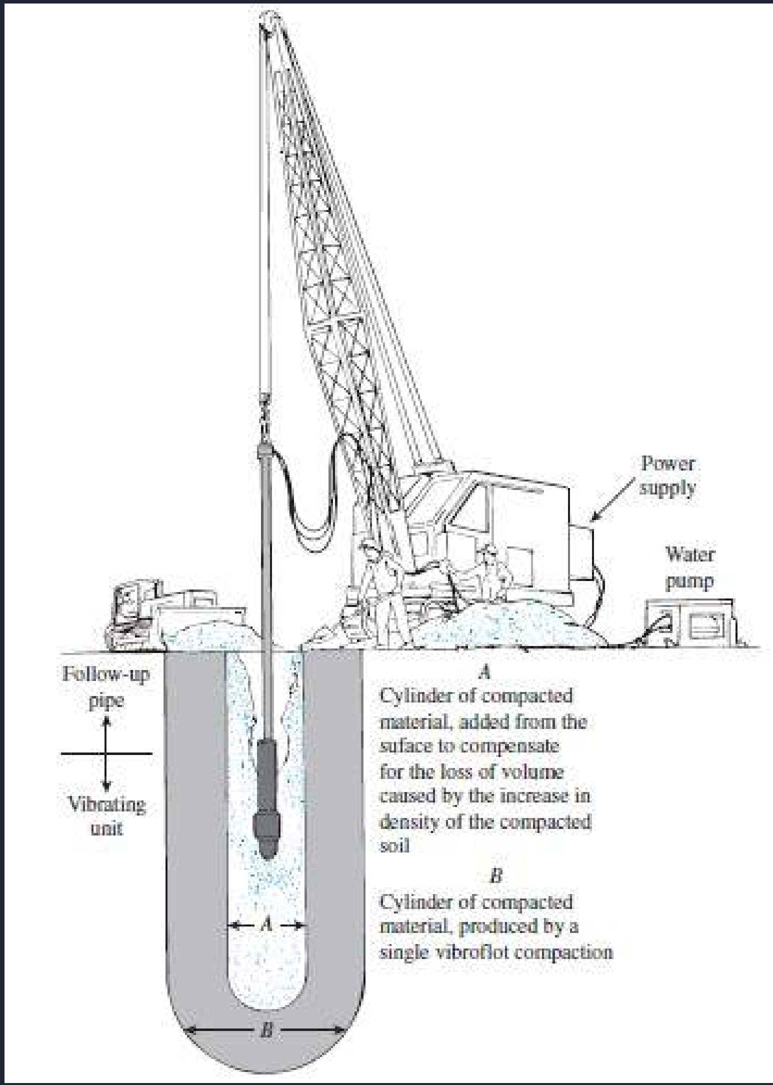
Source: IS 15284 (Part 1): 2003

STONE COLUMNS – VIBRATORY TECHNIQUE

STONE COLUMNS – Construction

MACHINERY

- Vibrofloat (Poker)
- Rig equipped with Crane power supply
- Water Pump
- Loader cum Dozer
- Compacting Roller

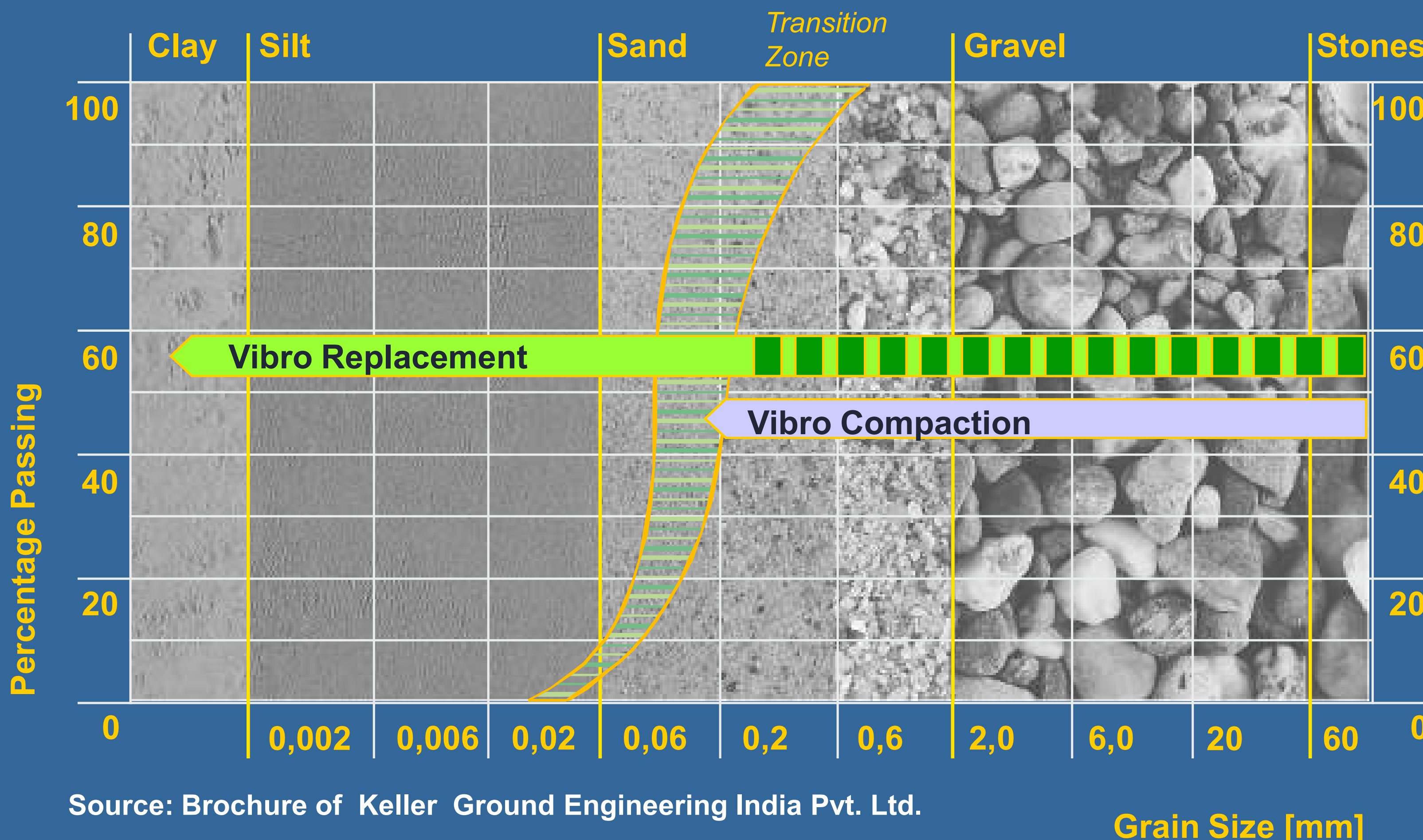


Source: Brown, 1976

Vibro Techniques

Vibro Techniques

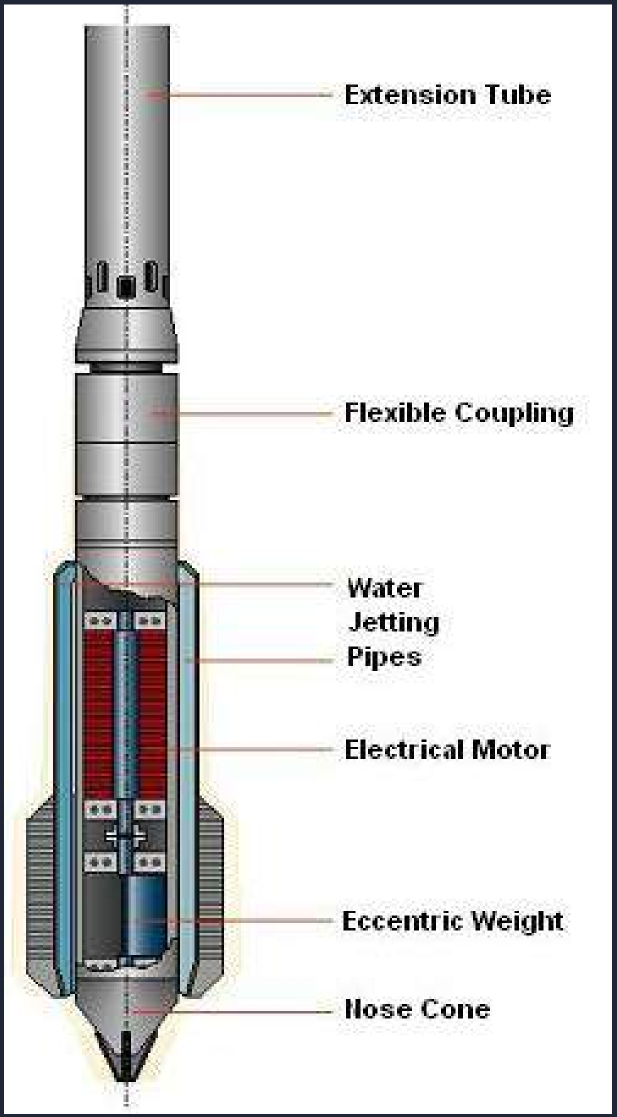
- **Vibro Compaction**
- **Vibro Replacement (Stone Columns)**



Source: Brochure of Keller Ground Engineering India Pvt. Ltd.

Vibro Compaction

Depth Vibrator S 300

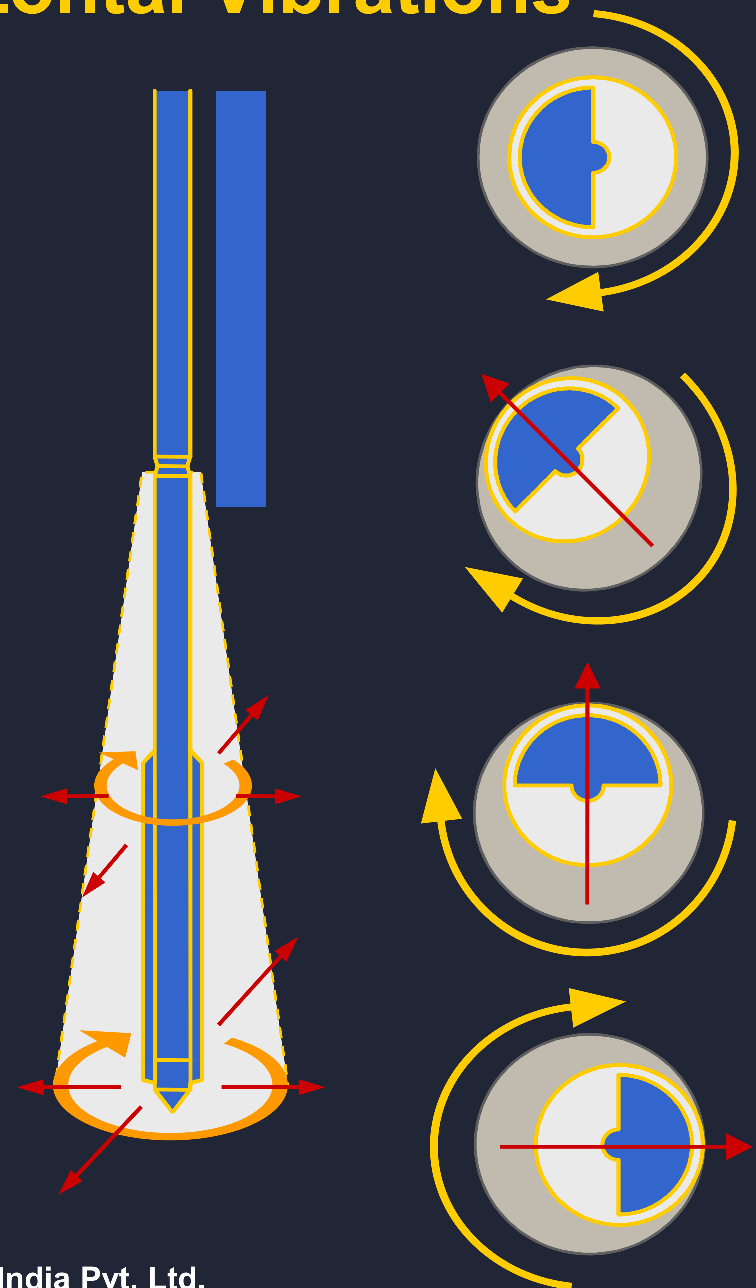


Source: Brochure of Keller Ground Engineering India Pvt. Ltd.

Vibro Compaction

Generation of Horizontal Vibrations

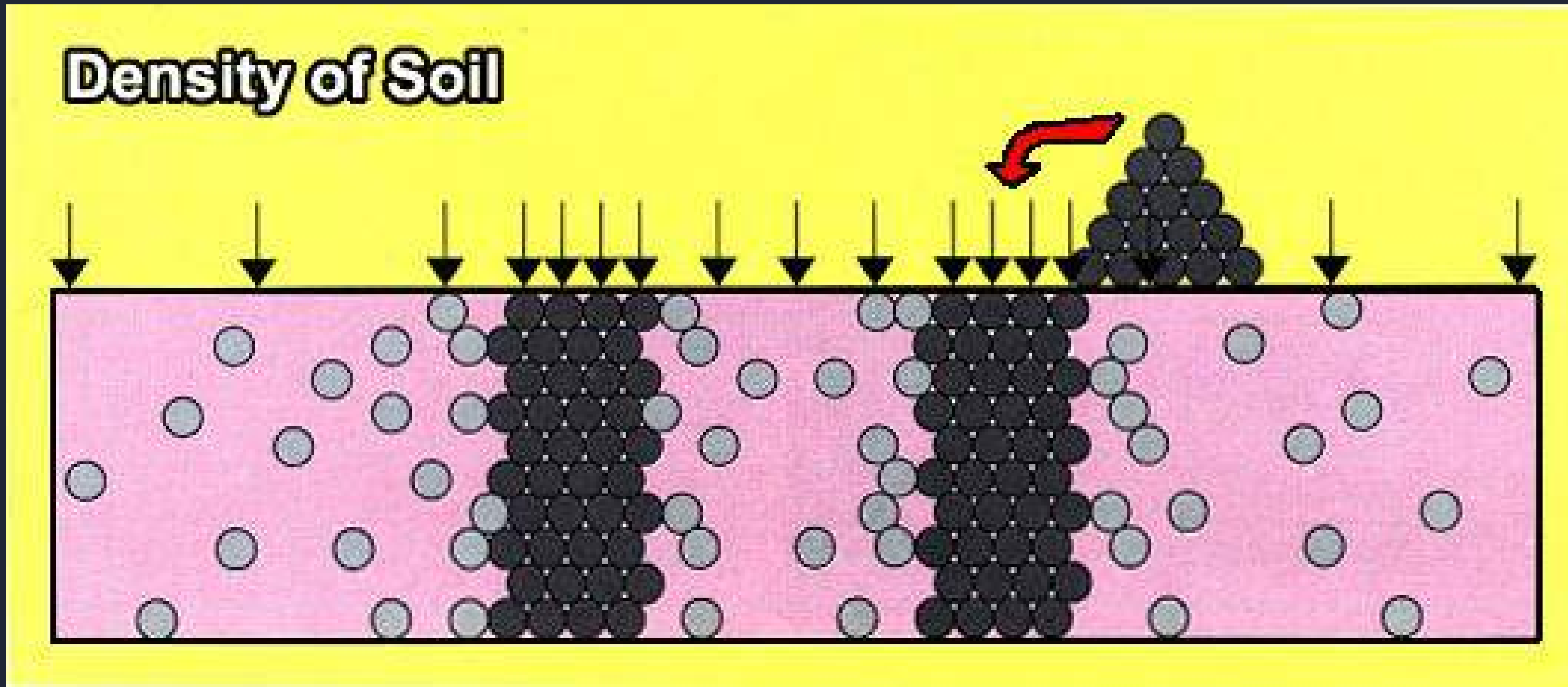
Details of S-300 Depth Vibrator	
Diameter of Vibrator	400 mm
Diameter of Vibrator Incl. water jetting pipes	765 mm
Height of Vibrator	2900 mm
Weight of Vibrator	2.6 t
Centrifugal Force	30 t
Frequency of Vibration	30 Hz
Amplitude of Vibration	25 mm
Motor Capacity	150 kW
Motor Voltage	380 V
Motor Speed	1775 rpm



Source: Brochure of Keller Ground Engineering India Pvt. Ltd.

Vibro Stone Columns

Vibro Replacement (Stone Columns)



Source: Brochure of Keller Ground Engineering India Pvt. Ltd.

Increases overall stiffness of compressible soils

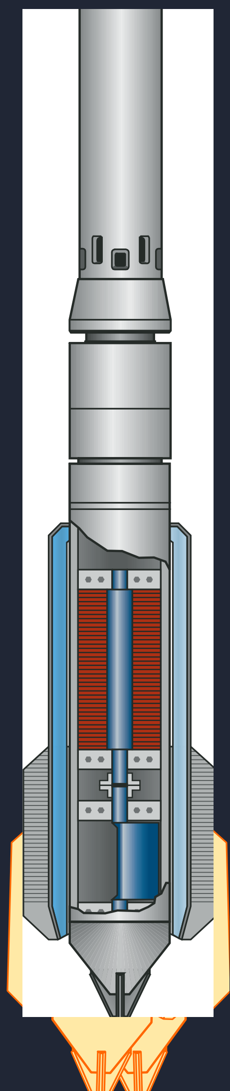
Increases shear strength

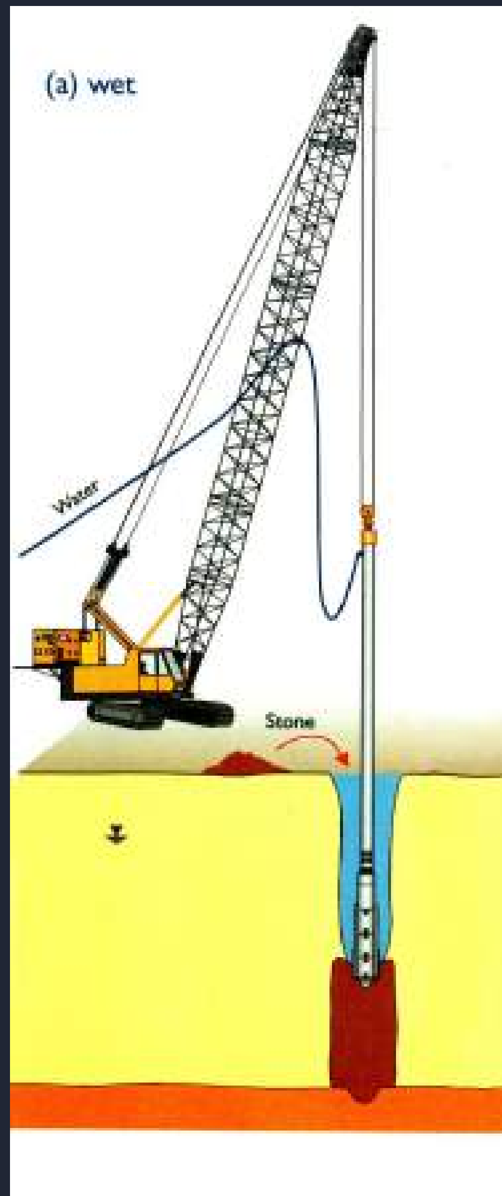
Allows rapid consolidation by providing drainage

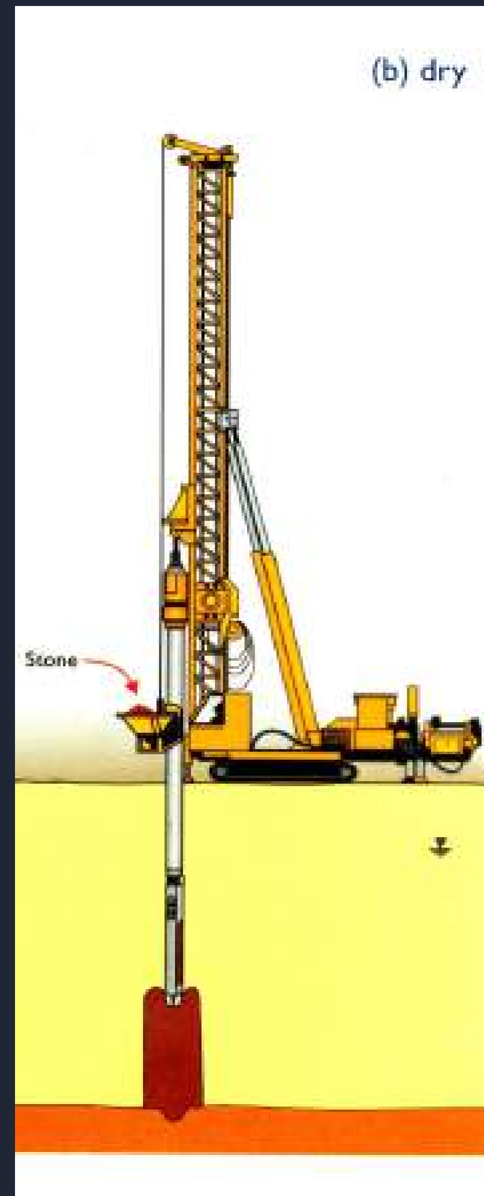
Vibro Stone Columns

VIBRATORS USED FOR STONE COLUMN INSTALLATION

Mono





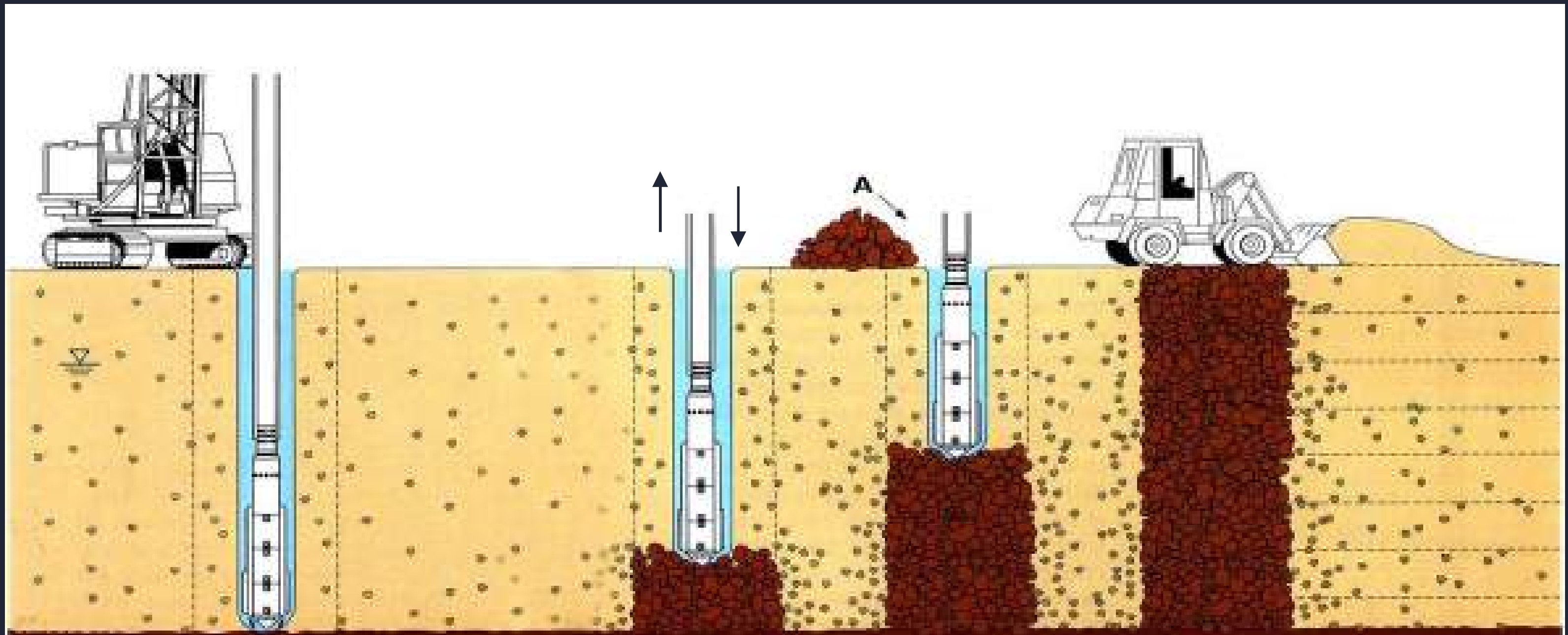


Beta

Source: Brochure of Keller Ground Engineering India Pvt. Ltd.

Vibro Stone Columns

Wet Method – Top Feed Method

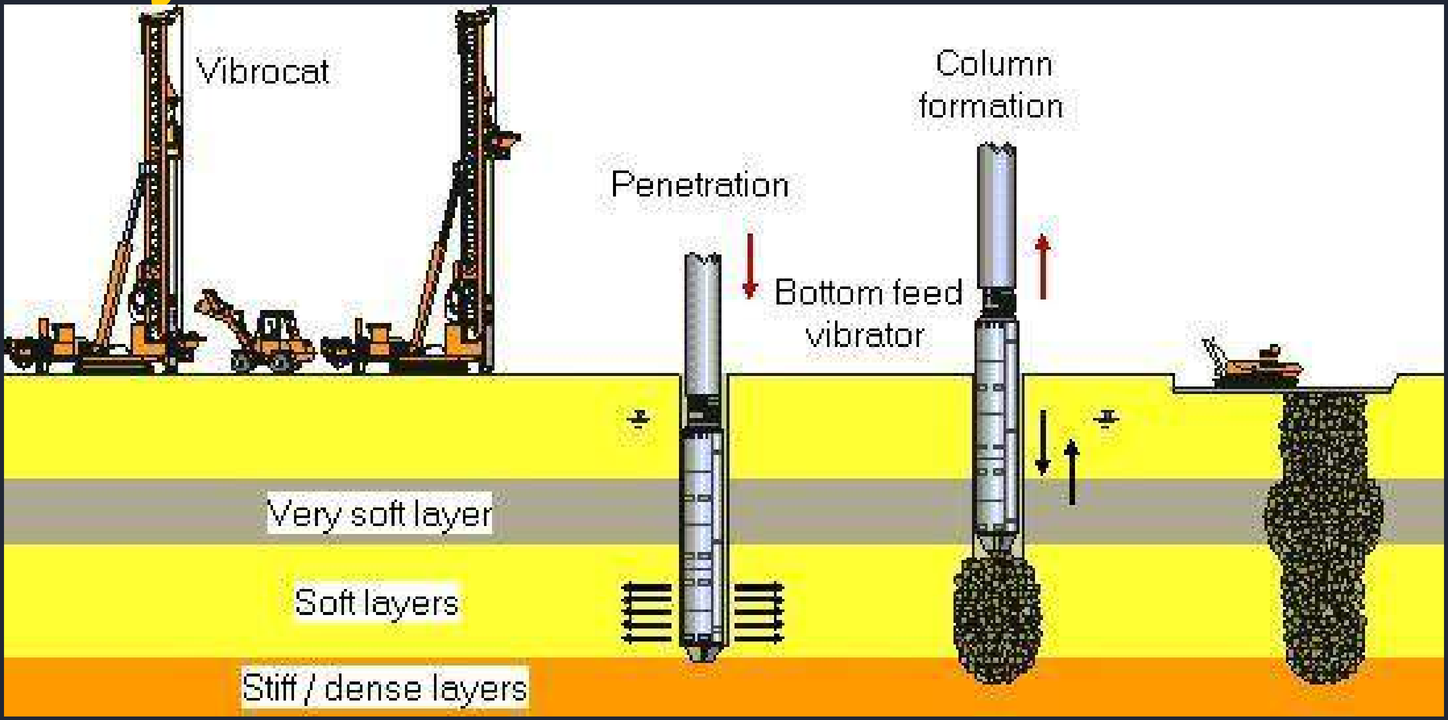


Source: Brochure of Keller Ground Engineering India Pvt. Ltd.

- Uses water jets - at a rate of 25 lit/sec
- Generally crane hung
- Penetration by jetting & vibratory force
- Stone feeding from top through borehole

Vibro Stone Columns

Dry Method – Bottom Feed Method



Source: Brochure of Keller Ground Engineering India Pvt. Ltd.

- No water jetting
- Custom built machines- Vibrocats / crane hung
- Penetration by pull down thrust & vibratory force
- Stone feeding from bottom through vibrator
- Maximum depth of treatment ~ 16m

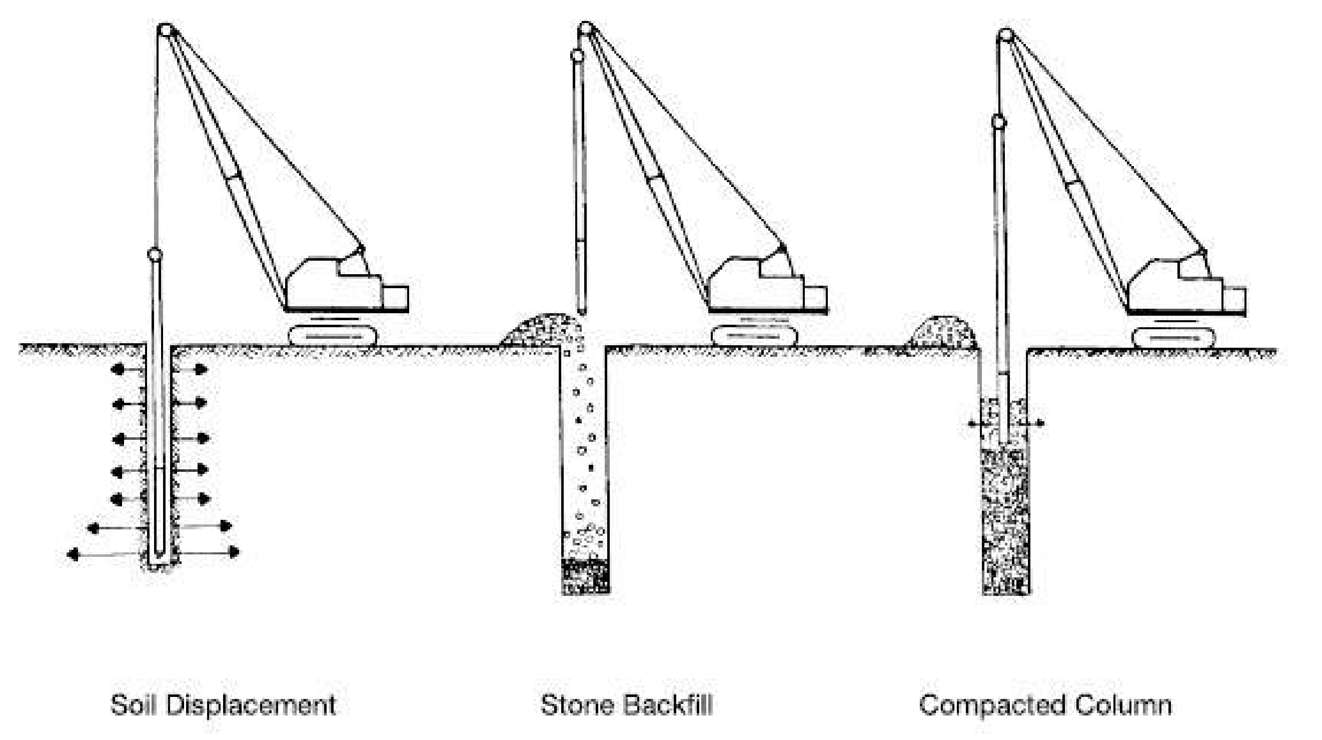
Source: IS 15284 (Part 1): 2003

VIBRO DISPLACEMENT METHOD

VIBRODISPLACEMEMT METHOD

OPERATION

1. Vibrating probe displaces the soil Laterally
2. Probe is withdrawn from hole affter reaching desired depth
3. Backfill is droped in the annular hole
4. Probe is lowered again to displace the stone laterally an downward
5. Process is repeated in steps to form compacted column

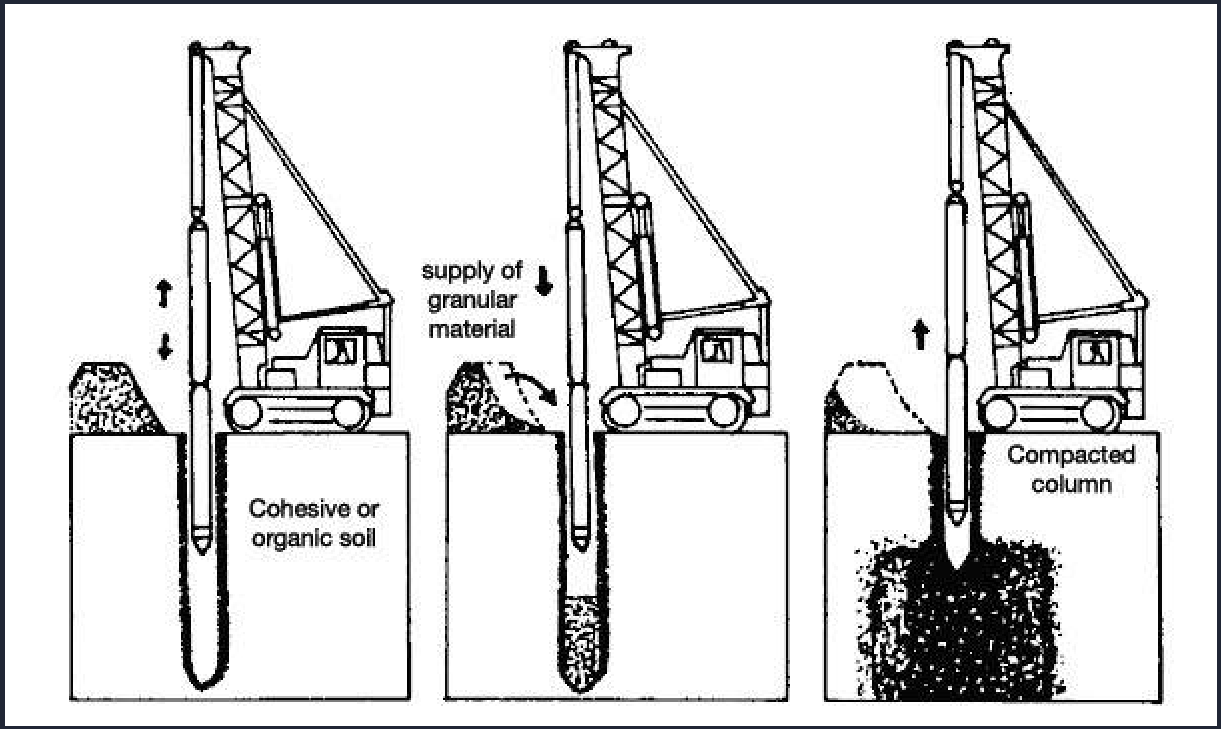


VIBRO DISPLACEMENT METHOD

VIBRODISPLACEMEMT METHOD

OPERATION

1. Hole is created by the probe by water jetting and flushed
2. Granular material is added in increment through the annular space
3. Stone is compacted in about lifts of 0.5m to1m
4. Compaction is achieved by vibration and ramming



STONE COLUMNS – FIELD CONTRIL

STONE COLUMNS – FIELD CONTROL

NON-DISPLACEMENT METHOD

SET CRITERIA

Hammer Energy : 20kNm
10mm for lat 5 blows

CONSUMPTION OF FILL

Measure diameter of column as formed during trial test

VIBRATORY TECHNIQUE

a) Vibrofloat penetration depth including the depth of embedment in firm strata.

b) Monitoring of volume of backfill added to obtain an indication of the densities achieved, and

c) Monitoring of ammeter or hydraulic pressure gauge readings to verify that the maximum possible density has been achieved in case of Vibrofloated columns.

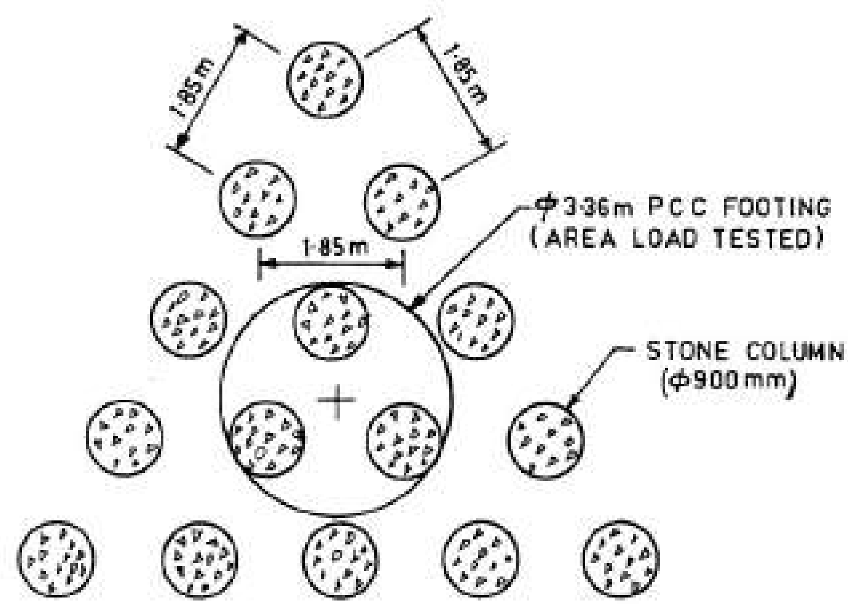
STONE COLUMNS – LOAD TEST

STONE COLUMNS – LOAD TEST (IS 15284 – Part 1)

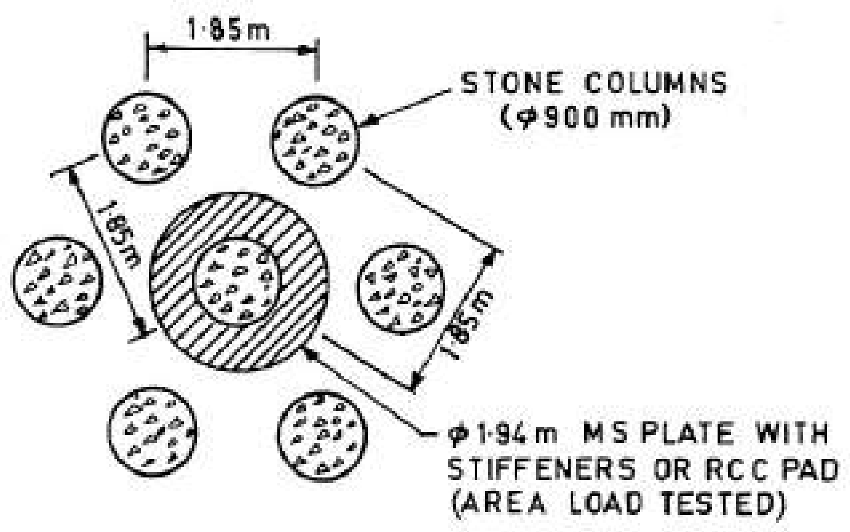
INITIAL LOAD TEST

Single Column : 1 Test for 500 or Part thereof
Three columns : 1 Test for 1000 or Part thereof
Max. Load : 1.5 times design load
Kentledge : 1.3 times max. load (Min.)

GROUP OF COLUMN TEST ARRANGEMENT



SINGLE COLUMN TEST ARRANGEMENT



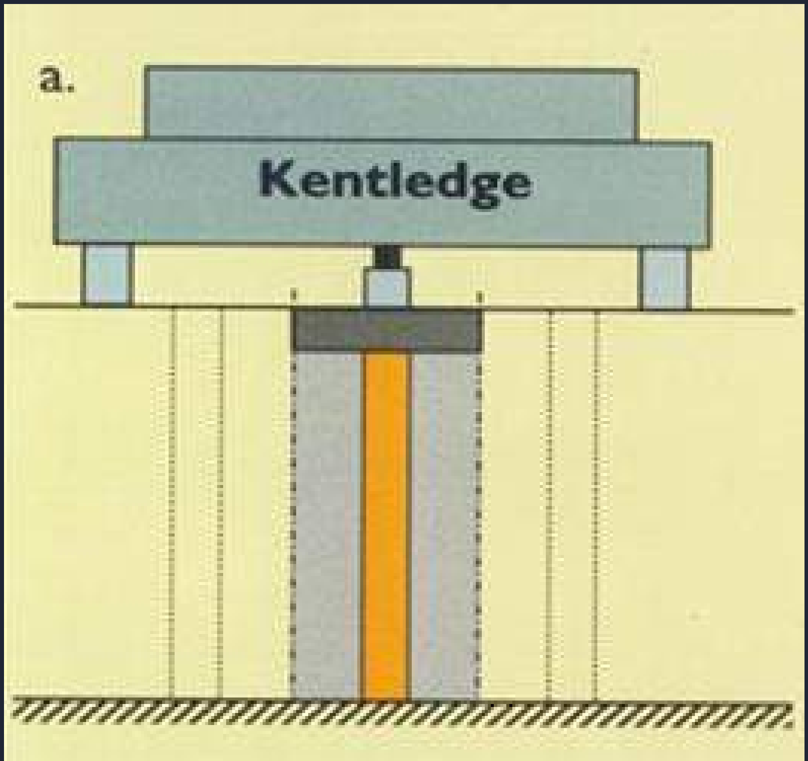
ROUTINE LOAD TEST

Single Column : 1 Test for 500 or Part thereof
Three columns : 1 Test for 1000 or Part thereof
Max. Load : 1.3 times design load
Kentledge : 1.1 times max. load (Min.)

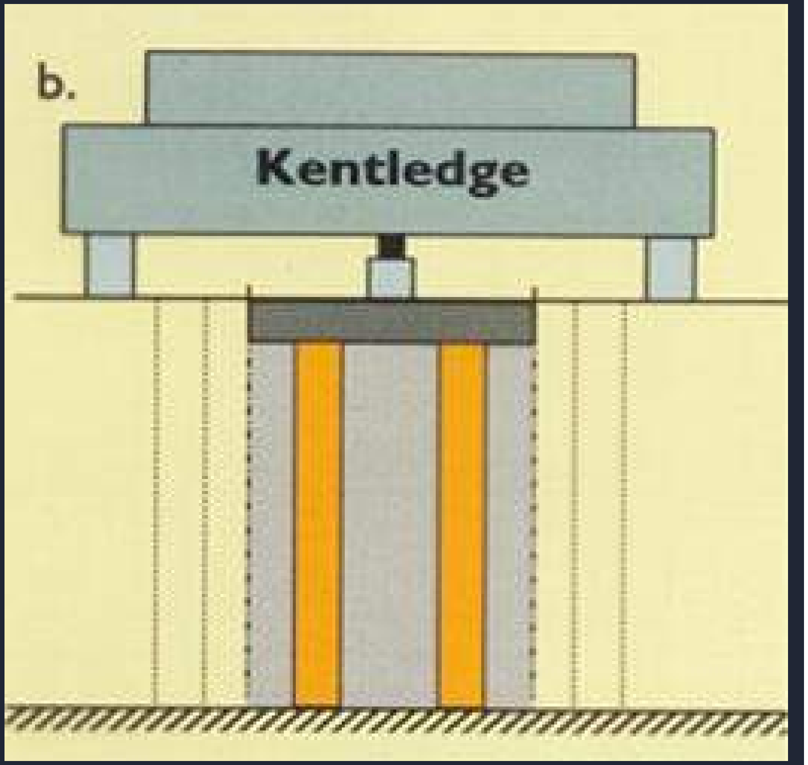
Vibro Stone Columns

Load Test Arrangement

Single Column Testing



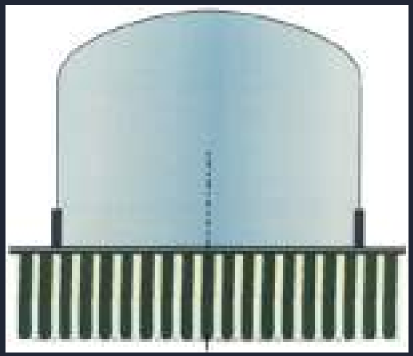
Group Column Testing



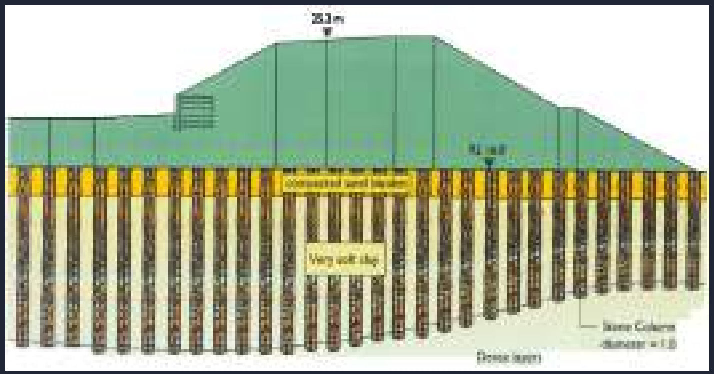
Source: Brochure of Keller Ground Engineering India Pvt. Ltd.

Stone Columns - Applications

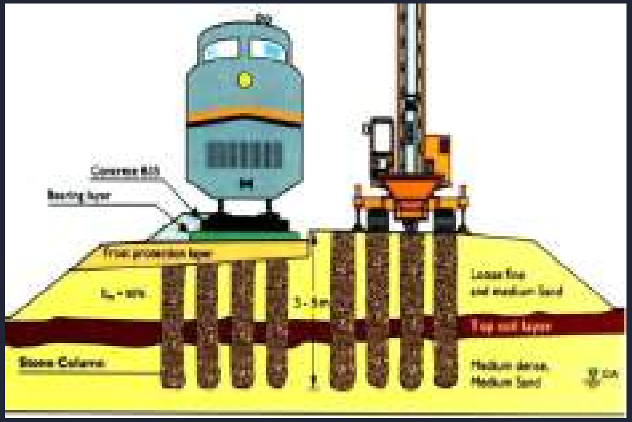
Typical Applications



Tank foundations



Highway embankments



Railway embankments

Source: Brochure of Keller Ground Engineering India Pvt. Ltd.

GROUND IMPROVEMENT

CONCLUSIONS

- Guidelines for Stabilizing soft clay deposit are presented.
- Comply with the specifications of IS 15284 (Part 1 & 2) and MOST.

GROUND IMPROVEMENT

REFERENCES

1. MOST Specifications (700-Geosynthetics), MORTH, Published by IRC.
2. IS 15284 (Part 1): 2003, "Design and Construction for Ground Improvement –Guidelines: Part 1 Stone Columns", BIS, New Delhi.
3. Brown,R.E., (1976), "Vibration Compaction of Granular Hydraulic Fills", ASCE, "National Water Resources and Ocean Engineering Convention , PP 1-30.
4. Das.B.M., (2004), "Principles of Foundation Engineering (Fifth Edition)", Brooks/Cole Publishing Company.
5. Brochure of Keller Ground Engineering Pvt. Ltd., Chennai.

TWO DAY WEBINAR ON DRAFT GUIDELINES ON GEO-SYNTHETICS FOR COASTAL PROTECTION AND PORT WORKS

10.12.2020 & 11.12.2020

GABION MATTRESS REVETMENT BEHIND JETTY AT HAZIRA PORT

PRESENTED BY :

Prof. NILANJAN SAHA

DEPARTMENT OF OCEAN ENGINEERING,

IIT MADRAS.

INTRODUCTION

- Reliance Industries Limited (RIL) commissioned their Hazira Manufacturing Division (HMD) on the banks of River Tapi.
- Due to tidal variations and floods during monsoon, around 940m has been observed to be under constant erosion.
- A necessary and suitable shore protection is to be adopted to prevent erosion and restrict ingress of flood water into HMD plant area.
- The novel construction methodology (as per site requirements) using gabion mattress revetments was proposed.
- Challenge was to execute the revetment behind two operational jetties.

SITE LOCATION

- The Hazira Manufacturing Division (HMD of Reliance industries ltd,) is located 25 km north west of Surat city along the river Tapi.
- The Oil Jetty I & II lies along the meandering area of the Tapi river were the current velocity is 1.8 m/s.

LAYOUT OF THE SITE

CAUSES FOR FAILURE OF EXISTING SHORE PROTECTION WORKS

- Non engineered shore protection was carried out using construction debris over in irregular slope.
- The shore is continuously exposed to tidal variation and storm surge.
- The Top surface of failed embankment was constructed using locally available soil which is silty clay.
- The traditional way of embankment protection is in-effective due to the drain cuts and river flow.
- No proper road access due to presence of LPG pipelines and hence periodic maintenance could not be done.

Problems at site

Problems at site

Problems at site

Meteorological data

- Tidal Elevation

Tidal level	[+m CD]
Mean High Water Spring	(+) 4.1 m
Mean High Water Neap(MHWN)	(+) 3.3 m
Mean Sea level(MSL)	(+) 2.3 m
Mean Low Water Neap(MLWN)	(+) 1.3 m
Mean Low Water Spring(MLWS)	(+) 0.4 m

- Water current As per the survey report, the maximum current velocity is in the order of 1.8 m/s (about 4 knots).

Meteorological data

Wind data

Month	Maximum Wind Velocity (kmph)/Direction	
	Morning	Evening
January	25-30/SW	40-45/NE
February	15-20/SW,NW	35-40/SW
March	25-30/SE,SW	35-40/SW
April	25-30/SW	35-40/SW
May	35-40/SW,NW	40-45/SW,NW
June	40-45/SW,SE	55-60/SW,NE
July & Aug	35-40/SW	45-50/SW
September	55-60/SW	25-30/SW
October	15-20/NE,SW	35-40/SW
November	25-30/SW	25-30/SW,NE
December	30-35/NE	30-35/SW

Bore log - Bore hole number 5

ANNEXURE II

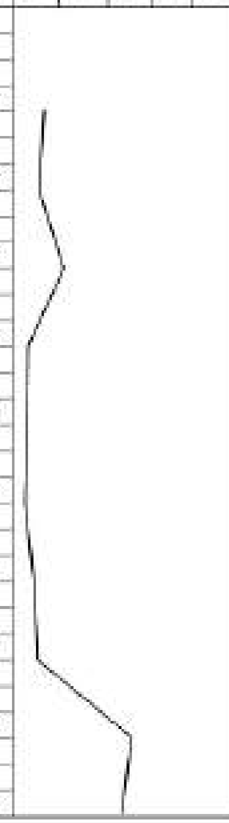
INDOMARINE SOLUTIONS

#24, [illegible] Nagar, [illegible], Chennai-600 097

Bore log

Name of work	Soil Investigation & Bathymetry Survey at Reliance Industries Limited, Hazira, Surat.					
Location					Sheet	1
Bore Number	G.L.		Bore type	Bore dia	Ground water level	
5			Calyx	150 mm	5.50m	
Work commenced on:	11.01.2020	Work completed on :	12.01.2020			

Depth from G.L. (m)	Thickness of Layer (m)	Soil Profile	Visual description of strata	Sample details: Depth	Sample details: Type	SPT - Details: Depth	0-15 Cm	15-30 Cm	30-45 Cm	N Value	Graphical Representation of N value (20, 40, 60, 80, 100)	Core Drilling: Run From	Run To	% C.R.R. (%)	% R.Q.D (%)
			Blackish medium sand, trace silt	1.50	SPT	1.50	5	7	7	14					
3.00	3.00			3.00	SPT	3.00	4	5	7	12					
			Blackish silty medium sand	4.50	SPT	4.50	2	7	15	22					
6.00	3.00			6.00	SPT	6.00	2	3	3	6					
			Brownish clayey silty sand	7.50	SPT	7.50	2	3	3	6					
9.00	3.00			9.00	SPT	9.00	2	3	3	6					
10.50	1.50		Brownish clayey sandy silt	10.50	SPT	10.50	2	4	4	8					
			Brownish clayey silty sand	12.00	SPT	12.00	3	4	5	9					
13.50	3.00			13.50	SPT	13.50	10	22	30	52					
15.00	1.50		Blackish medium sand, trace silt	15.00	SPT	15.00	13	19	25	45					



Properties considered for Soil

Depth (m)	Soil	N (SPT value)	c (kN/m²)	Φ (deg)	γ_{unsat} (kN/m²)	γ_{sat} (kN/m²)	E (kN/m²)
0-3	SP-SM	21	1	33	16	18.74	50*10^3
3-6	SP-SM/SM	20	1	33	16	17.37	50*10^3
6-9	SC-CH	6	37.28	0	16	21.78	20*10^3
9-10.5	SC-CH	6	37.28	0	16	21.78	20*10^3
10.5-13.5	SC-CH	9	51.99	0	16	22.86	20*10^3
13.5-15	SM	30	1	36	17	18.89	60*10^3
>15	SM	27	1	35	17	19	50*10^3

Comparison of soil parameters considered

Depth (m)	Previous design		Latest design	
	Soil parameters	Description	Soil parameters	Description
0-3	N = 13, C = 0 T/m^2 Φ = 28^0	Clayey silty sand	N = 21, C = 0 T/m^2 Φ = 33^0	Dense medium sand
3-6	N = 2-3, C = 2.2 T/m^2 Φ = 0^0	Silty clay	N = 20, C = 0 T/m^2 Φ = 33^0	Dense medium sand
6-9	N = 12-52, C = 0 T/m^2 Φ = 32^0	Clayey silty sand	N = 6, C = 3.8 T/m^2 Φ = 0^0	Soft clay
9-10.5	N = 12-52, C = 0 T/m^2 Φ = 32^0	Clayey silty sand	N = 6, C = 3.8 T/m^2 Φ = 0^0	Soft clay
10.5-13.5	N = 12-52, C = 0 T/m^2 Φ = 32^0	Clayey silty sand	N = 9, C = 5.3 T/m^2 Φ = 0^0	Soft clay
13.5-15	N = 12-52, C = 0 T/m^2 Φ = 32^0	Clayey silty sand	N = 30, C = 0 T/m^2 Φ = 33^0	Dense medium silty sand
15-20	N = 12-52, C = 0 T/m^2 Φ = 32^0	Clayey silty sand	N = 27, C = 0 T/m^2 Φ = 33^0	Dense medium silty sand

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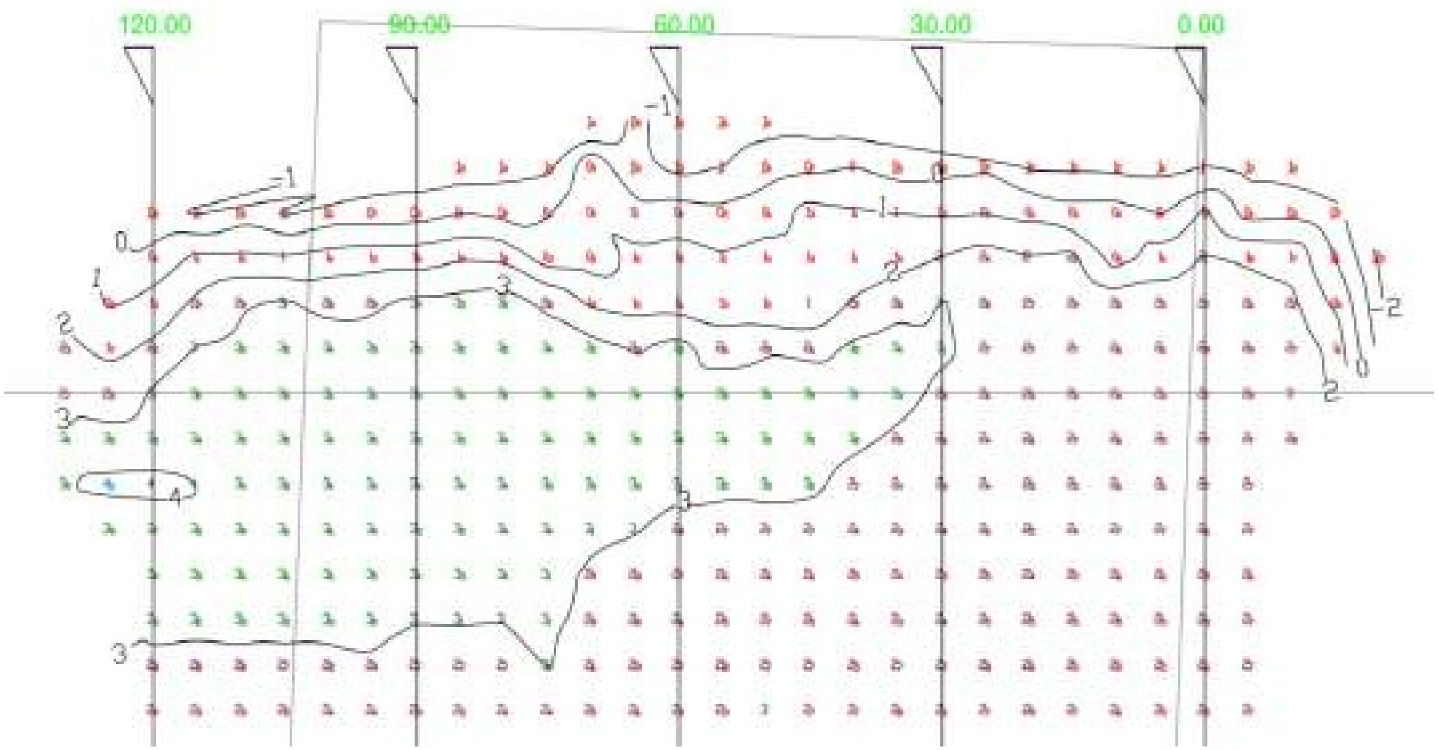
Bathymetric survey



- Maximum depth = 4.2m below CD and depth varying from 0m contour to 3m contour.

Revetment Thickness considered

Revetment Thickness Calculation As per IRC: 89-1997 In put parameters (from client):

Assumed bed Level	= 0.00m
HTL/HFL	= 4.79 m (from provided report)
Total depth of water	= 4.80m
Required free board	= 1.5m
Total height of the protection work	= 6.30 m
Maximum Velocity, u	= 3.0 m/sec
Unit weight of stones, γg	$=25 kN/m^3$
Unit weight of Water, γw	$= 10 kN/m^3$

Considering these factors, the revetment is designed as given in the following section.

Design

The expression given by IRC: 89-1997 for thickness of Gabion revetment

$$t = \frac{v^2}{2g(Sm - 1)}$$

Where,

t = Thickness of Revetment (m)

v = maximum velocity

g = ground acceleration

S_m = Specific gravity of Mattress

$$e = \frac{0.25+0.0684}{(d_{50})^{0.21}}$$

Where

d50 = Mean diameter of stone filled in the gabions = 150mm

e = $(0.245+0.0684) / (150)^{0.21}$ =0.11

Sm = 2.65 x (1-0.11) = 2.35

Substituting the values,

$$t = \frac{3^2}{2*9.81(2.35-1)} = 0.34 \text{ m}$$

According to Section 6.1.1.1 of FHWA HEC 11, the minimum thickness of mattress is 0.3m. As per 4.3 of HEC 11 thickness should be increased by 1.50 times as the mattress is placed underwater to provide for uncertainties associated with placement.

T = 1.5xDn

T = 0.51 = 0.6 m

Hence provided revetment thickness 0.6m along the slope will be safe

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Scour Depth (using Lacey's formula)

Depth of Scour ,$R = 1.35\ (q^2/f)^{1/3}$

Where

Q = Discharge Intensity in cumecs per meter width

q = Mean velocity x Depth of the water

$q = 3 \times 4.8 = 14.4\ m^3/s\ /\ m$ width

f = Silt factor $= 1.76\sqrt{m_r}$,

m_r = weighted mean diameter of the bed material = 0.228 mm

$f = 1.76\sqrt{0.20}$

$= 0.787$

$R = 1.35\ (14.4^2/0.787)^{1/3}$

$= 8.65$ m

Mean Depth of Scour (below bed level, considering water depth=6m), $R_m = 8.65 - 4.8 = 3.85$m

Maximum anticipated scour depth = $1.5 \times R_m = 1.5 \times 3.85 = 5.78$ m

Scour Depth considered

- The scour depth of 6.0m will be considered for detailed analysis, assuming discharge intensity as 14.5m^3/s/m width (considering depth of water as 5m and maximum velocity as 3m/s).
- Hence if the discharge goes beyond 14.5m^3/s/m width, the design may not hold good and erosion might take place, in spite of toe protection.

Note : As per Bathymetric survey report, the maximum depth observed in the survey area is 4.2m below CD and maximum current velocity is 1.8m/s.

Discharge capacity of Tapi river

Table: Discharge Capacity of Tapi River Basin

S No	Year	Annual rainfall data (mm)	Maximum discharge capacity (cumecs)	Remarks
1	1995	1364	16873.5	Last 25 years discharge
2	1996	1125.4		
3	1997	1031.4		
4	1998	1432		
5	1999	954		
6	2000	785		
7	2001	1180		
8	2002	1129		
9	2003	1730		
10	2004	1962		
11	2005	1894		
12	2006	1418		
13	2007	1940		
14		1962		
15	2019	1143	9830	Maximum discharge carrying capacity is decreased in course of time
Ref:	https://www.suratmunicipal.gov.in/TheCity/WeatherMonthlyReport			
NB	1) The river hydraulic data are very useful for analysis, narrowing of the Tapi river is revealing one. 40 years back it could carry 28310 m3/s (10 Lakhs Cusec) of water. It has been reduced to 9910 m3/s (3.5 Lakhs Cusecs). Analysis shows that after the flood of 2006 safe carrying capacity of river near Surat is reduced to 4531 m3/s to 5660 m3/s (1.60 to 2.0 Lakhs Cusecs).			
	Ref: Calculating Discharge Carrying Capacity of River Tapi, Patel Chandresh G., Dr. P. J. Gundaliya, Department of Civil Engineering,U V P C E, Kherva, Ganpat University.			
	https://pdfs.semanticscholar.org/c5ef/0a17a9af086d0e47c237e1d27041167c8028.pdf			

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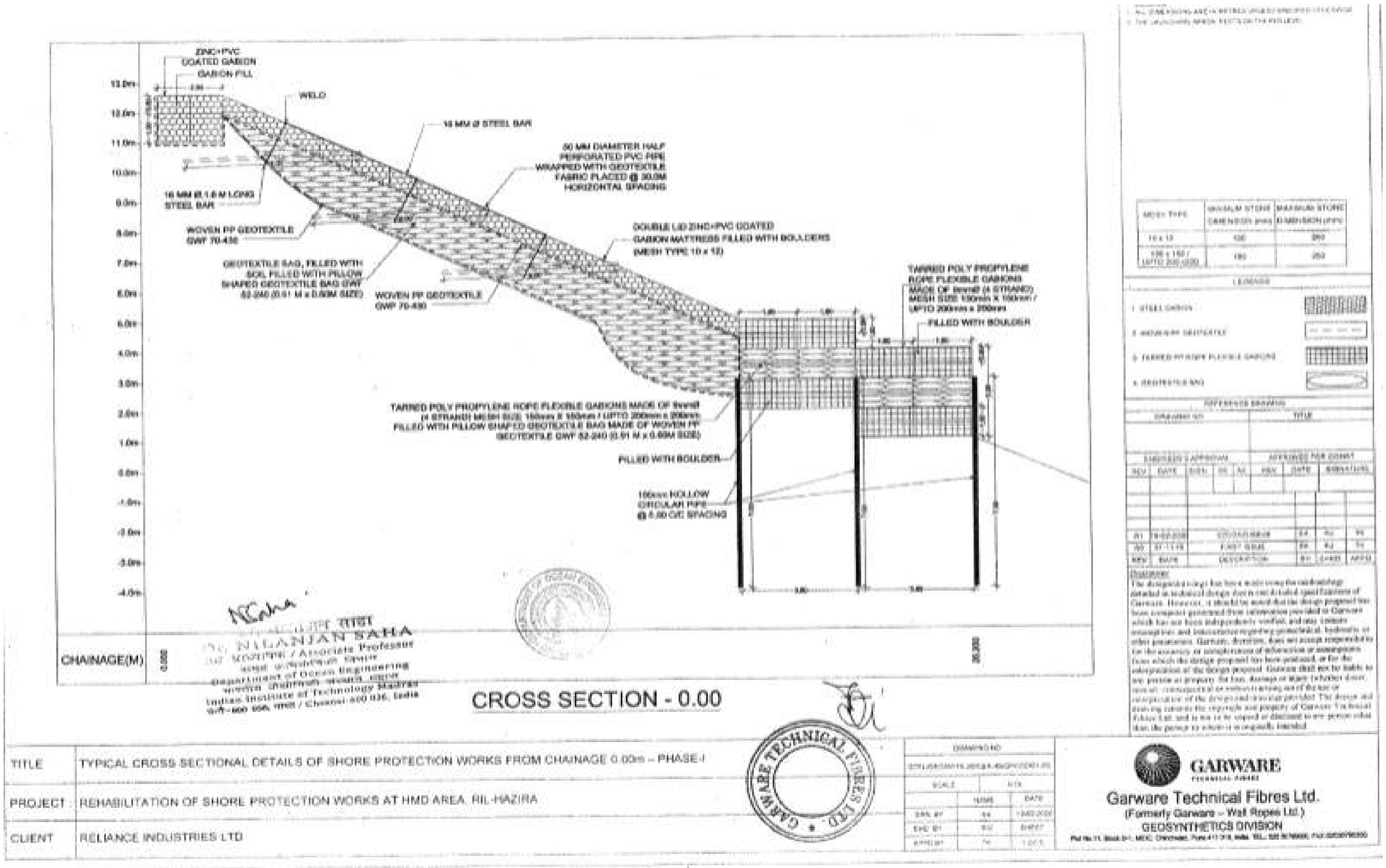
Cross section at chainage 0.0m



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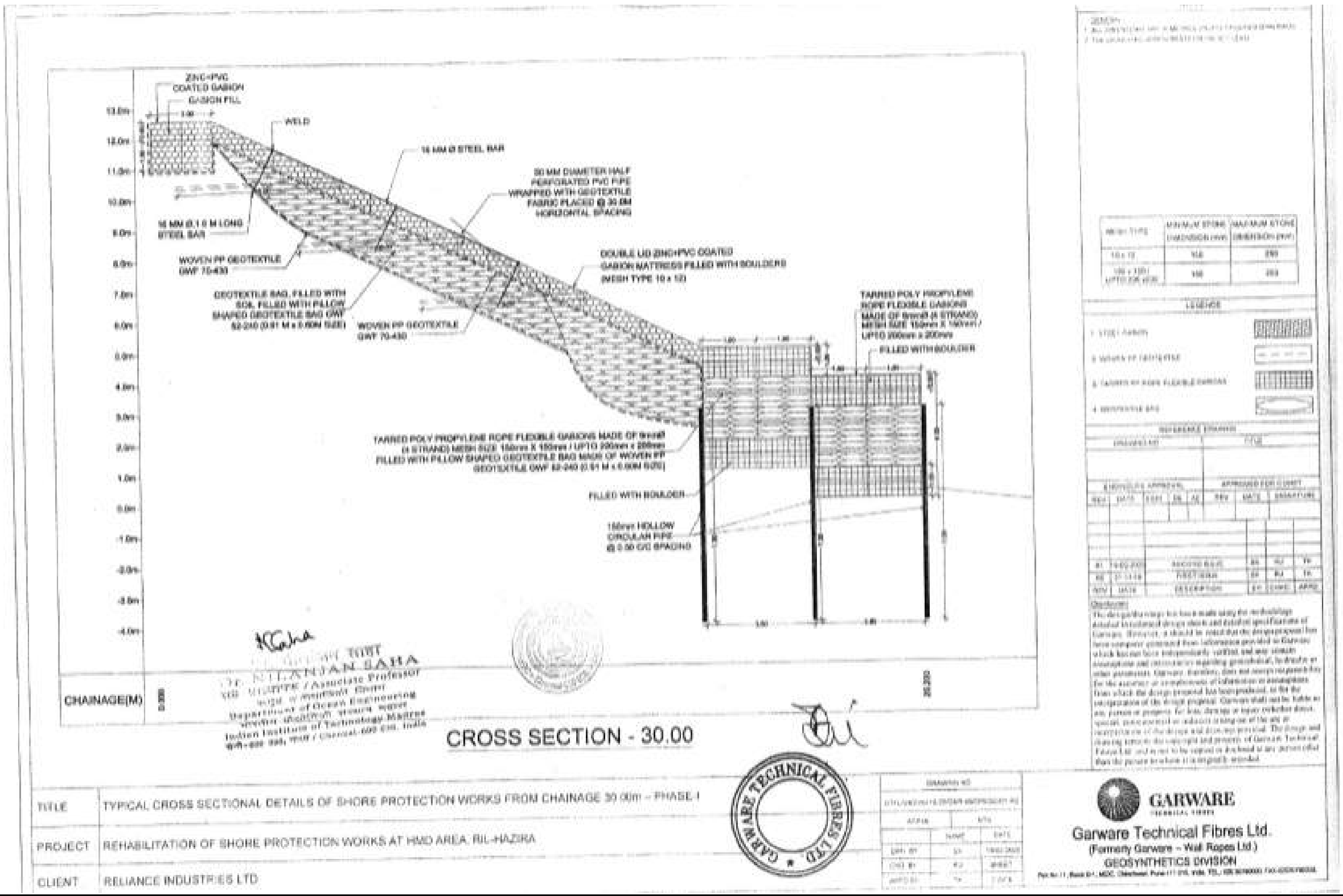
10th and 11th Dec 2020

Cross section at chainage 30.0m



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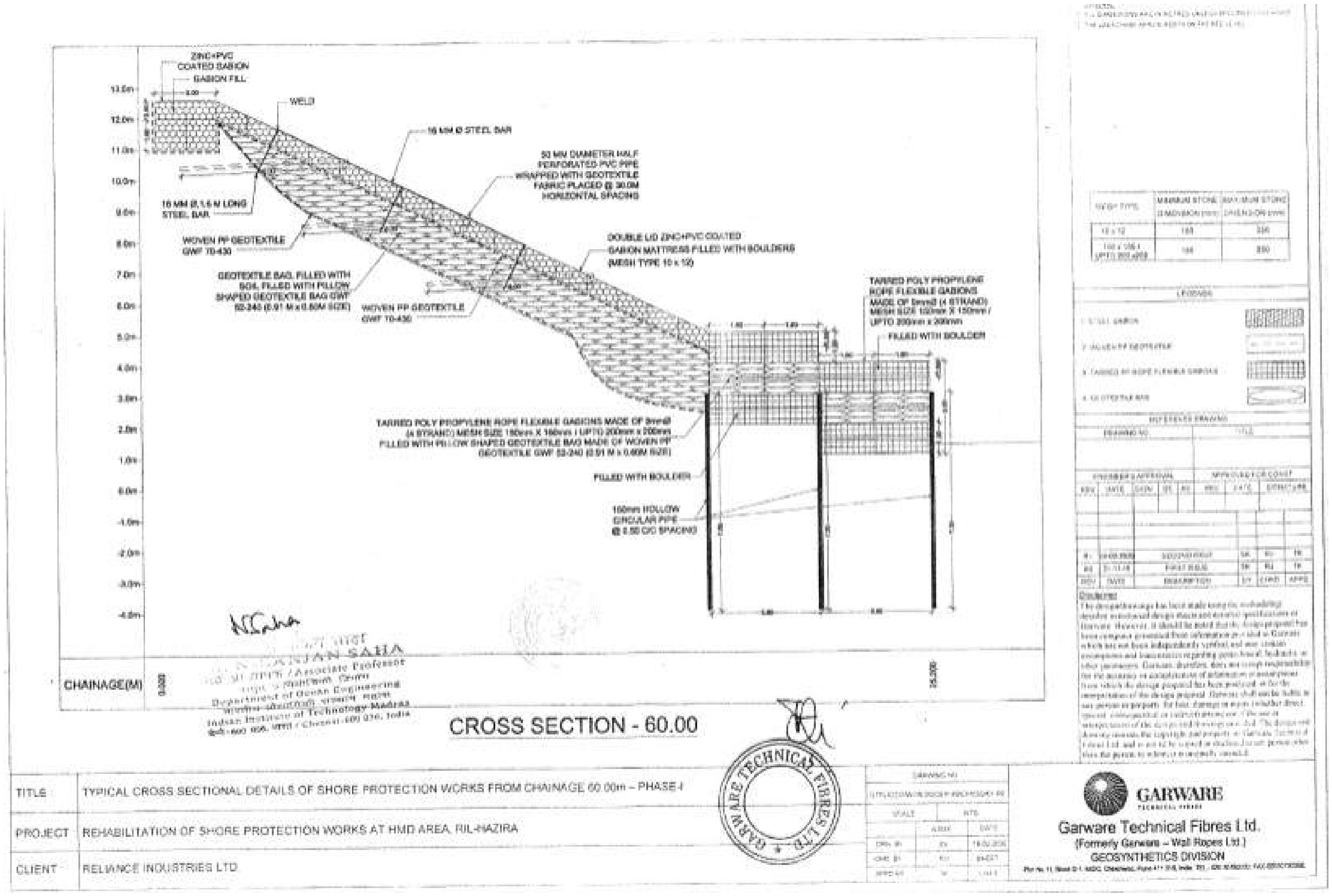
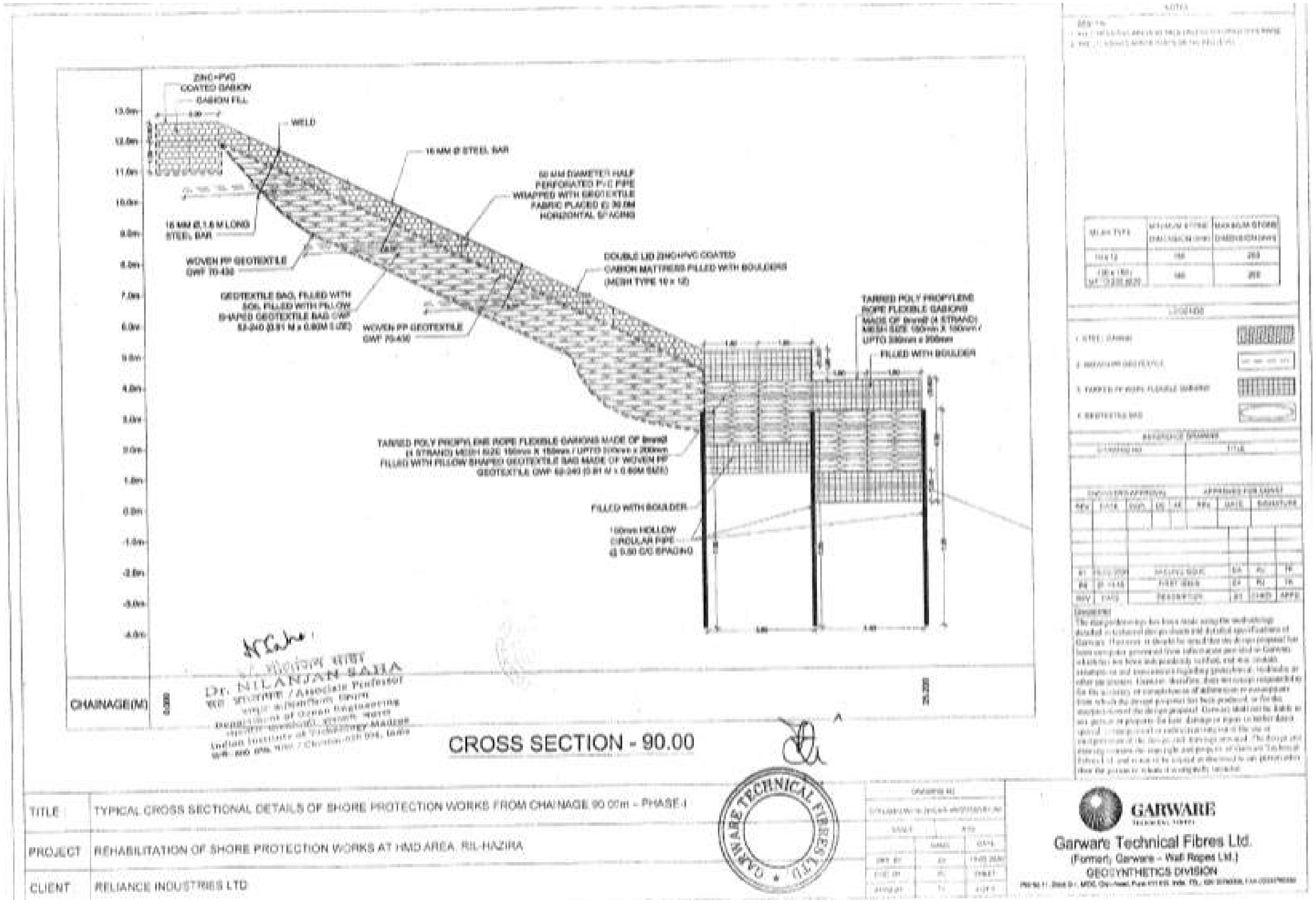
10th and 11th Dec 2020

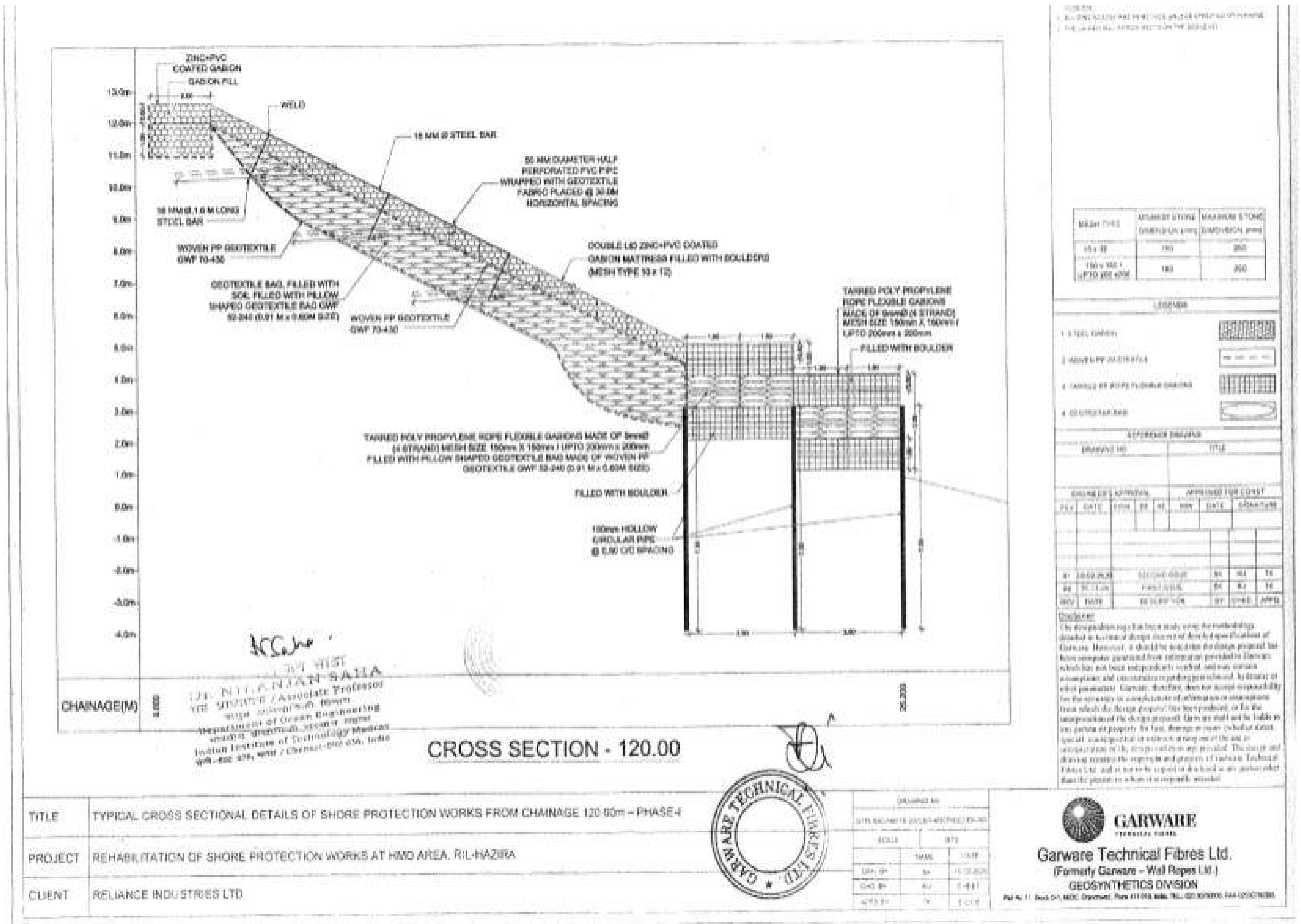
Cross section at chainage 60.0m



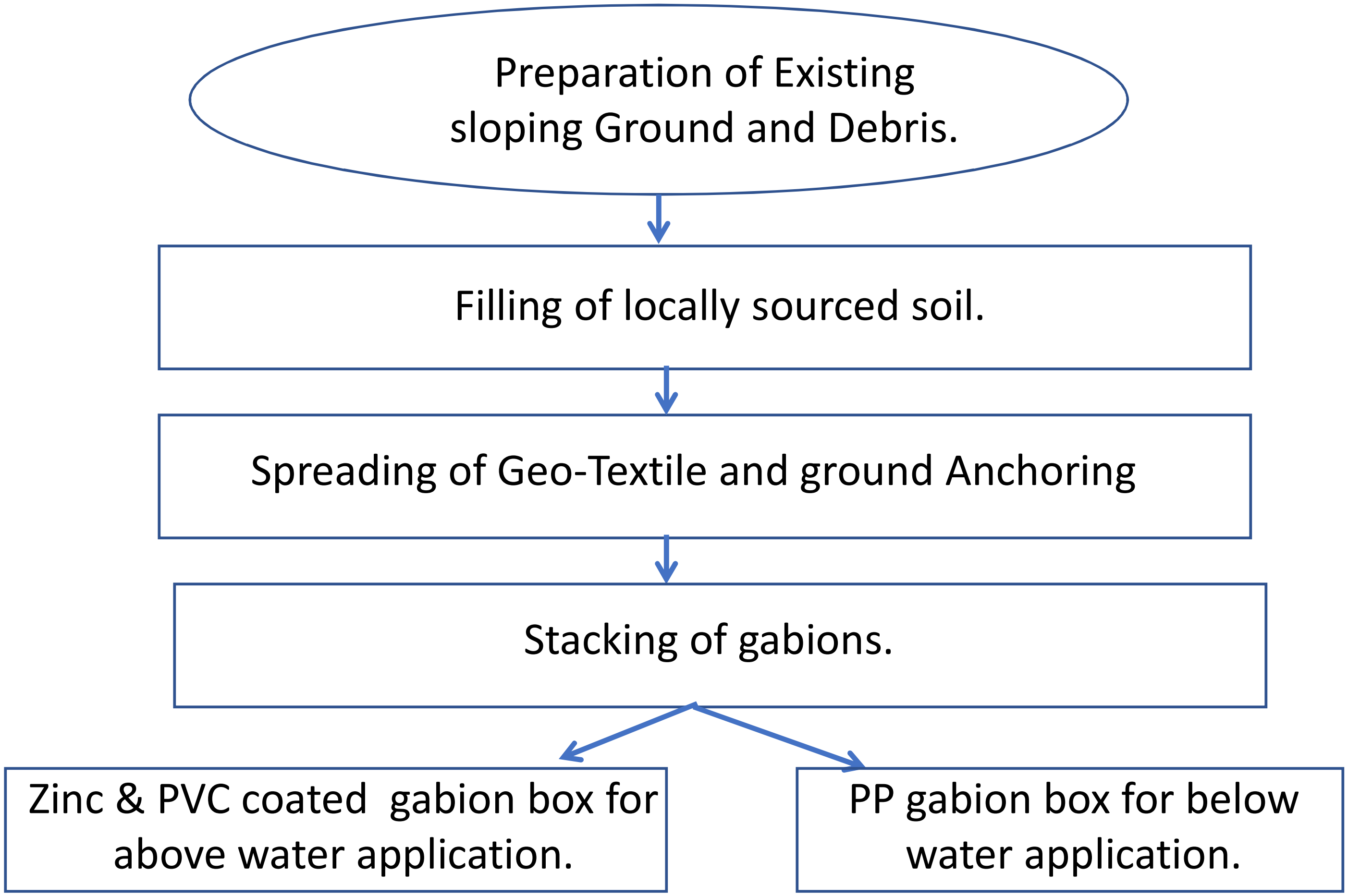
Cross section at chainage 90.0m



Cross section at chainage 120.0m



Materials and Methodology.



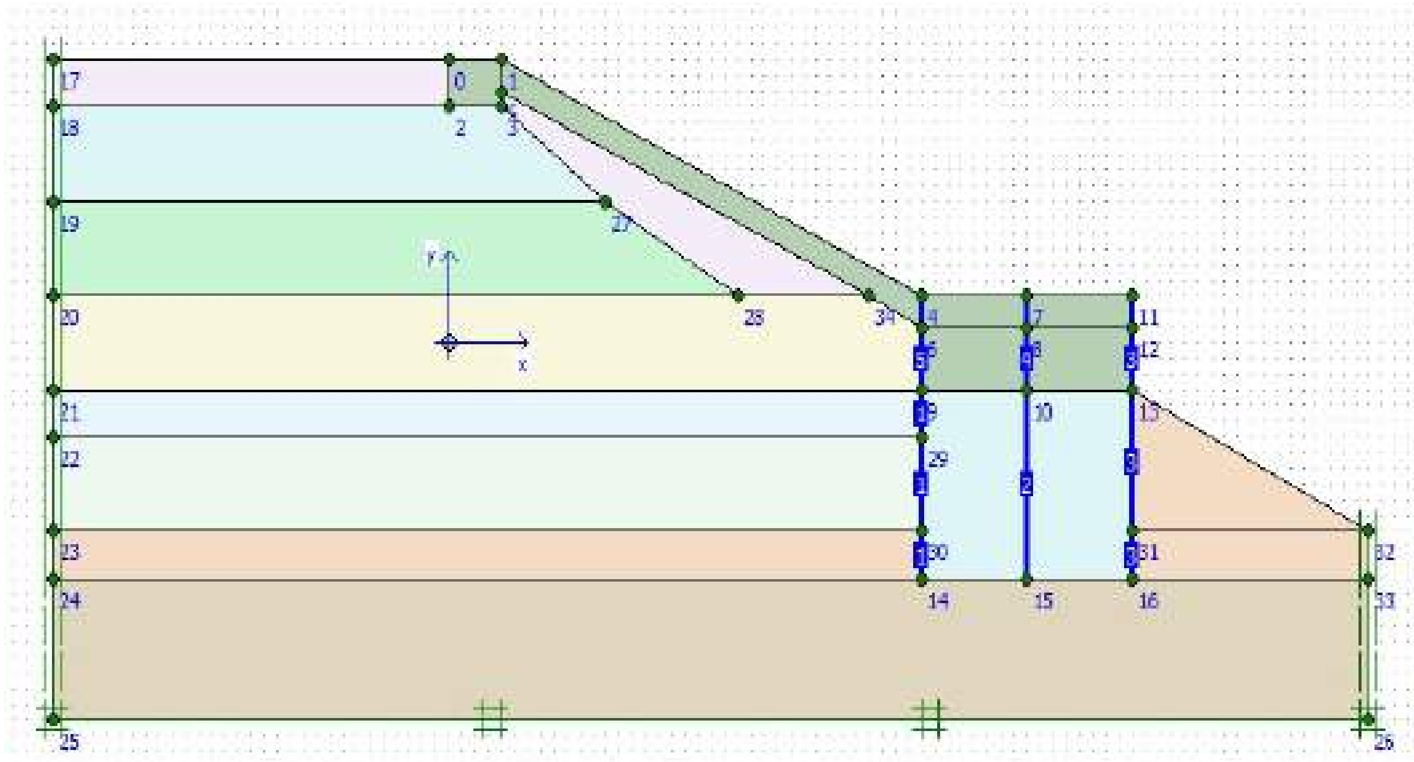
Materials and Methodology.

- The zinc and poly-vinyl chloride (PVC) coated gabion boxes are used for the sub-surface application .
- Polypropylene (PP) flexible tar coated rope gabion of mesh size 100x 100mm were used for the submerged condition, which are stacked during low water level .
- These gabions are placed on the top of woven polypropylene geotextile.
- The gabions on the slope portion are anchored to the prepared sloping bed with 16mm dia and 1.6m long steel bar.
- These bars are anchored at the top and bottom in the trenches of size 2m by 1.8m filled with gabion boxes.

Materials and Methodology.

- All the gabion boxes are filled with boulders manually.
- The flexible gabion boxes in the apron of 4m length with 1m thickness are provided without steel anchor so that the apron are anchored to the trench and can carter the scour profile.
- The protection work has been organised from the river front using floating barges, in places where road access was not available.

The PLAXIS 2D Model



Conclusion

- The traditional way of bank protection, methods and materials were in effective over this particular site.
- Site Specific remedial measure was proposed and constructed for the erosion prone zone.
- The river bund was reinforced with geo-textile and protected and gabion protection to arrest the erosion due to river discharge.
- The proposed and designed cross section was analyzed numerically for slope stability using plaxis 2D.
- The slope stability analysis for designed revetment indicates the factor of safety more than 1.5 against the minimum factor of safety of 1.4.

GEOSYNTHTICS IN PAVEMENTS OF PORT AREAS

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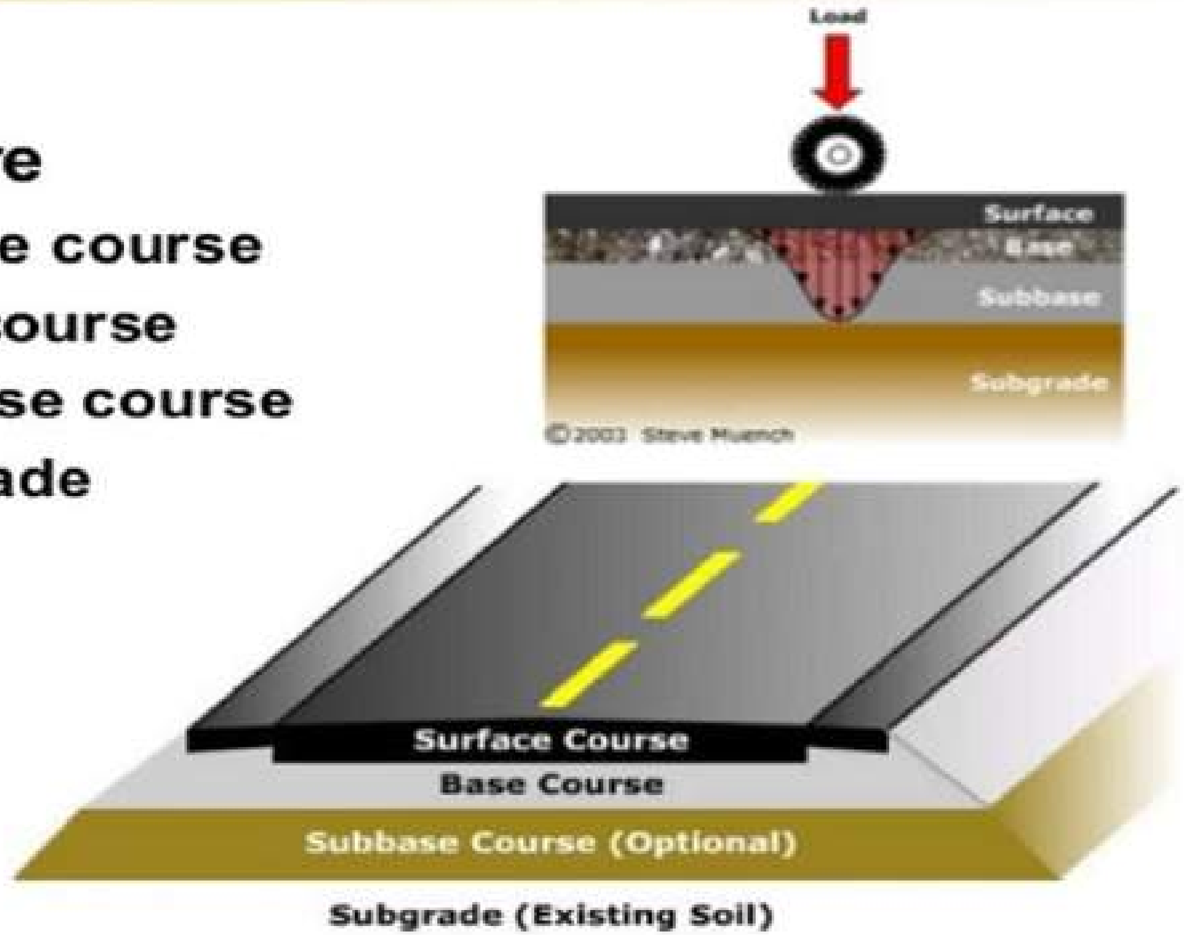
Acknowledgments to Dr. Lekshmi Suku and Sudheer Prabhu
Department of Science and Technology, New Delhi
Maccaferri India
Field studies: CRRI, New Delhi and KRRDA, Bangalore

Flexible Pavement

- Structure
  - Surface course
  - Base course
  - Subbase course
  - Subgrade



Pavement failures

Damages due to lack of drainage and confinement

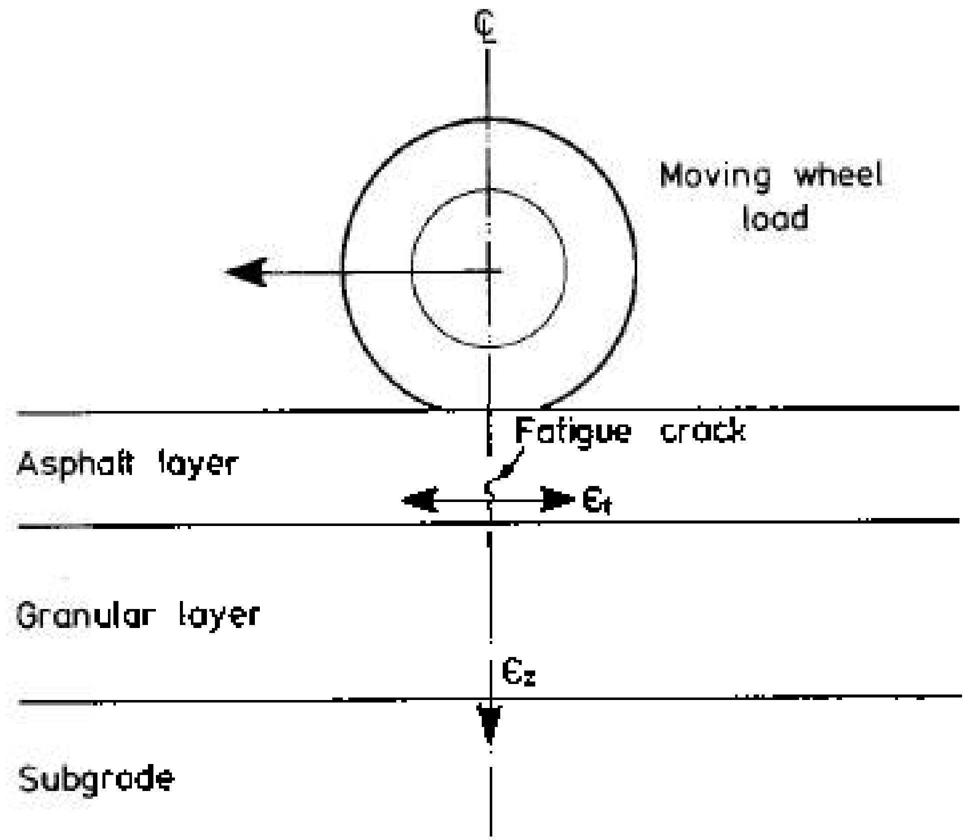
There is a need for proper design and construction methods in this connection to address the above factors

Webinar on Geo-synthetics for Coastal Protection and Port works
10th and 11th Dec 2020

Fatigue and Rutting

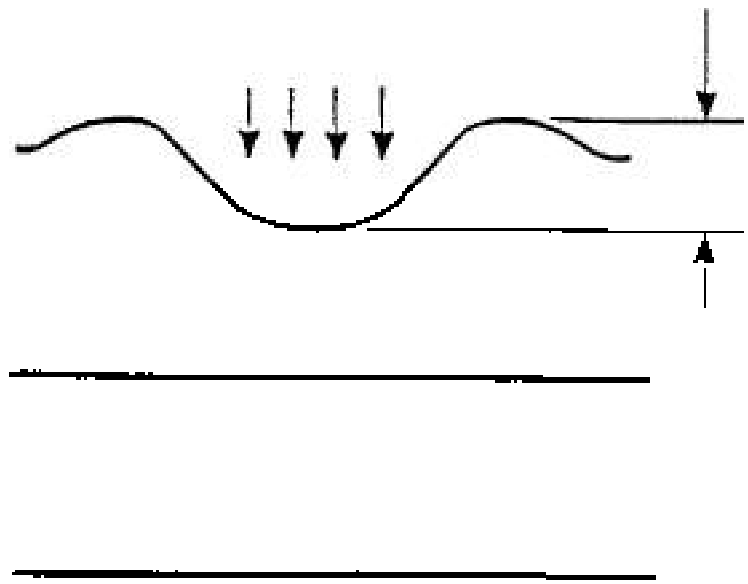
Fatigue Cracking Principles

- Crack initiation depends on tensile strain
- Crack propagation depends on tensile stress
- Interaction of asphalt layer and foundation



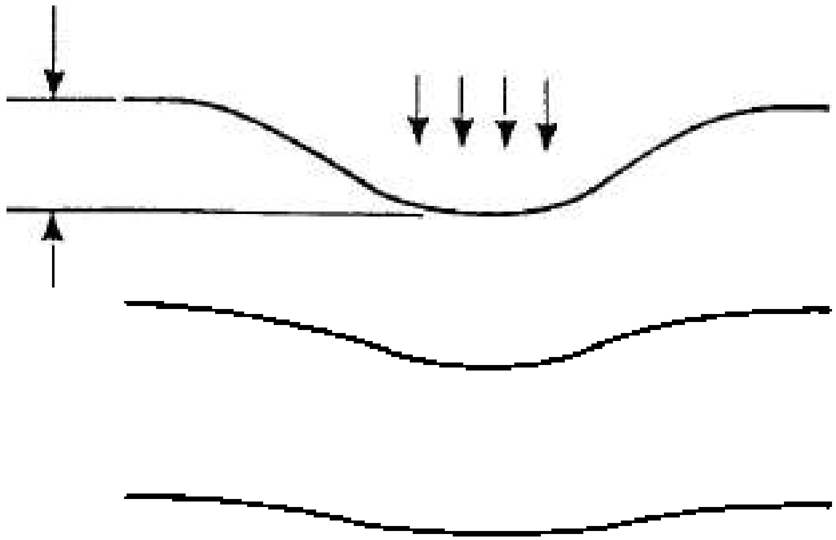
Rutting Mechanism for Asphalt Layer

- Permanent shear strains near surface
- High temperatures
- Heavy wheel loads
- High traffic volume



Foundation Rutting

- Lack of adequate load spreading
- Granular layer or soil
- Problems of water

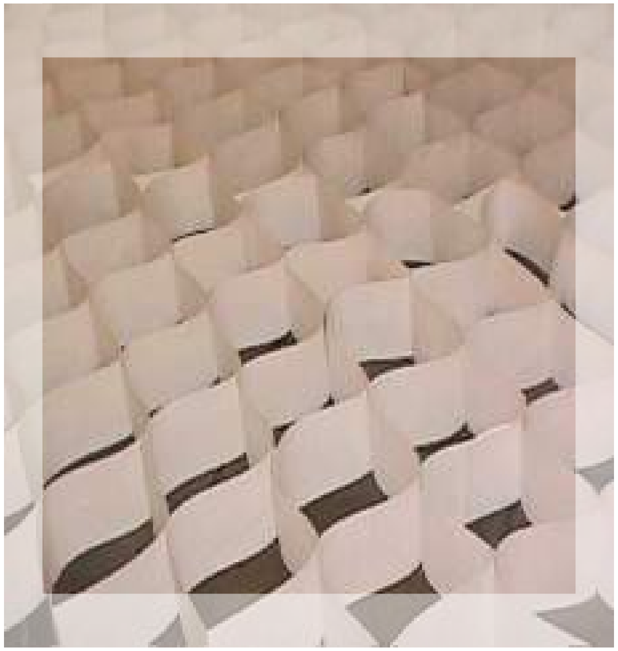


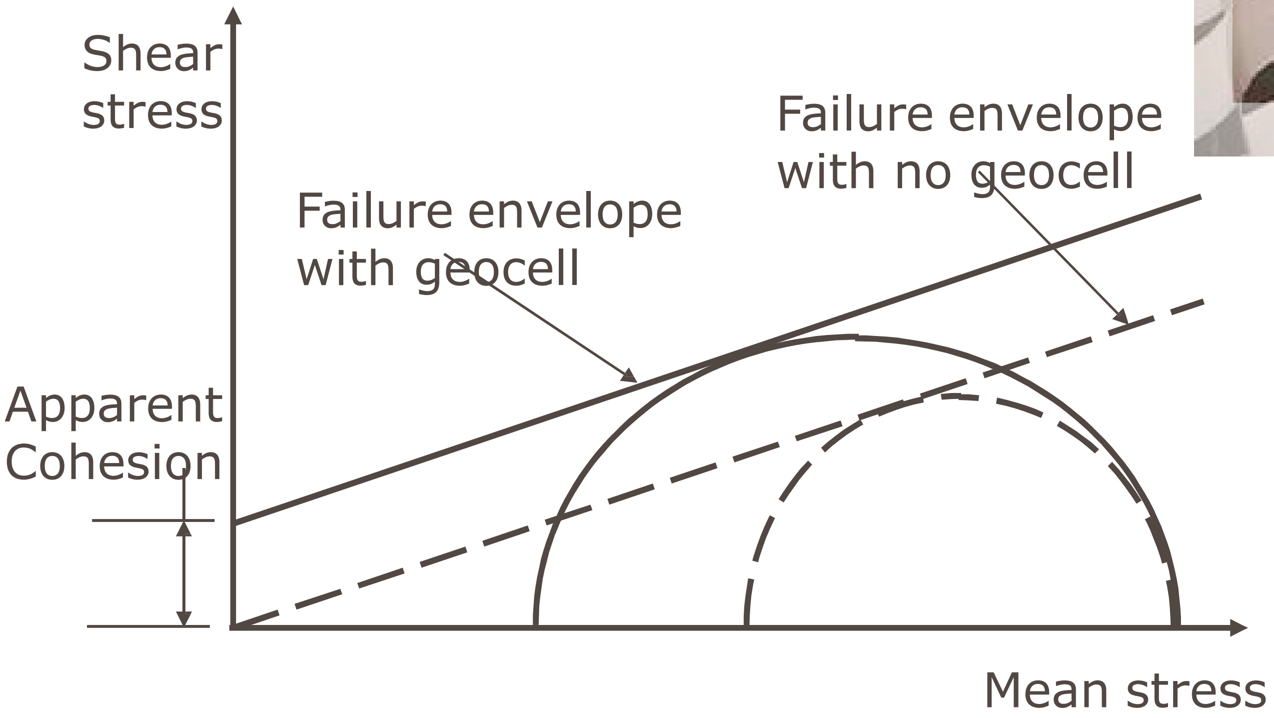
Prof. Sivakum

Webinar on Geo-synthetics for Coastal Protection and Port works
10^{th} and 11^{th} Dec 2020


GEOGRIDS

The development of 'apparent cohesion' in geocells





Use of geosynthetics results in significant savings, improved performance and very good serviceability in both short term and long term

Geosynthetics have made it possible to construct roads and pavements in seemingly difficult situations such as marshy stretches, soft and organic deposits and in expansive soil areas

Functions of Geosynthetics in Roadways

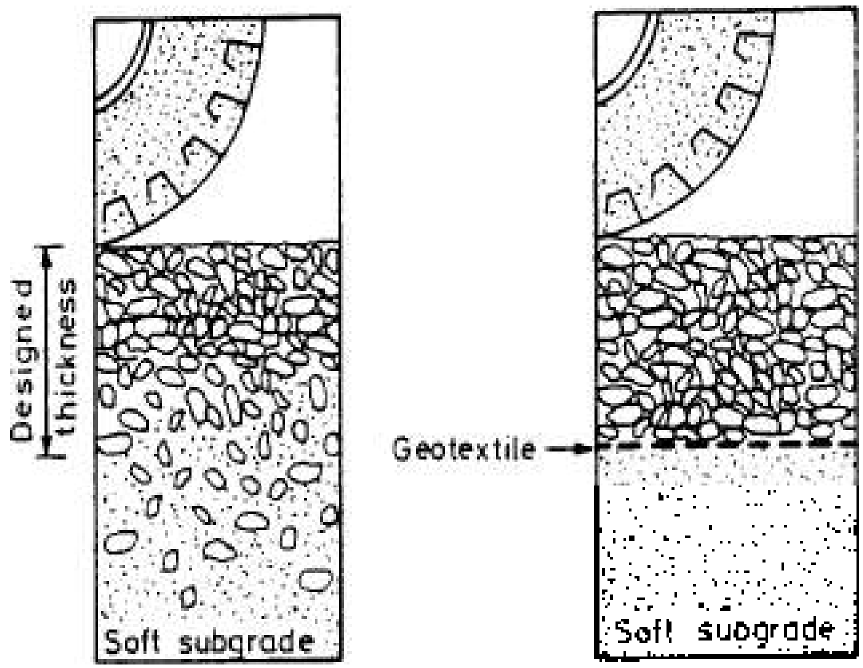
1. Acts as a separator to prevent two dissimilar materials (subgrade soils and aggregates) from intermixing. Geotextiles and geogrids perform this function by preventing penetration of the aggregate into the subgrade (localized bearing failures)

2. Soft subgrade soils are most susceptible to disturbance during construction activities such as clearing, grubbing, and initial aggregate placement. Geosynthetics can help minimize subgrade disturbance and prevent loss of aggregate during construction

Functions of Geosynthetics (contd..)

3. The system performance may also be influenced by secondary functions of filtration, drainage, and reinforcement. The geotextile acts as a filter to prevent fines from migrating up into the aggregate due to high pore water pressures induced by dynamic wheel loads

4. It also acts as a drain, allowing the excess pore pressures to dissipate through the geotextile and the subgrade soils to gain strength through consolidation and improve with time



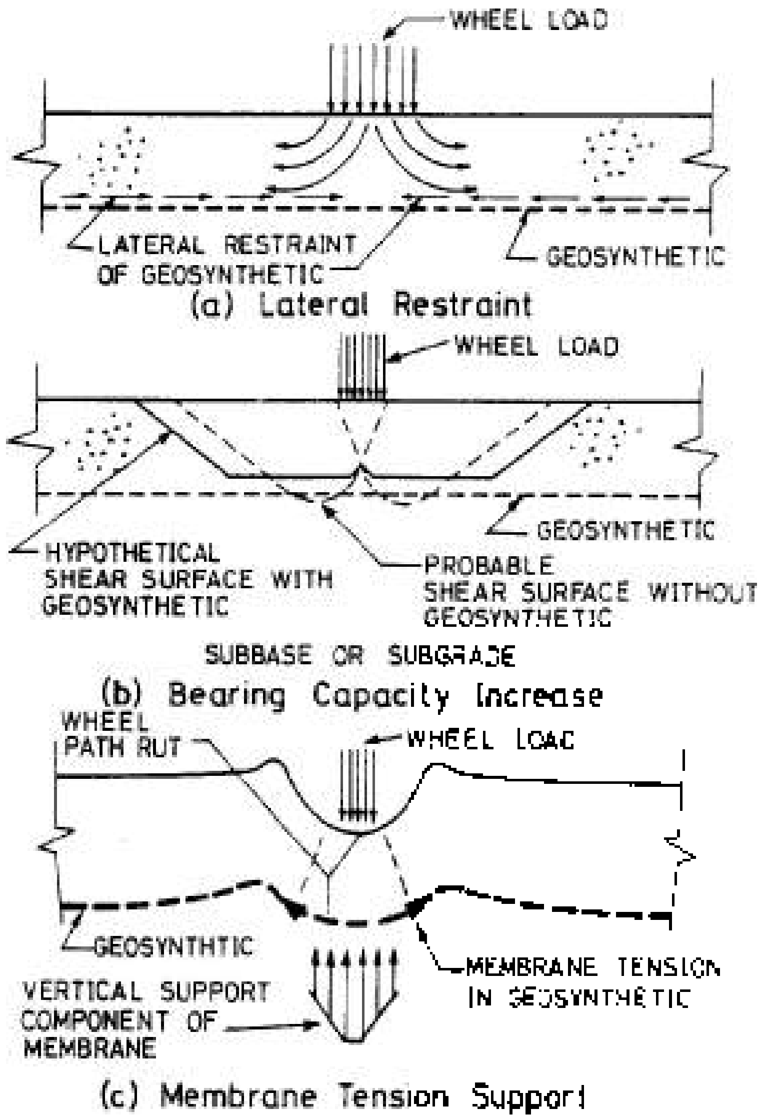
Concept of Geotextile Separation

Mechanisms

1. Lateral restrainment of the base and subgrade through friction and interlock between the aggregate, soil and the geosynthetic

2. Increase the system bearing capacity by forcing the potential bearing capacity failure surface to develop along alternate, higher shear strength surfaces

3. Membrane support of the wheel loads



Reinforcement Functions

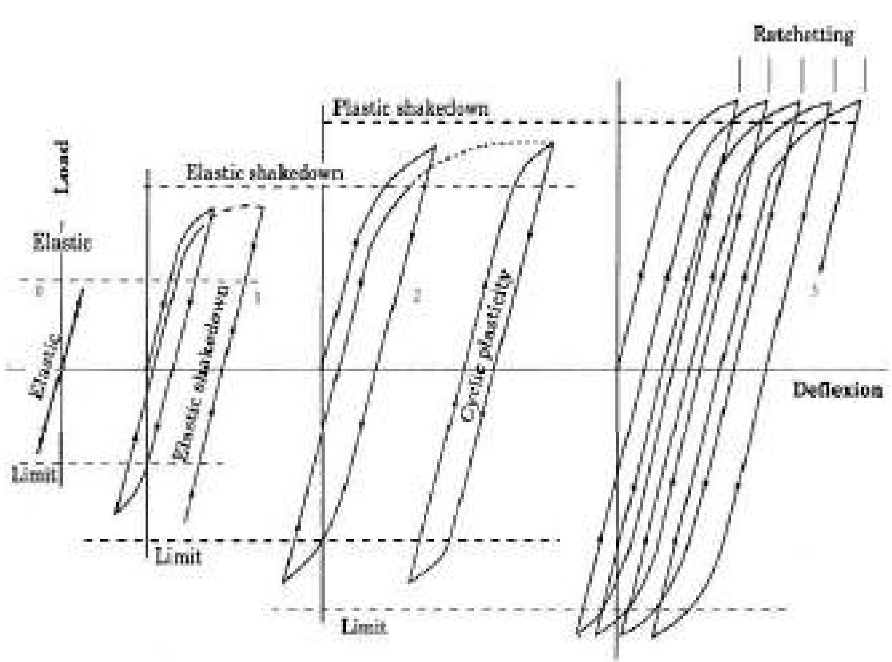
Summary of Recommendations

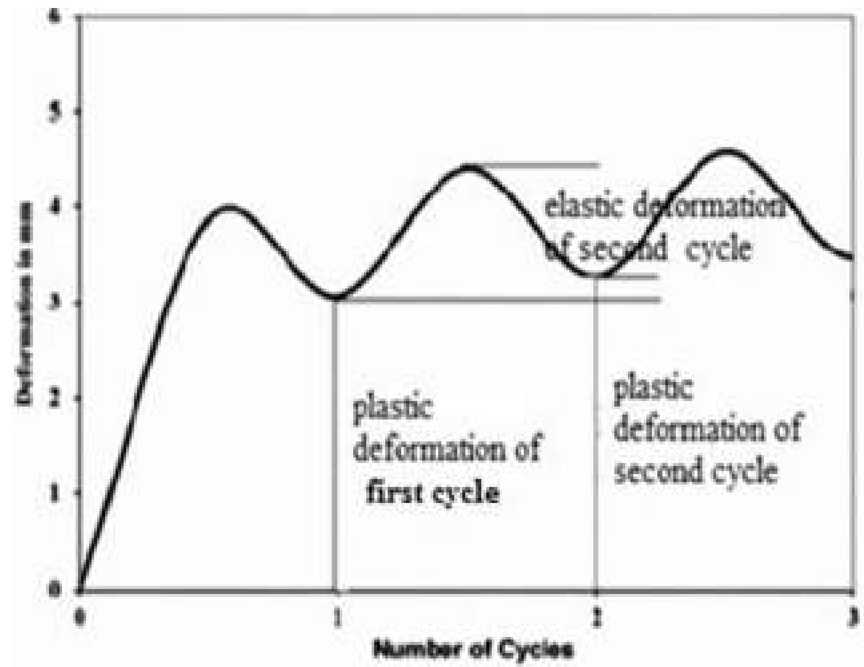
Effectiveness of Geosynthetics as a function of subgrade strength

Undrained Shear Strength(kPa)	Subgrade CBR	Functions
60 - 90	2 - 3	Filtration and possibly separation
30 - 60	1 -2	Filtration, separation, and possibly Reinforcement
< 30	< 1	All functions, including reinforcement

Overview of presentation

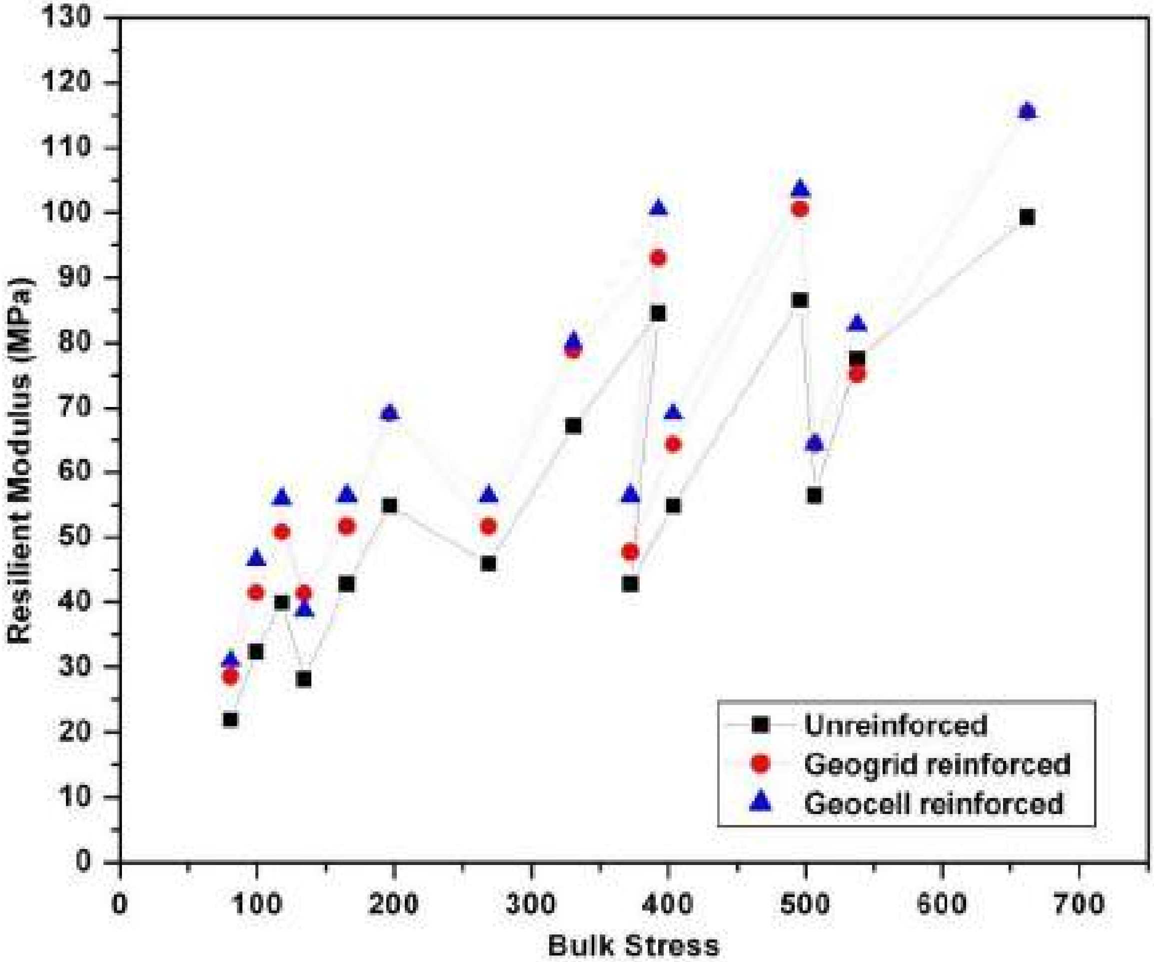
- Background
- Materials and Methods
- Effect Of Geogrid and Road Mesh reinforcement In Granular Bases Under Repeated Loading
- Field studies
- Reflections on pavement design
- Conclusions





Pavement response for different loading conditions (Wermeister et al.2001)

Effect of Geogrid and Geocell reinforcement on Resilient Modulus (AASTHO T-307) (Elastic and Elastic Shakedown Range)

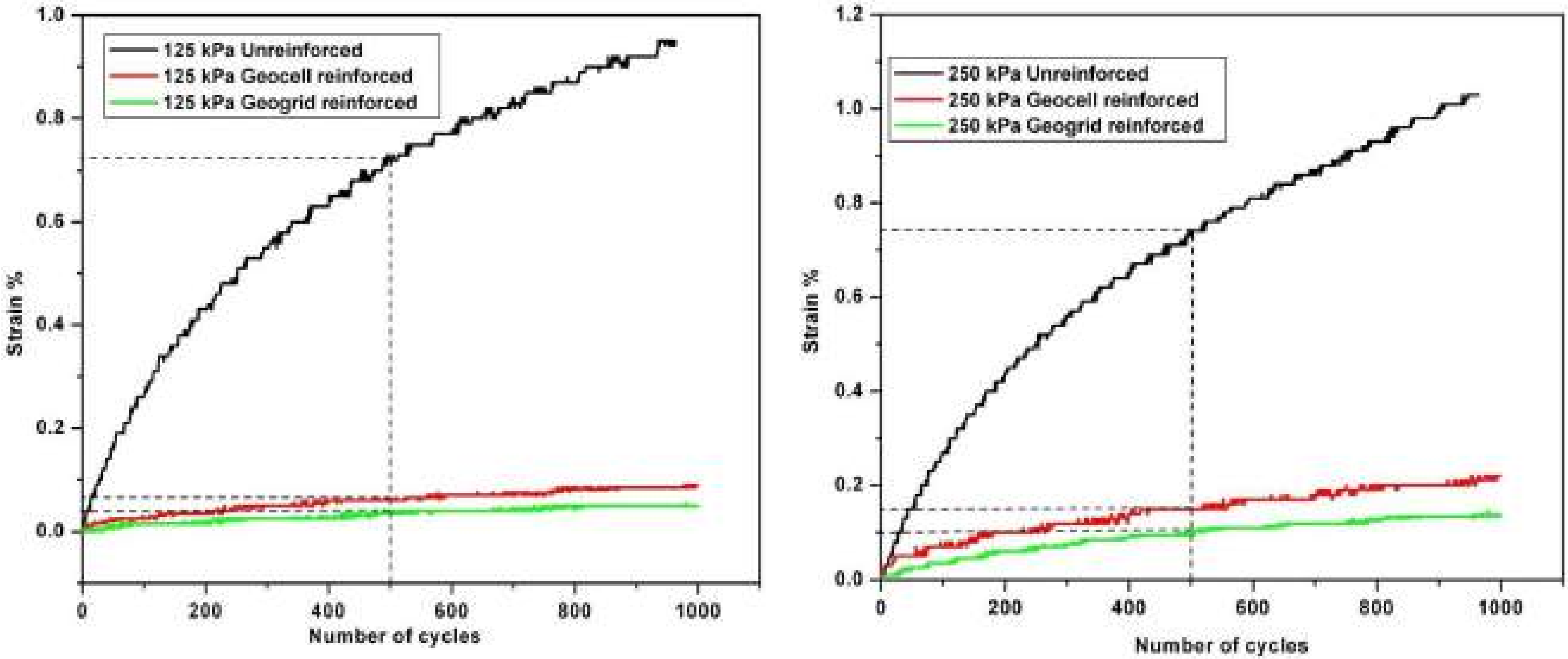


- Geosynthetics increases the resilient modulus of the material.
- Geocell provides higher confinement which results in the reduction of resilient deformation and subsequent increase in resilient modulus.
- Geogrids employ both membrane effect as well as interlocking effect and reduces the resilient deformations effectively compared to the unreinforced sections.

Effect of Geogrid and Geocell reinforcement on Resilient Modulus (Plastic Shakedown and Incremental Collapse Range)

- The experiments were carried out for confining pressures of 100kPa and 150kPa
- The loading was applied on the sample without rest period

Permanent Strains (100 kPa confining pressure)



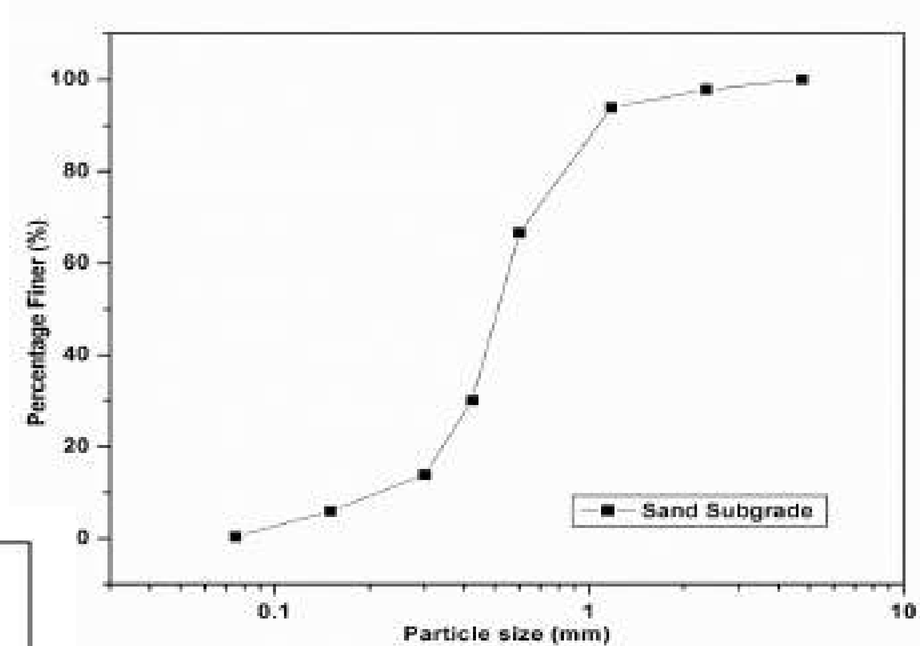
Permanent strain vs no of cycles for reinforced and unreinforced samples at deviaotic stresses of (a) 125 kPa (b) 250kPa

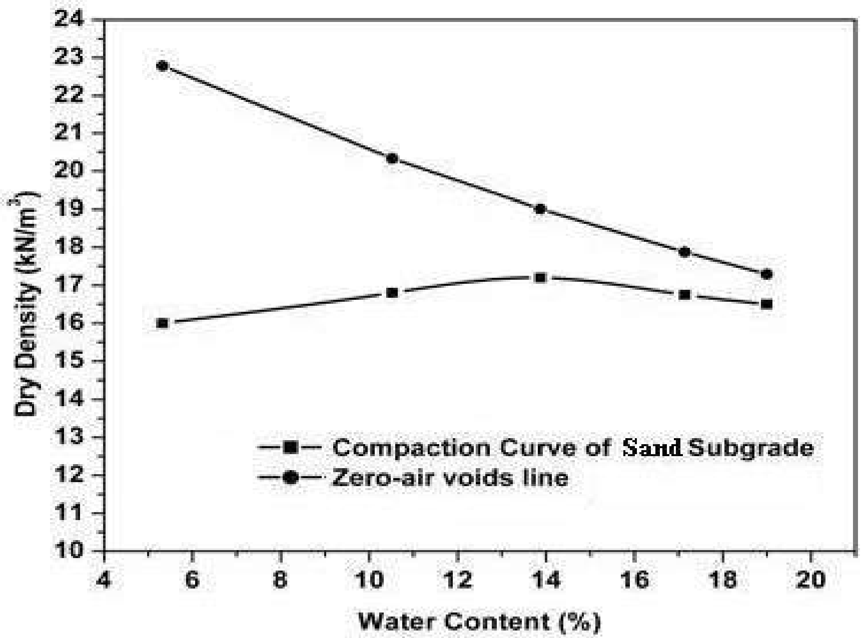
Materials And Methods

- Experiments were carried out on unreinforced, and road mesh aggregate systems to evaluate their performance under static and repeated loading conditions.
- The materials that were used in the plate load tests to represent the subgrade and base layer of pavements were:
  - Subgrade Soil - sand
  - Base layer - Aggregate

Plate load test setup with data acquisition system

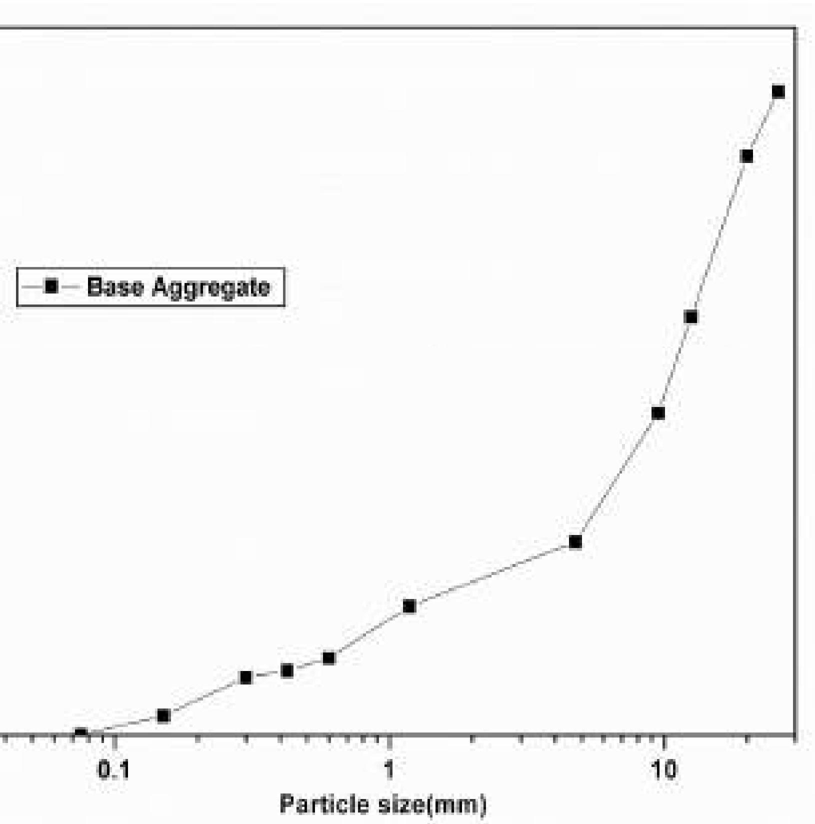
Material Properties
Sand Subgrade

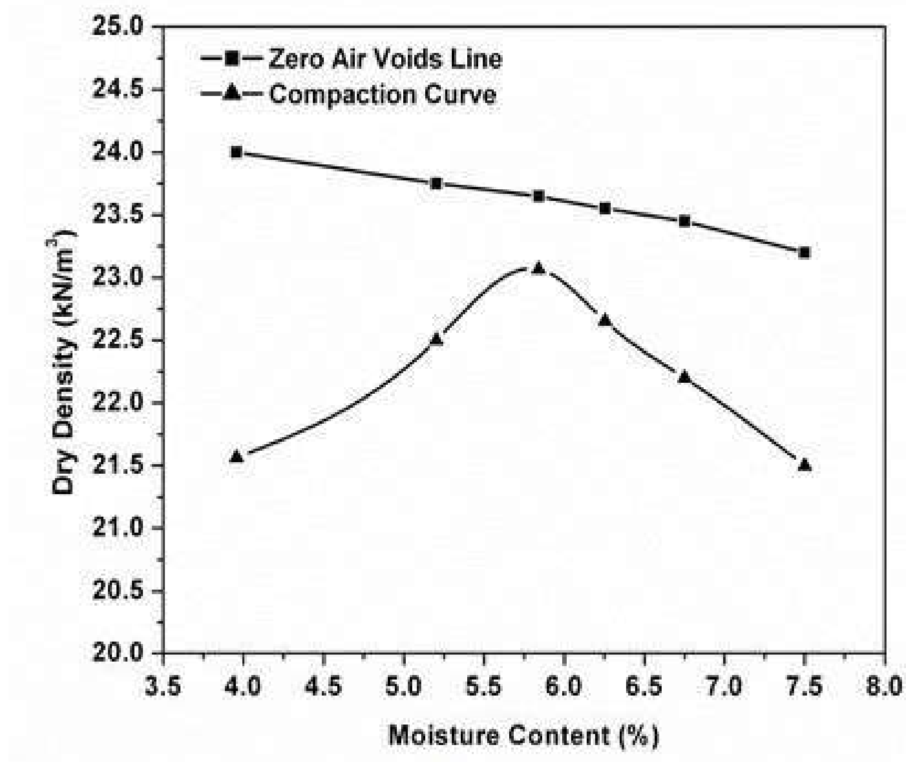




Parameter	Value
D_{10}	0.2mm
D30	0.4mm
D_{60}	0.48mm
Coefficient of Curvature(C_c)	1.67
Coefficient of uniformity(C_u)	2.4
Relative density	55%
CBR (soaked)	7%
CBR (unsoaked)	10%

Material Properties
Base Aggregate





Properties	Values
CBR (soaked)	78.45%
CBR(unsoaked)	115%
Aggregate Impact Value	24%
Aggregate Crushing Value	24%

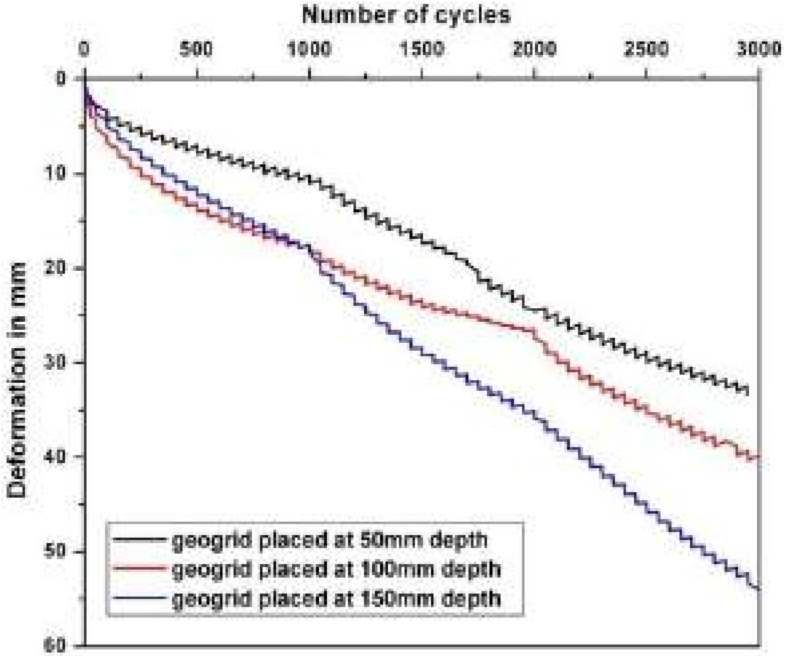
Material Properties – Geogrid

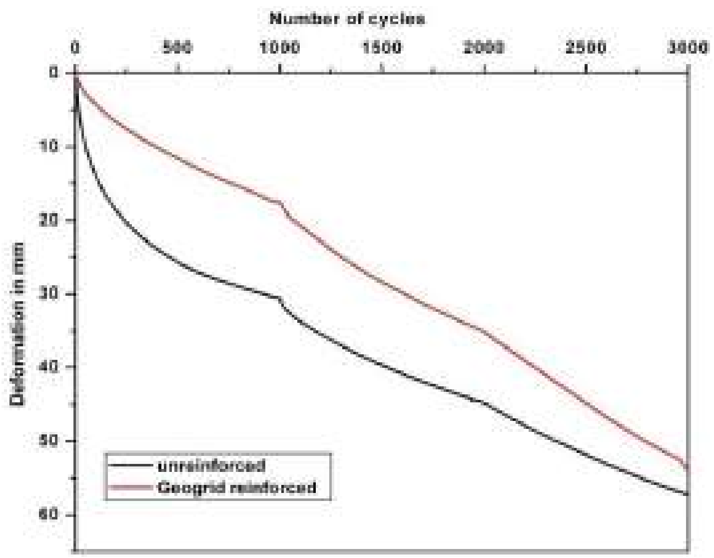
In the present investigation, biaxial geogrid was used as reinforcement in Base because of its high tensile strength and stiffness

- A geotextile, was used in the experiments as an interface between the subgrade and the base material in order to avoid the intermixing of the two materials.
- Strain gages having a gauge resistance of 350Ω (ohms) and a gauge factor of 2 was used to measure the strains
- The pressure gauges were placed at different depths of the base and at the interface of two layers.

Properties of Geogrid

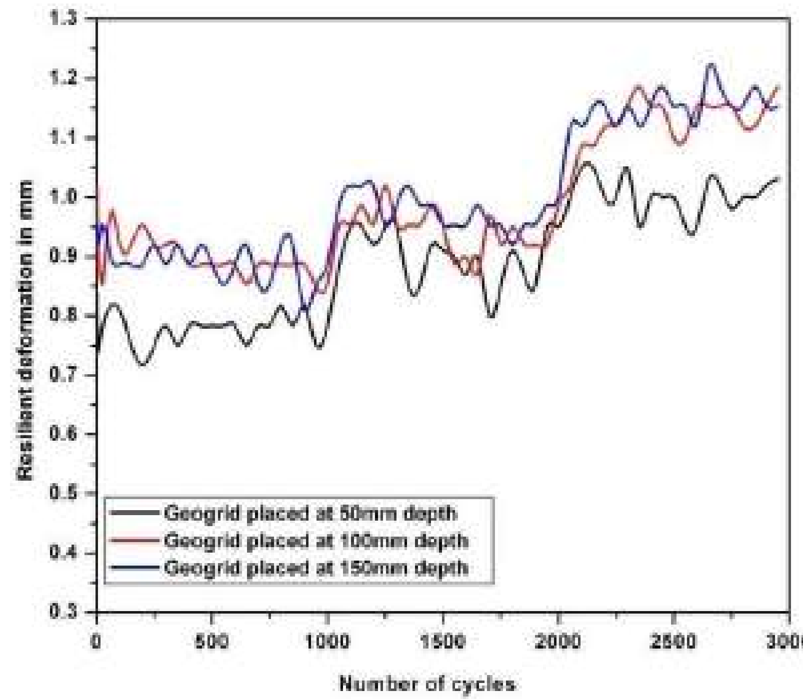
Polymer	Polypropylene
Aperture Size (MD, XMD)	30mm, 30mm
Ultimate tensile strength	20 kN/m
Mass Per Unit area(g/m^2)	200
Shape of the aperture opening	Square





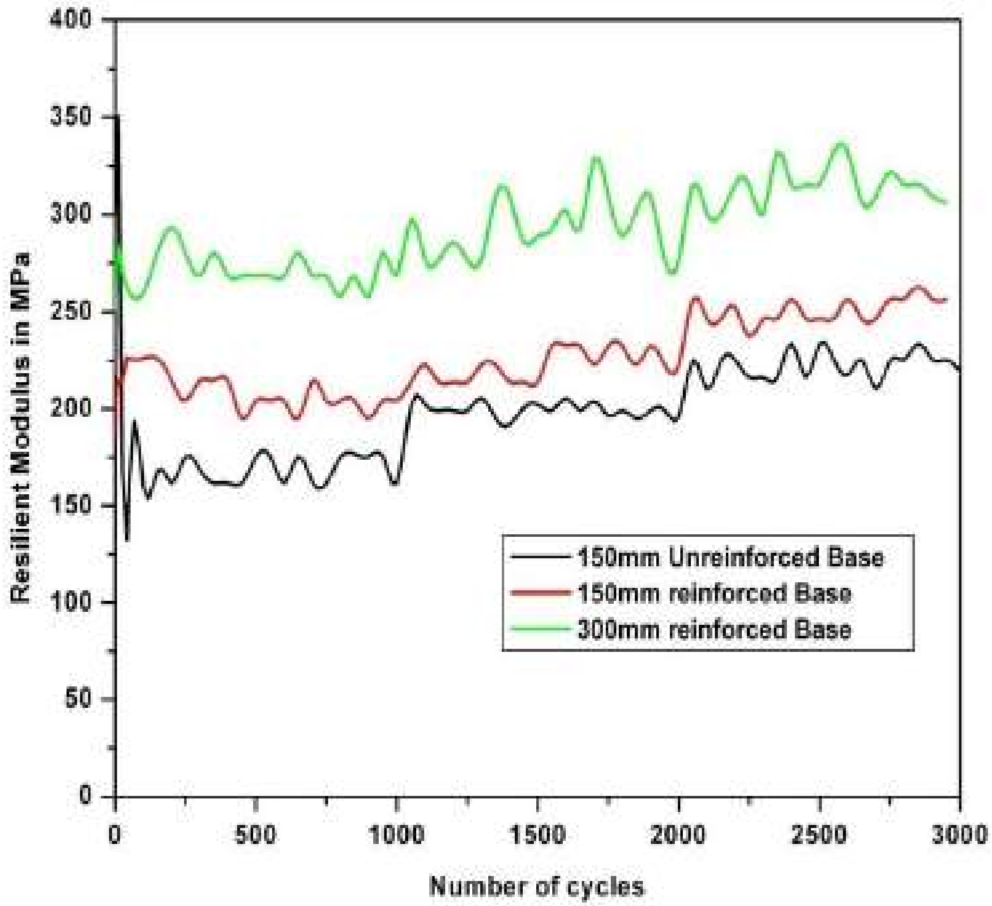
Optimum Depth of placing the geogrid

- Placed at 50mm depth
- Placed at 100mm depth
- Placed at 150mm depth



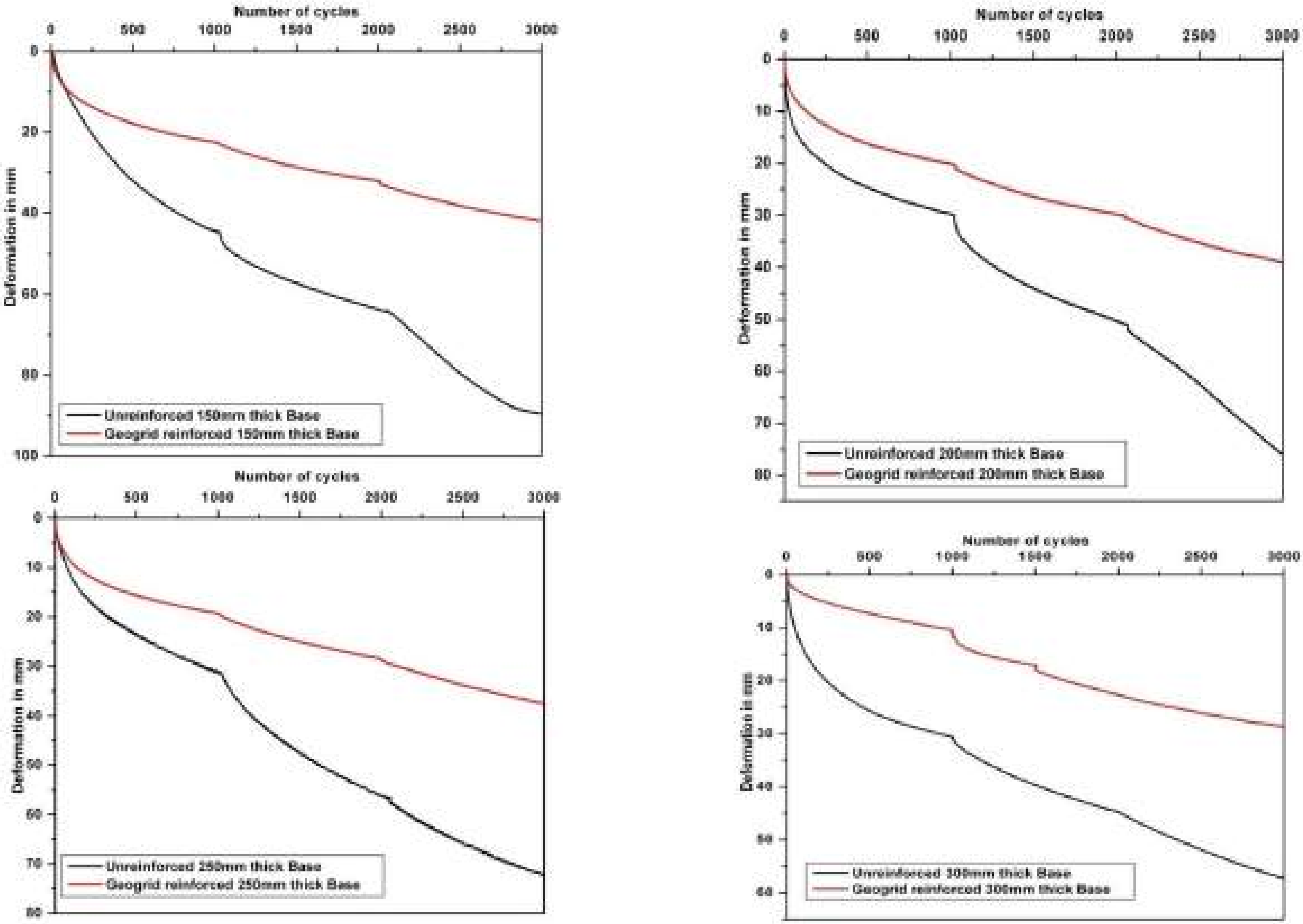
Resilient Modulus

$$\delta = \frac{BIq\left(1-\nu^{2}\right)}{E}$$

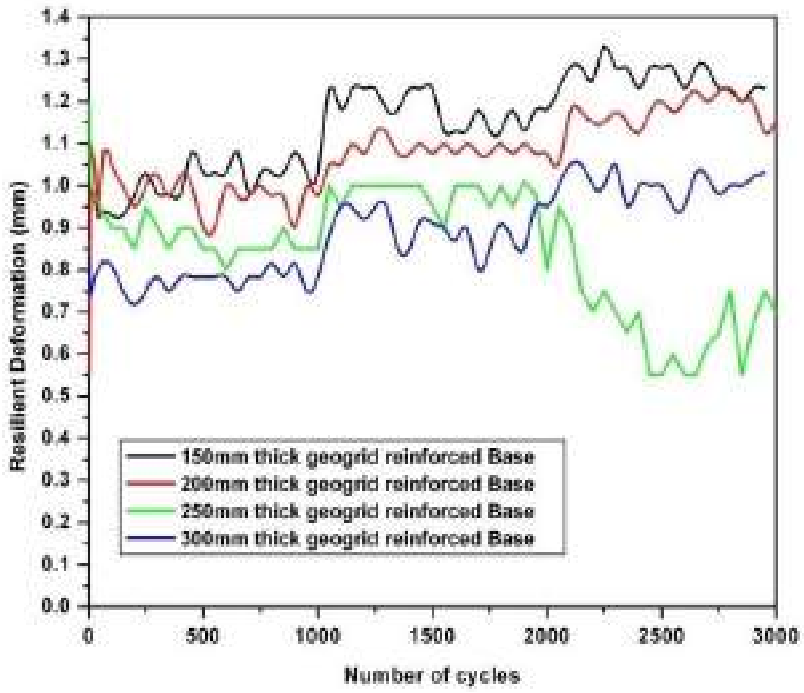


- The interlocking property of the geogrids as well as membrane effect of the geogrid resulting in increased resilient modulus of the base layer

Permanent Deformation Studies

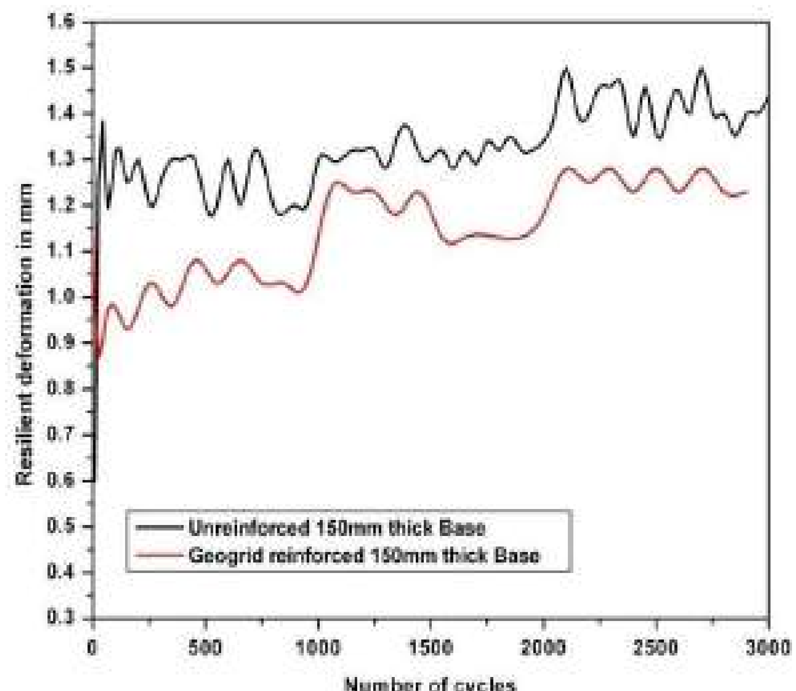


Resilient Deformation Studies

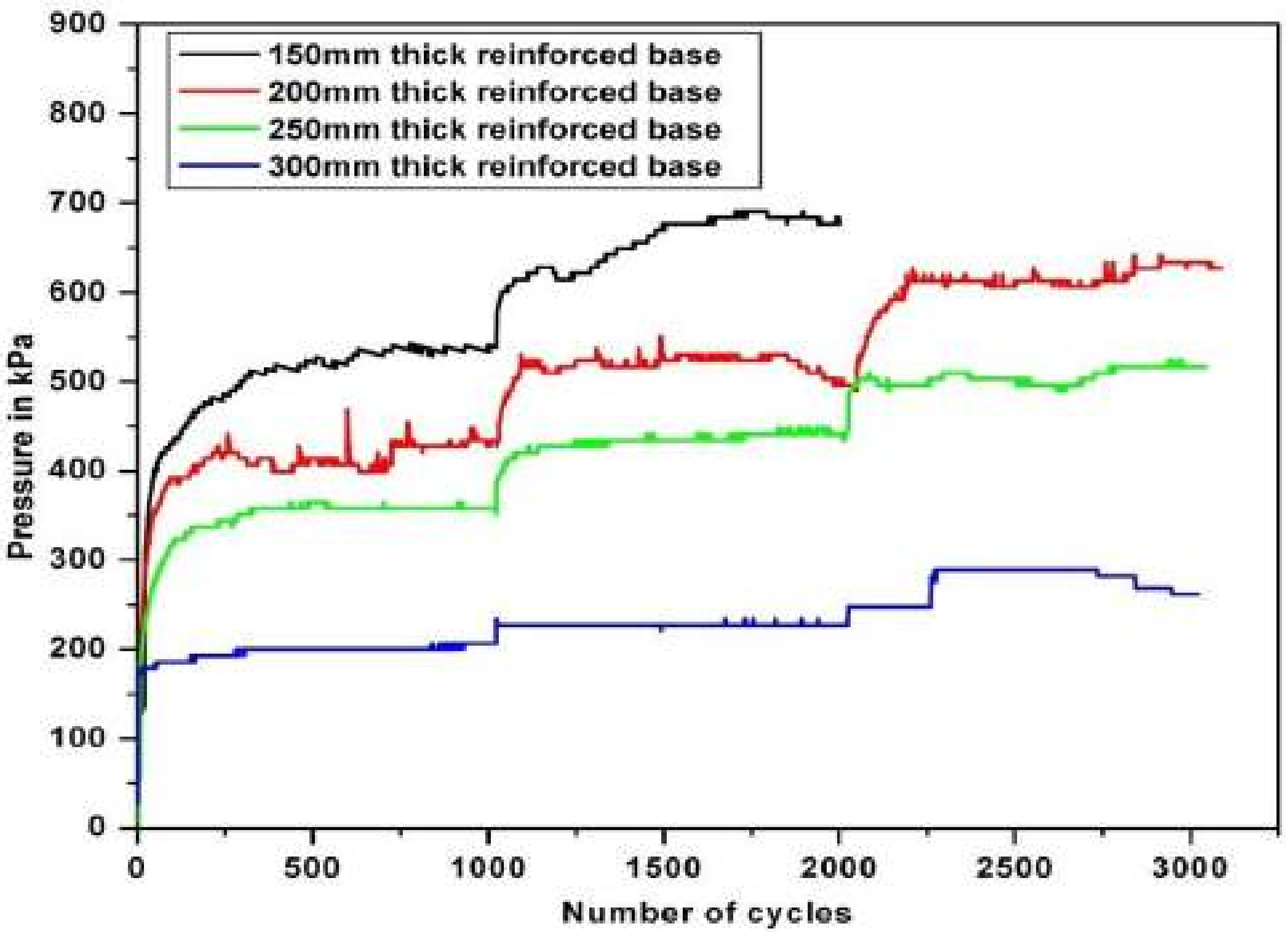


- A trend of increasing resilient deformation was visible with decrease in thickness of Base

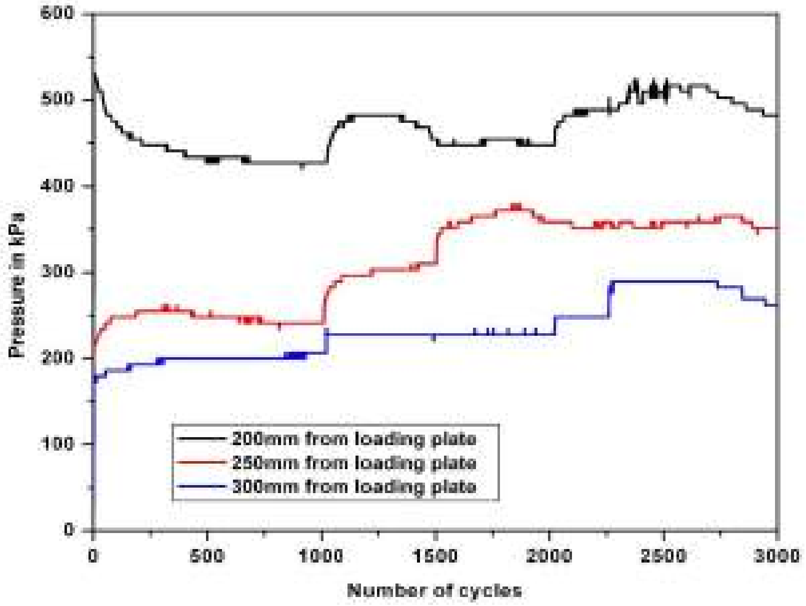
- The resistance offered by the geogrid leads to very low permanent and resilient deformation of the reinforced section compared to the unreinforced section



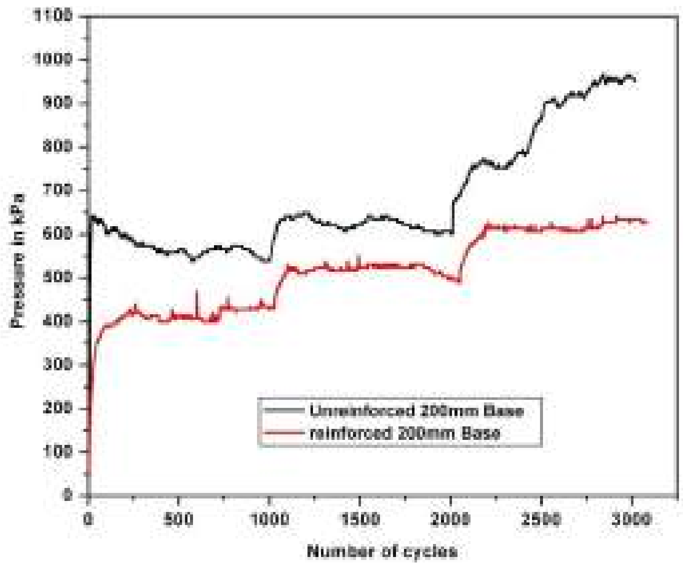
Pressure distribution in base layer

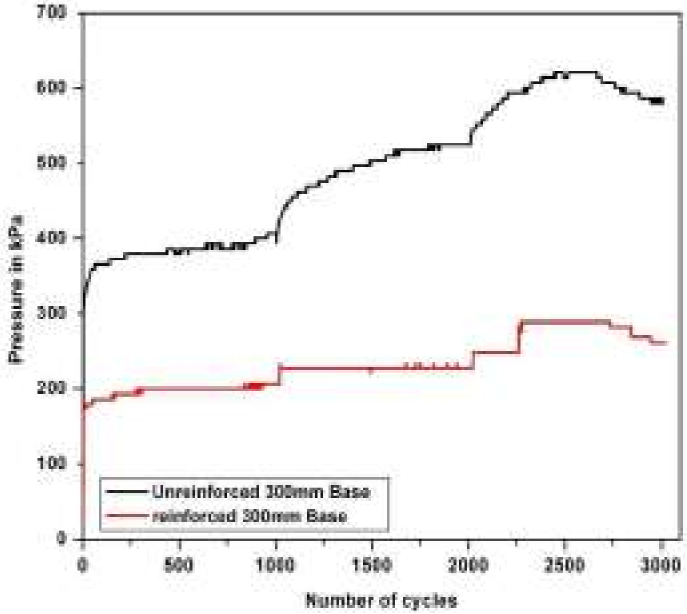


- As the thickness of the base layer increased the pressure getting transferred to the subgrade decreased
- The reinforcement was effective in reducing the pressure by 50 to 80% of the pressure applied on top of the loading plate.

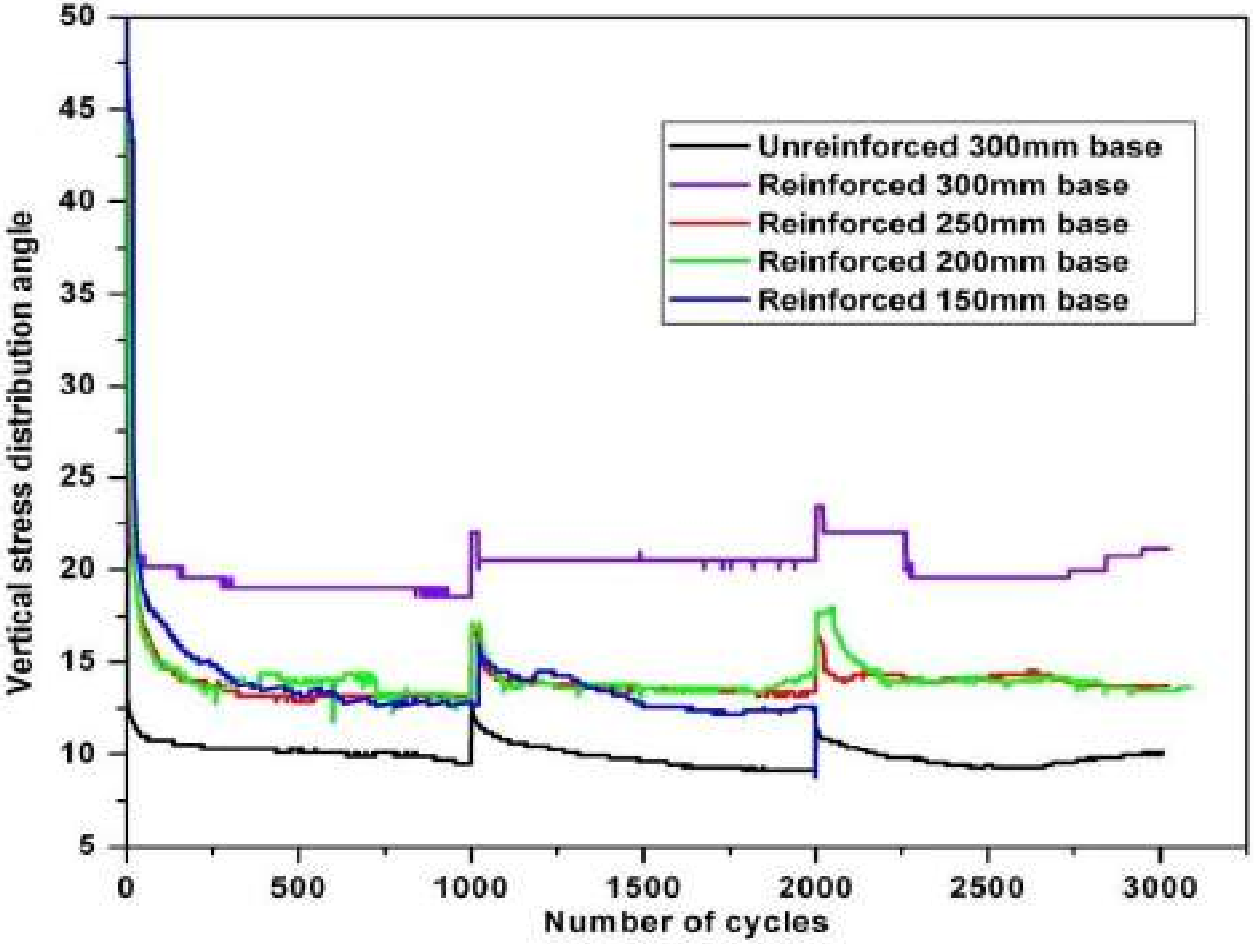


- The pressure applied on top of the pavement was reduced by 60% at 200mm depth, 75% at 250mm depth and 80% at 300mm depth
- At higher loading, the pressure increases and the difference between the pressure at the subgrade level for reinforced and unreinforced sections are clearly visible



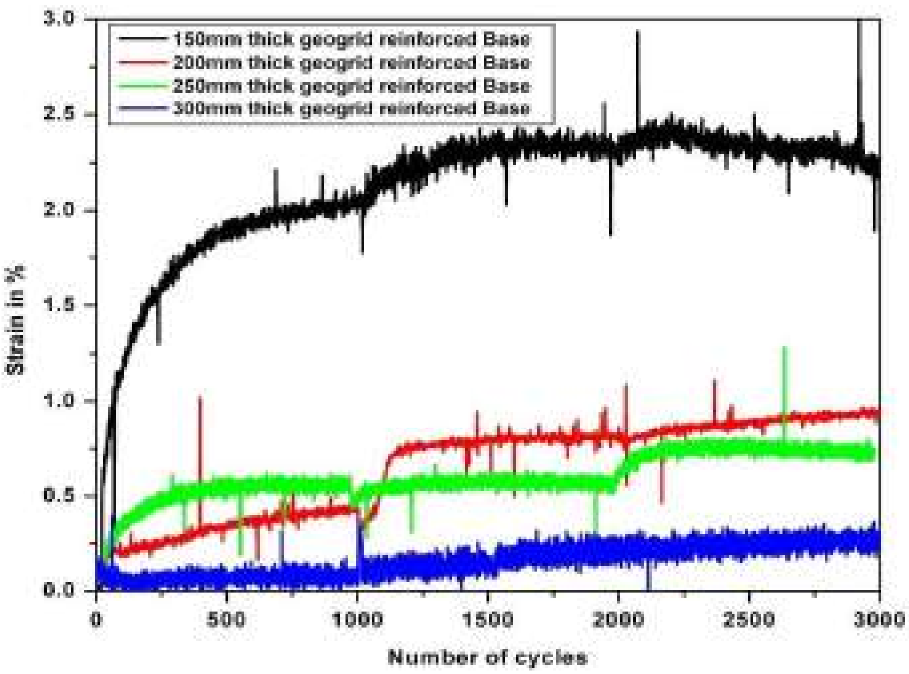


Stress Distribution Angle

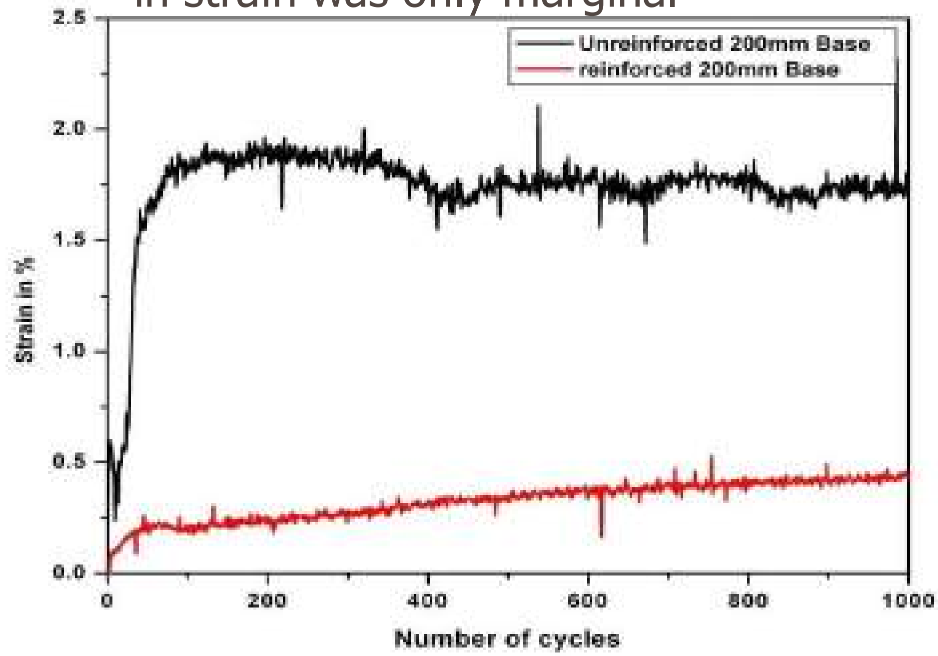


- The initial stress distribution angles were very high for all the sections including both the reinforced and unreinforced sections and it decreased with increase in number of cycles.
- All the geogrid reinforced sections were observed to have better stress distribution angle than the 300mm unreinforced section
- Geogrid reinforcement is effective in reducing the pressure transferred to the top of the subgrade by distributing the load to a wider area compared to the unreinforced section.

Strain on the subgrade



- For 300mm base thickness, the ultimate strains was less than 0.4% while for 150mm reinforced base layer, the subgrade strains exceeded 2%.
- After the initial cycles, the increase in strain was only marginal



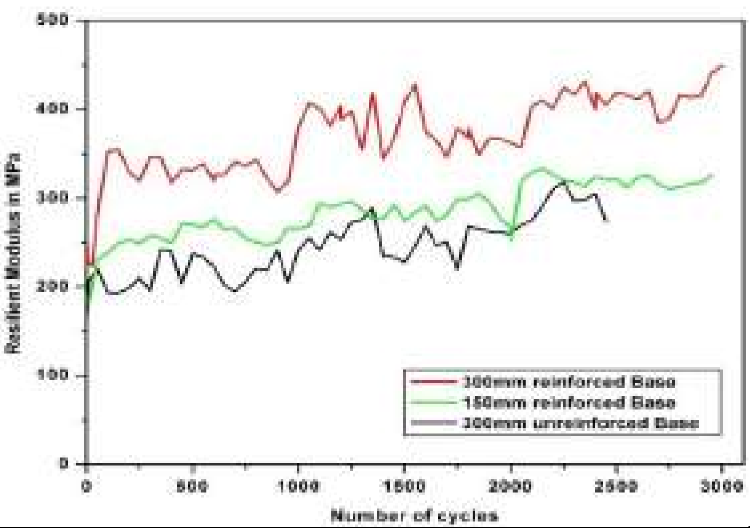
- The geogrids reduced the strains from1.8% for the unreinforced section to 0.4% for the reinforced

Resilient Modulus

The resilient modulus of the base section was back calculated from the resilient deformations of the plate load studies.

$$\delta = \frac{BIq(1-v^2)}{E}$$

δ : Resilient deformation of a particular cycle
B: Diameter of the loading plate
q: pressure applied on the top



- The geocell reinforced section of least base layer thickness, 150mm has resilient modulus higher compared to the unreinforced section of base layer thickness 300mm.

- As the number of cycles increases, the resilient modulus values also increases which is in accordance with the shake down theory of pavements.

Auto-correlation function and scale of fluctuation

Scale of fluctuations for unreinforced and reinforced sections

Section	Mean (MPa)	Standard of Deviation	COV (%)	Scale of fluctuation (cycles)
Unreinforced 200mm	197	45	22.84	40
Unreinforced 250mm	230	46	20	49
Unreinforced 300mm	258	47	18.21	45
Geogrid reinforced 200mm	277	45	16.24	87
Geogrid reinforced 250mm	307	43	14	92
Geogrid reinforced 300mm	380	47	12.36	97

Material Properties - Road mesh

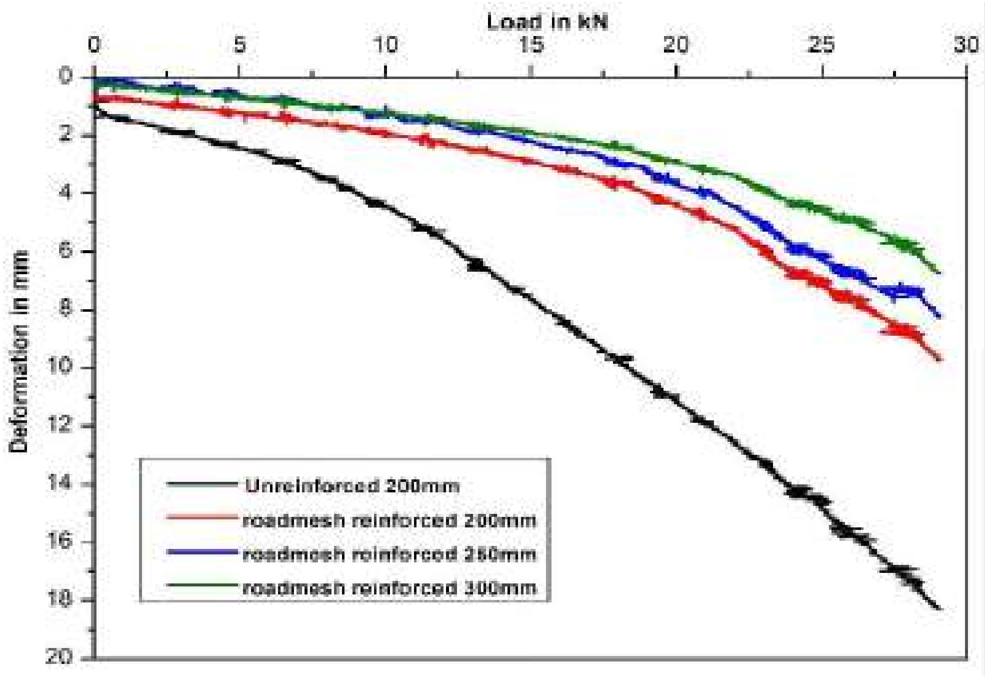
The road mesh used in the present study is a double twisted steel wire mesh with a tensile strength of 40 kN/m in both direction.

Properties of Road mesh

Properties	Values
Diameter of wire mesh	2.4mm
Diameter of transverse rod	4mm
Ultimate tensile strength	40 kN/m

Road mesh used in the study

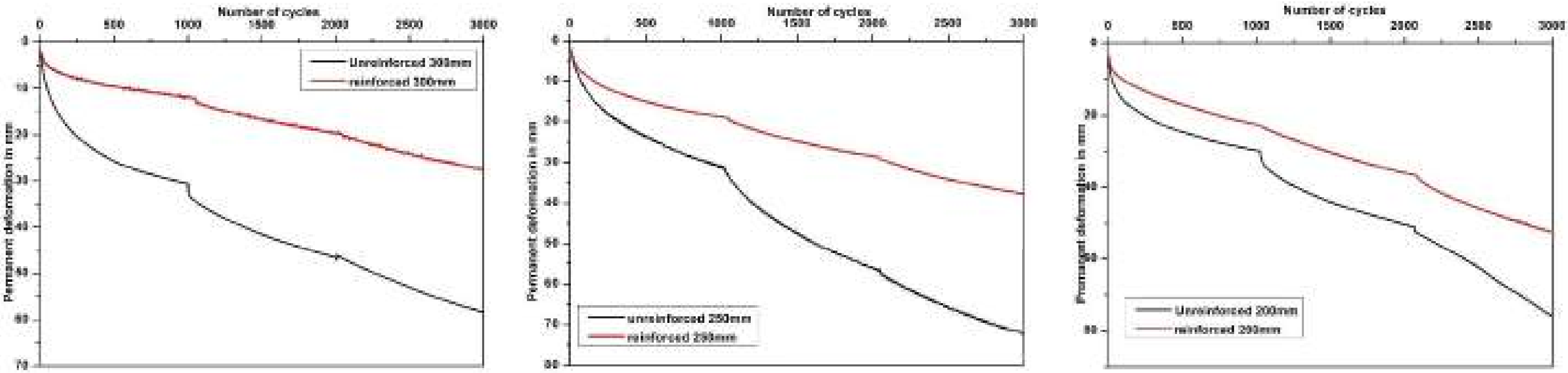
Experimental Results - Static Plate Load Test Results



Load – Deformation results of unreinforced and reinforced sections

Experimental Results - Repeated Plate Load Test Results

Permanent Deformation Studies



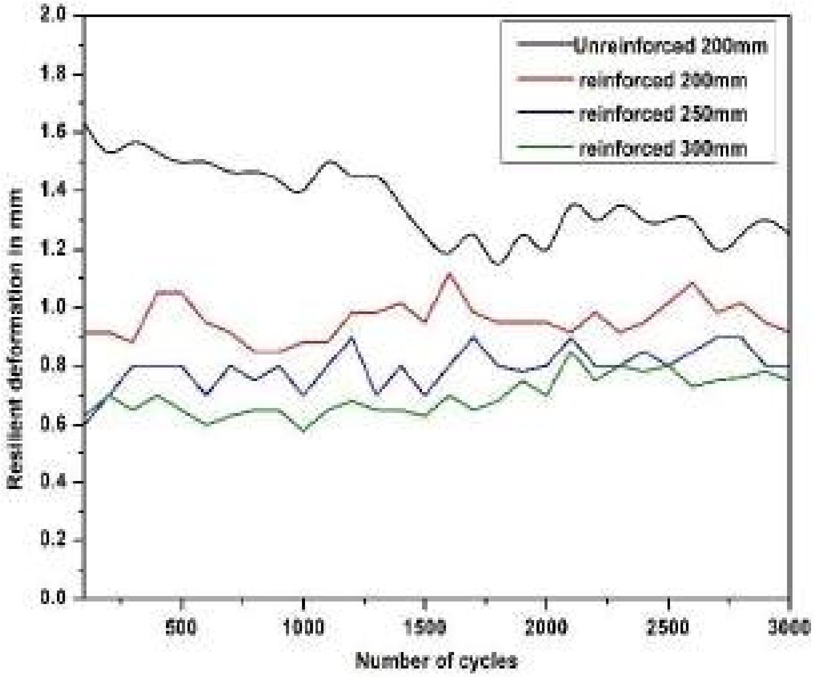
300mm 250mm 200mm

Permanent deformation vs number of cycles for reinforced and unreinforced sections of various thicknesses

- The rate of permanent deformations was more in the case of unreinforced sections when compared to the reinforced section.
- All the road mesh reinforced sections reduced the deformations by at least 40% when compared to unreinforced sections of same thickness

Experimental Results - Repeated Plate Load Test Results

Resilient Deformation Studies



Resilient deformation vs number of cycles

- The rebound was more in the case of unreinforced section compared to that of reinforced section.
- As the thickness of the geocell reinforced base section increases, the resilient deformation decreases.
- The section with lower resilient deformation provided better pavement performance when compared to the section with higher resilient deformation.

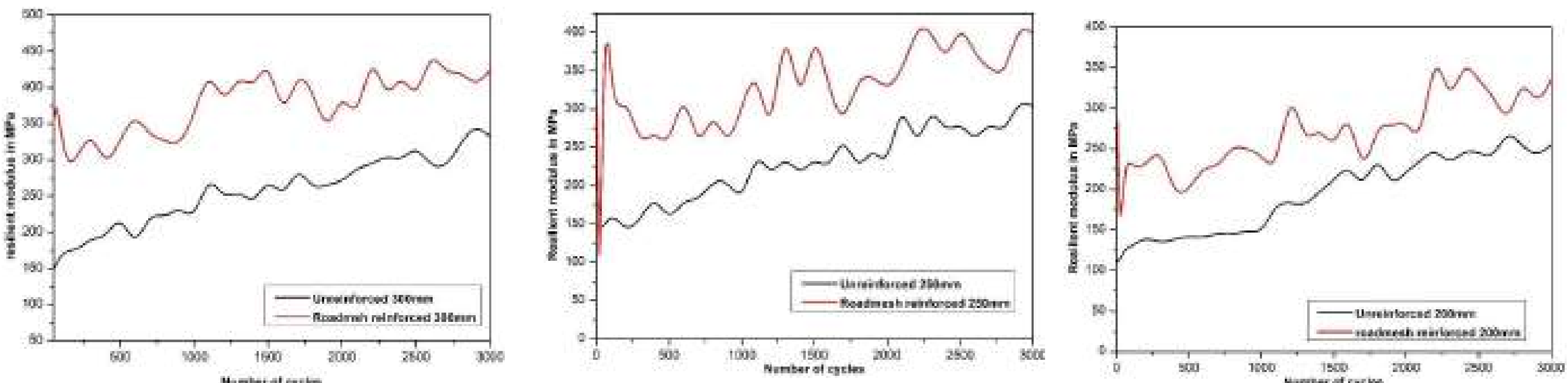
Experimental Results - Repeated Plate Load Test Results

Resilient Modulus

The resilient modulus of the base section was back calculated from the resilient deformations of the plate load studies.

$$\delta = \frac{BIq(1-v^2)}{E}$$

δ : Resilient deformation of a particular cycle
B : Diameter of the loading plate
q : pressure applied on the top



300mm 250mm 200mm

Resilient modulus vs. number of cycles for road mesh reinforced and unreinforced base layers

Experimental Results - Repeated Plate Load Test Results

Modulus Improvement Factor

- This increased modulus of the reinforced base layer is defined by the MIF(Modulus Improvement Factor). Kief et. al (2011) defines the MIF as

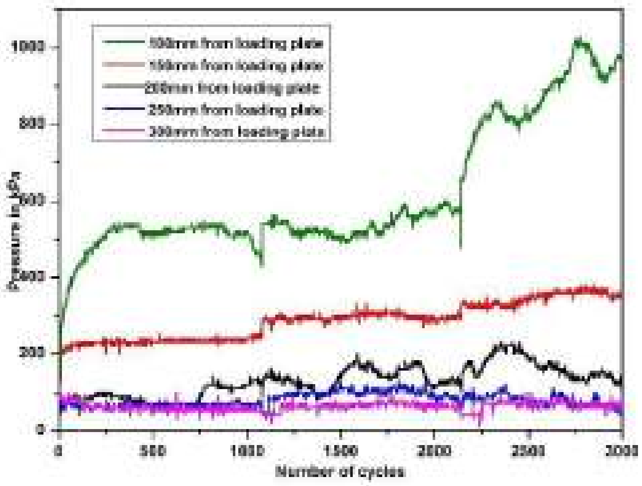
$$MIF = \left(\frac{E_{bc\,(reinforced)}}{E_{bc\,(unreinforced)}}\right)$$

- The unreinforced base layer of 200mm, 250mm and 300mm thickness showed an average modulus of 190MPa, 227MPa and 254MPa.

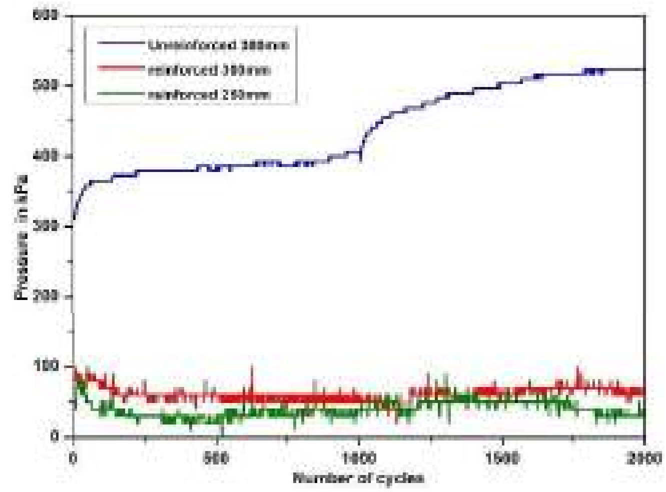
- The road mesh reinforced layer showed an average modulus of 275 MPa, 320 MPa and 370 MPa for 200mm, 250mm and 300mm sections respectively. This gives a modulus Improvement factor of ***1.42***, ***1.4*** and ***1.45*** for 200mm, 250mm and 300mm respectively.

Experimental Results - Repeated Plate Load Test Results

Pressure distribution in base layer



Pressure at different depths of 300mm road mesh reinforced section

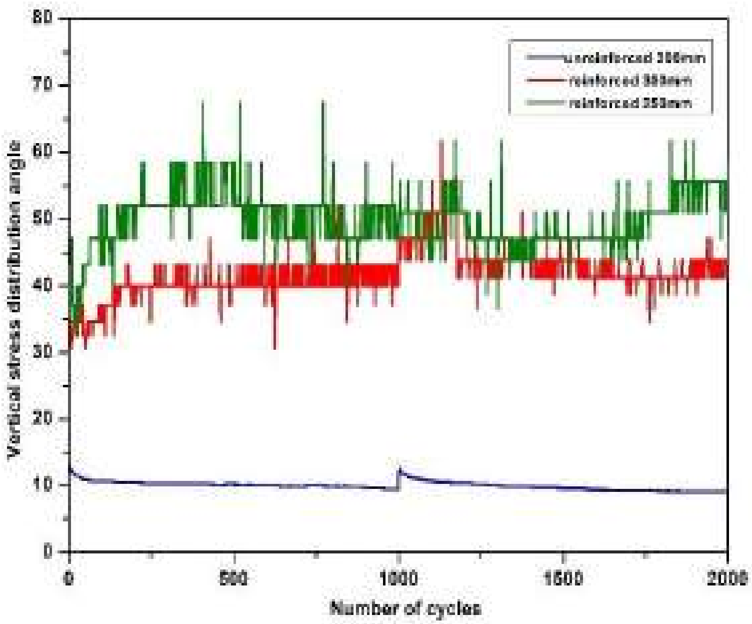


Pressure on top of subgrade for reinforced and unreinforced base layer

- The pressure applied on top of the pavement was reduced by 50% at 100mm depth, 70% at 200mm depth, 80% at 250mm depth and 85% at 300mm depth.
- The road mesh reinforcement along with the base aggregate reduced the pressure getting transferred to the subgrade layer.

Experimental Results - Repeated Plate Load Test Results

Stress Distribution Angle



Stress Distribution angle vs number of cycles

The stress distribution angle is a measurement of the distribution of load by the base layer

$$P_i = \frac{P}{\pi (r + h \tan \alpha)^2}$$

Higher stress distribution angle indicates that the load is getting distributed to wider area and lower stress distribution angle indicates the thinner dissipation of loads

Summary And Conclusions

- The road mesh reinforced sections reduced the permanent deformations compared to the unreinforced section with same base layer thickness by a considerable amount under static as well as repeated loading.
- It was noted from the study that the road mesh reinforcement could reduce more than 40% of the permanent deformation when compared to that of unreinforced section with a lesser deformation rate.
- The reinforced sections showed more resilient modulus values when compared to the unreinforced section which explains the lesser permanent and resilient deformations of reinforced sections.
- The reinforced sections showed a MIF values ranging from 1.4 to 1.45 depending upon the thickness of the reinforced section.
- The study showed a 40% increased stress distribution angle values for reinforced section when compared to the unreinforced section.
- The study clearly showed the effectiveness of road mesh reinforcement in reducing the deformations and thereby reducing the pressure getting transferred to subgrade level leading to increased pavement life.

Field Studies

PROJECT SITE

Longitudinal cross section

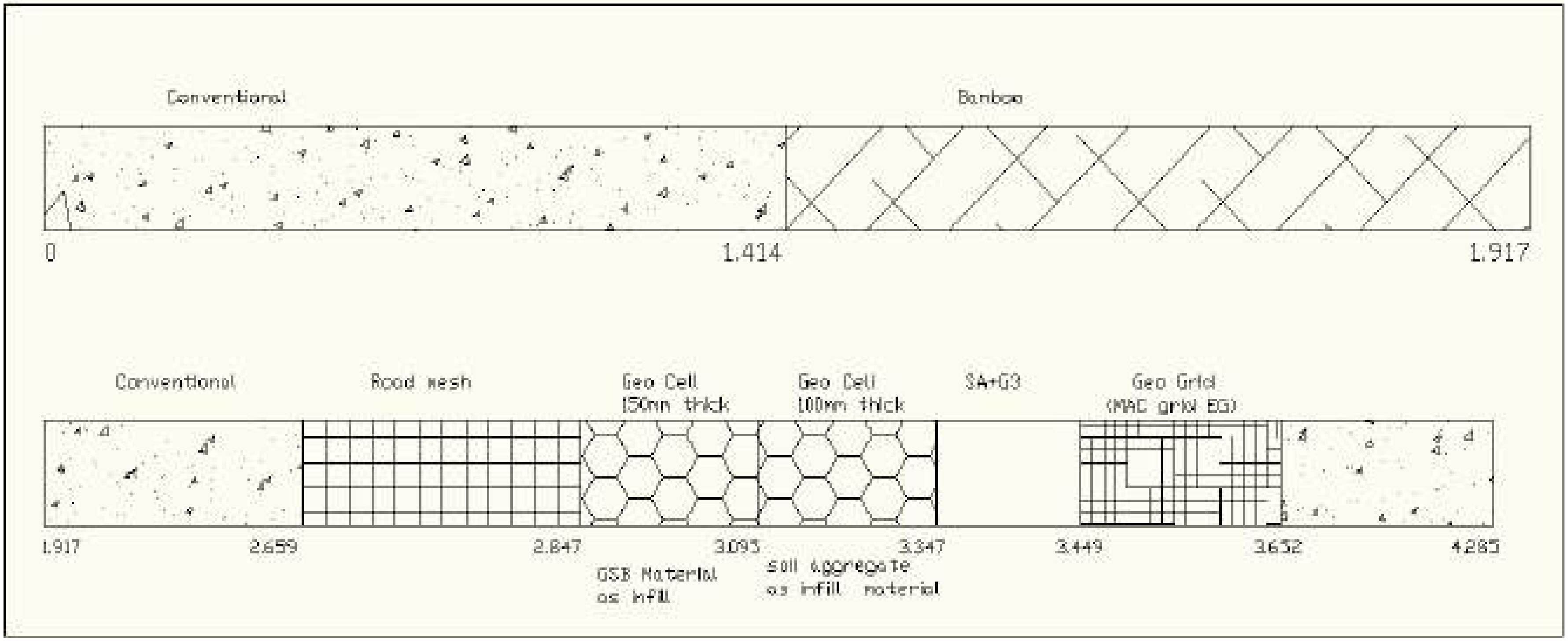


Table- Cross section details of different experimental sections

Type of Experimental Section	Cross section details
Conventional	20mm PMC, 150 mm Granular base, 125mm GSB, compacted subgrade
Geocell-150mm	20mm PMC, 150mm geocell infilled GSB, compacted subgrade
Geocell-100mm	20mm PMC, 100mm Geocell in filled GSB, compacted subgrade
Bamboo	20mm PMC, 250 mm Granular base infilled in bamboo grid, 125mm GSB, compacted subgrade
Road Mesh	20mm PMC, 200 mm Granular base infilled in road mesh, 125mm GSB, compacted subgrade
Geogrid	20mm PMC, 100 mm Granular base above geogrid, 125mm GSB, compacted subgrade

Fig. Laying of geocell and placing of infill

Fig. Spreading and anchoring geogrid

Fig. Laying of bamboo and placing of infill

Webinar on Geo-synthetics for Coastal Protection and Port works
10th and 11th Dec 2020

Prepared Test Pit

GEOCELL

GEOGRID

BAMBOO

Resilient modulus values of Different Component Layers Based on FWD Data (I Year)

Type of experimental section	Chainage		Length of section (m)	E-value s of different component Layers								
	From (km)	To (km)		Pre-mix carpet	Water bound macadam	Bamboo with soil aggregate	Road mesh + soil aggregate	150mm geocell with GSB	100mm geocell with GSB	Geogrid with soil aggregate	GSB	Subgrade
Conventional-1	0.9	1.414	514	566	757	-	-	-	-	-	377	127
Bamboo	1.414	1.917	503	667	-	498	-	-	-	-	305	86
Conventional-2	1.917	2.659	742	405	563	-	-	-	-	-	199	91
Road mesh	2.659	2.847	188	444	-	-	451	-	-	-	304	75
Geocell-150mm	2.847	3.095	248	520	-	-	-	248	-	-	-	54
Geocell-100mm	3.095	3.347	252	533	-	-	-	-	432	-	-	81
Geogrid	3.449	3.652	203	428	-	-	-	-	-	561	388	79
Conventional-3	3.652	4.285	633	536	707	-	-	-	-	-	241	114

Resilient modulus values of Different Component Layers Based on FWD Data (III Year)

Type of experimental section	Chainage		Length of section (m)	E-value s of different component Layers								
	From (km)	To (km)		Pre-mix carpet	Water bound macadam	Bamboo with soil aggregate	Road mesh + soil aggregate	150mm geocell with GSB	100mm geocell with GSB	Geogrid with soil aggregate	GSB	Subgrade
Conventional-1	0.9	1.414	514	533	656	-	-	-	-	-	256	114
Bamboo	1.414	1.917	503	565	-	310	-	-	-	-	148	73
Conventional-2	1.917	2.659	742	514	358	-	-	-	-	-	125	67
Road mesh	2.659	2.847	188	539	-	-	338	-	-	-	157	68
Geocell-150mm	2.847	3.095	248	519	-	-	-	207	-	-	-	57
Geocell-100mm	3.095	3.347	252	523	-	-	-	-	244	-	-	78
Geogrid	3.449	3.652	203	525	-	-	-	-	-	410	291	79
Conventional-3	3.652	4.285	633	520	707	-	-	-	-	-	265	112

Materials	Cost of materials in rupees	Total cost in rupees
Geogrid in m^2	139	193.47
Bamboo in m^2	167	316
Geocell in m^2 Height-100mm)	270	387.8
Geocell in m^2 Height-150mm)	415	592
Conventional method in m^3	-	728.12

(Rural road specifications are adopted for the construction of road from Kestur to Mellahallimala in Kollegal Taluk, Chamarajanagar and cost data provided by KRRDA).

Influence of road construction activities in terms of GHG emissions in India (Nanda et al. 2011)

S.No	Work Executed	GHG emis /1000 tonn
1	Earthwork-procurement and execution	4.38 to
2	Excavation in rock & disposal	1.88 to
3	Crushing of rock for aggregate production	2.74 to
4	Granular sub-base course (GSB) construction	11.40 t
5	Wet mix macadam construction	13.59 t
6	Preparation & Construction of bituminous mixes	90.12 t
7	Preparation & Construction of dry lean concrete	128.45
8	Preparation & Construction of cement concrete pavement	223.83

Implications

- For a rural road 5.5m wide, there is a thickness reduction of GSB to the extent of 0.15m leading to savings of 1898.5 t of GSB material per km.
- This leads to reduction in carbon foot print of 21.6 t per km which is significant.

DEVELOPMENT OF DESIGN CHARTS

- **Analytical Model by Yang and Han (2013):**

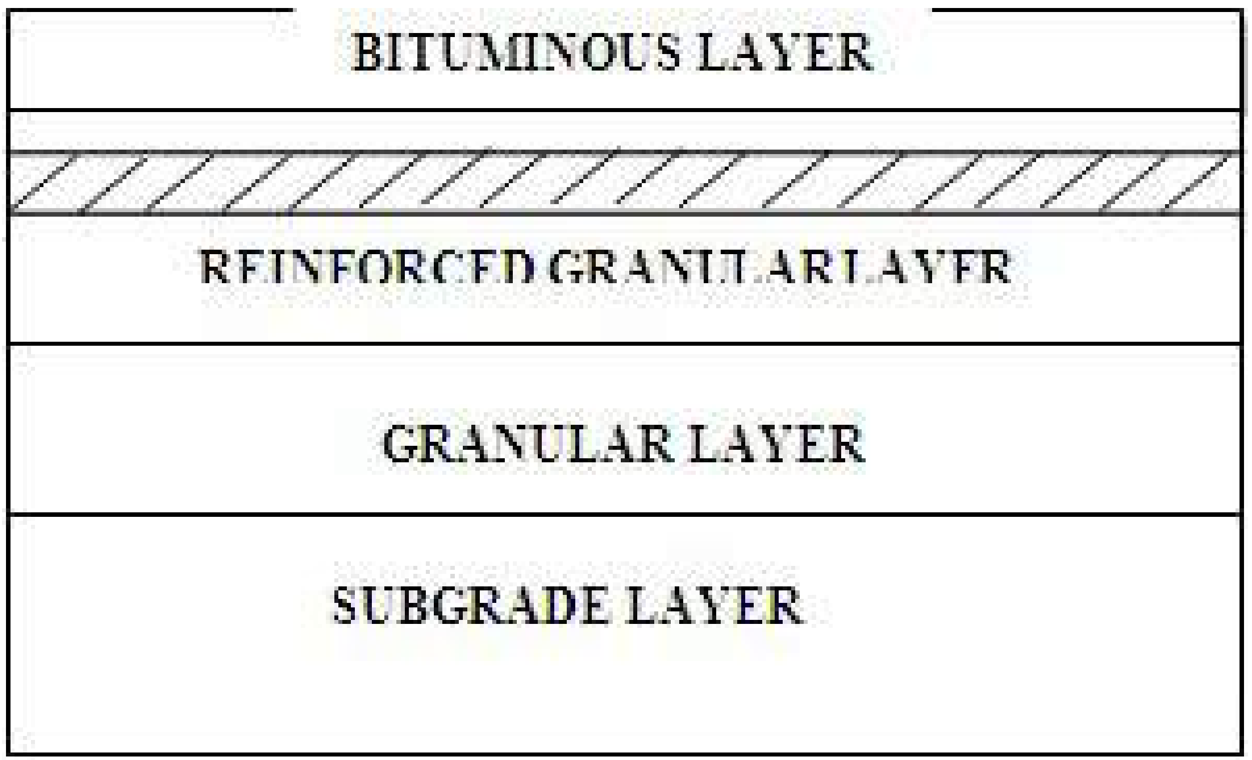
The additional confining pressure induced by the geocell reinforcement was calculated using the following equation:

$$(\Delta\sigma_3) = \frac{M}{D}\left[-\frac{\Delta\sigma_3}{M_{r,1}} + \frac{\sigma_1 - (\sigma_3 + \Delta\sigma_3)}{M_{r,2}}\right]\left(\frac{\varepsilon_0}{\varepsilon_r}\right)^{e^{\left(\frac{\rho}{Nlim}\right)\beta}}\left(\frac{1+sin\psi}{1-sin\psi}\right)$$

Where: M=tensile stiffness of the sample (kN/m); D= diameter of the sample (m); N_{lim}= number of cycles; ε_o, ε_r, ,ρ, β,=resilient model parameters; and ψ=dilation angle

Input parameters for reinforced UGM properties of geosynthetics

k_1	k_2	k_3	$\varepsilon_0/\varepsilon_r$	ρ	β	ψ
450	0.56	0	150	2000	0.3	0^o



Typical section considered for analysis

Pavement Design Catalogues for IRC: SP: 72-2007

Sub grade strength (CBR)	Cumulative ESAL Applications						
	10,000 to 30,000	30,000 to 60,000	60,000 to 1,00,000	1,00,000 to 2,00,000	2,00,000 to 3,00,000	3,00,000 to 6,00,000	6,00,000 to 1,000,000
Very poor (CBR = 2)	200 100	75 150 100	75 100 100 100	75 100 100 150	75 100 150 150	75 100 225 150	75 150 200 225
Poor (CBR = 3 to 4)	200	275	75 100 150	75 100 100 100	75 100 100 150	75 150 100 150	75 150 150 150
Fair (CBR = 5 to 6)	175	250	275	75 100 125	75 100 150	75 100 100 100	75 150 100 100
Good (CBR = 7 to 9)	150	175	225	75 100 100	75 100 125	75 100 150	75 150 150
Very Good (CBR =10 to 15)	125	150	175	75 150	75 100 100	75 100 125	75 150 125

- All dimensions are in mm

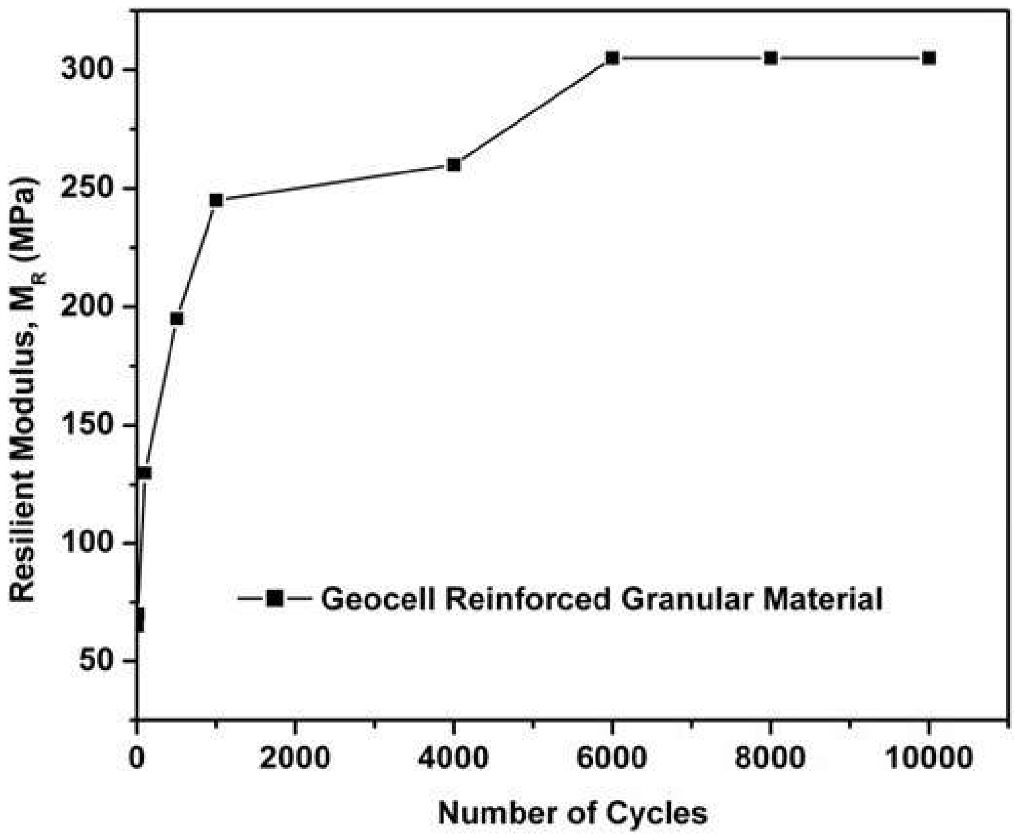
Bituminous surfacing

Base of gravel (CBR not less than 100)

Gravel base (CBR not less than 80)

Granular sub base (CBR not less than 20)

Modified soil/improved subgrade (CBR not less than 10)



Resilient modulus versus number of cycles
(through Yang et. al (2013) method)

Modified Pavement Design Catalogues for IRC: SP: 72-2007

Sub grade strength (CBR)	Cumulative ESAL Applications						
	10,000 to 30,000	30,000 to 60,000	60,000 to 1,00,000	1,00,000 to 2,00,000	2,00,000 to 3,00,000	3,00,000 to 6,00,000	6,00,000 to 1,000,000
Very poor (CBR = 2)	150	75 150	75 175	75 200	75 250	75 300	75 350
Poor (CBR = 3 to 4)	125	175	75 150	75 200	75 250	75 300	75 325
Fair (CBR = 5 to 6)	125	200	200	75 175	75 200	75 225	75 275
Good (CBR = 7 to 9)	125	150	175	75 150	75 175	75 200	75 250
Very Good (CBR =10 to 15)	125	150	150	75 150	75 200	75 200	75 200

Bituminous surfacing   Geocell reinforced granular material   (All dimensions are in mm)

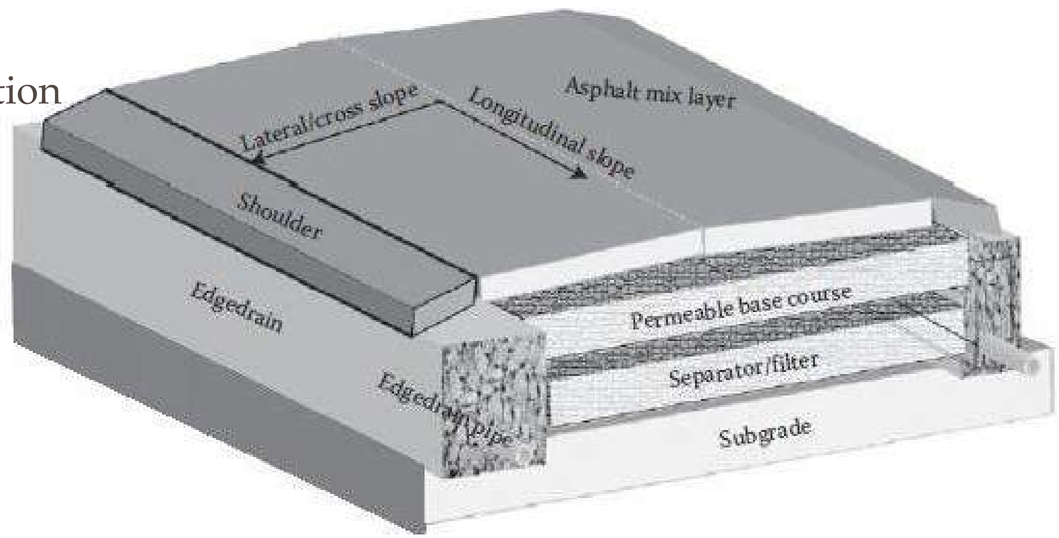
Pavement Drainage

➢ **Surface Drainage System**
Function: To reduce Surface Infiltration

- Longitudinal and Cross slope
- Impermeable surface course
- Sealing joints and cracks

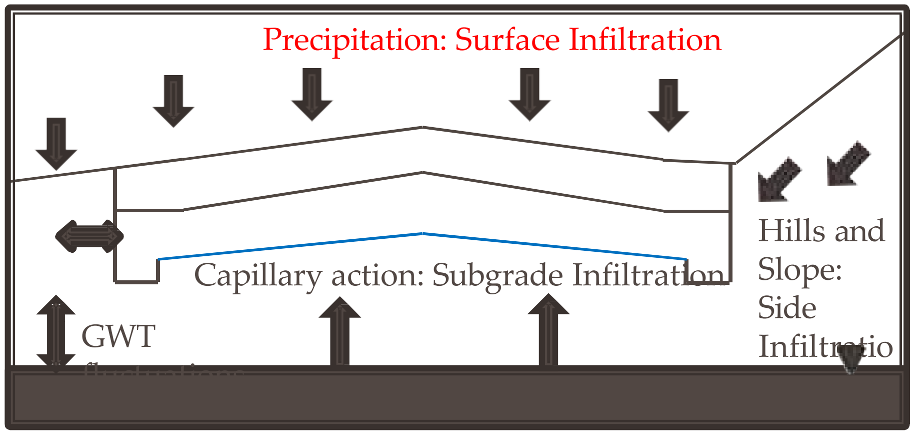
➢ **Subsurface Drainage System**
Function: To rapidly drain off the infiltrated moisture

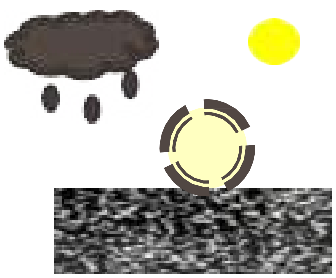
- **Drainage/Permeable layer**
- Filter/Protective layer
- Edge drain system



Source: Mallick, R. B., and El-Korchi, T. (2009). "Pavement Engineering Principles and Practice", CRC Press. pp. 129

- All Pavement systems are exposed to weathering actions of nature



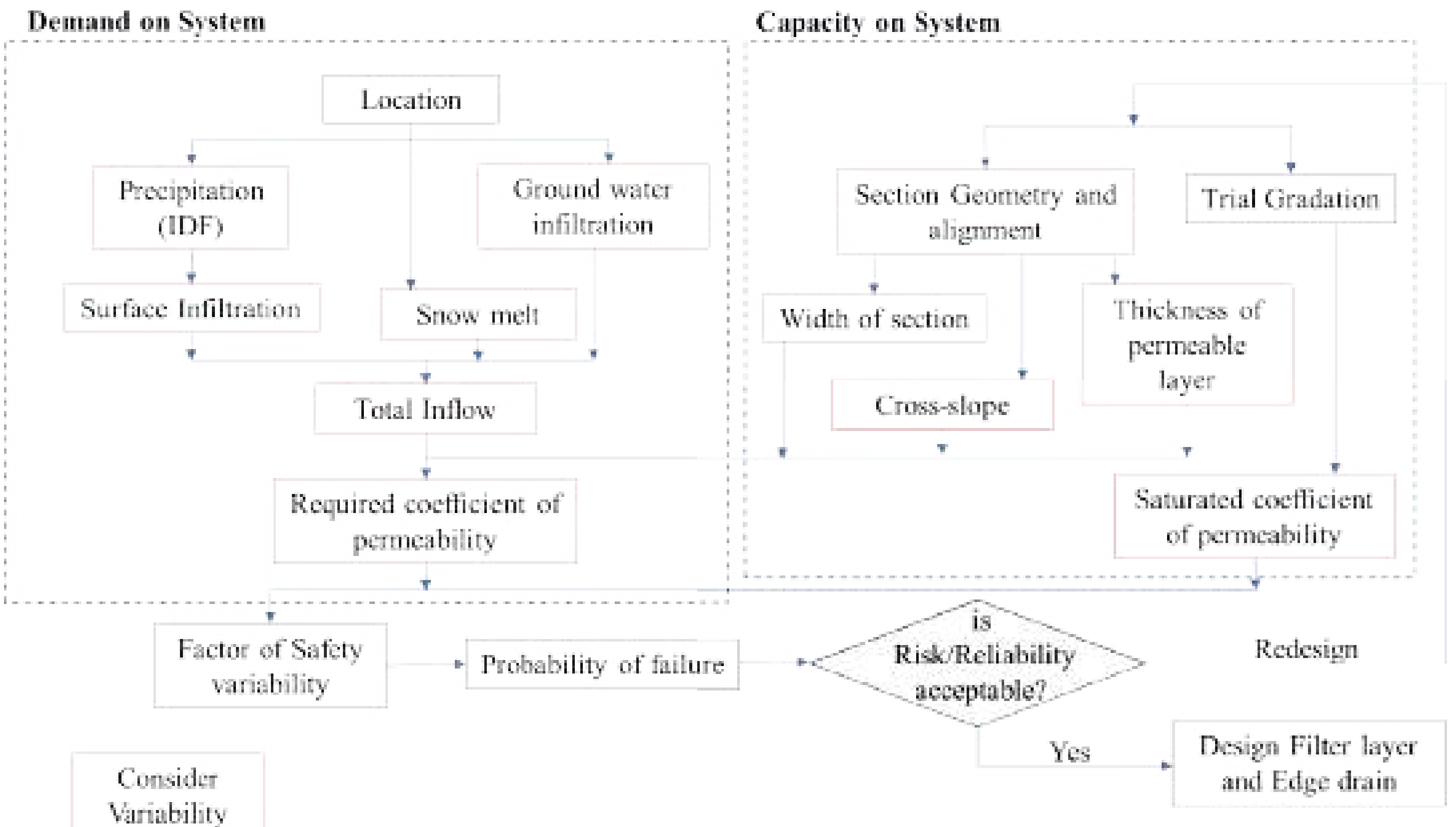


Loading

Distresses

Failure

Risk Analysis of Permeable Layer



Framework for risk-based design of the pavement subsurface drainage system

Risk Assessment for Indian Condition

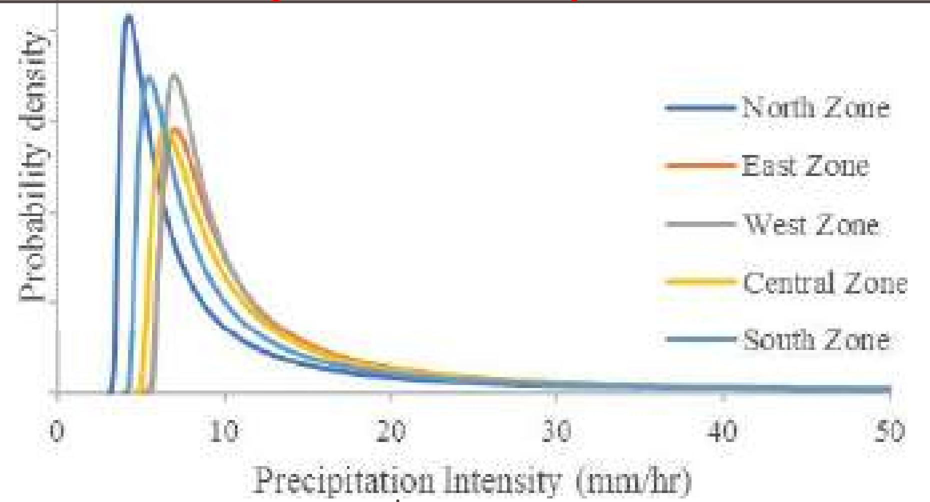
Ram Babu et al. (1979)

$$I = \frac{K.T^a}{(t+b)^n}$$

K, a, b and *n* are region specific constants
T is~Frequency/Return period
t is precipitation duration

t U (0 – 24 hour)

Precipitation Variability for India



I

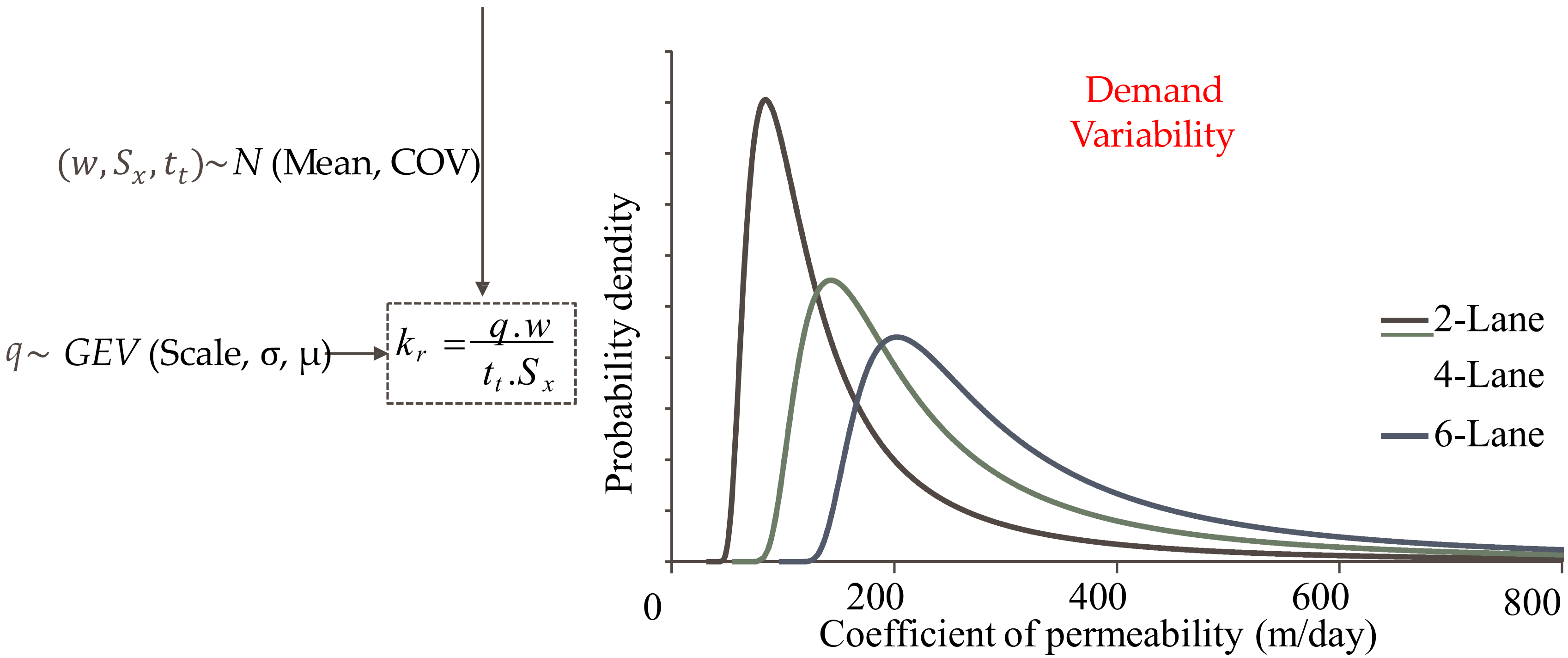
Infiltration = *I* * *U ~ (0.33 – 0.67)* + Groundwater fluctuations

Infiltration Coefficient

66

Table 1 - Cross-section details as per number of lane (IRC SP:73) 2007)

Parameter	**2-lane**	**4-lane**	**6-lane**
One-way width (m)	5	8.5	12
Cross slope (%)	2.5	2.5	2.5
Thickness (m)	0.25	0.25	0.25



Risk Assessment for Indian Conditions

Table 3- Probability of failure (p_f) of permeable layer for Mumbai region

Gradation	1	2	3	4	5	6
2-Lane	1	0.963	0.332	0.032	0.006	0.004
4-Lane	1	1	0.684	0.090	0.016	0.013
6-Lane	1	1	0.896	0.169	0.031	0.025

p_f	
0.00 – 0.05	Good
0.06 – 0.20	Good
0.21 – 0.80	Good
0.81 – 1.00	Poor

*Risk = Probability of failure * Consequences*

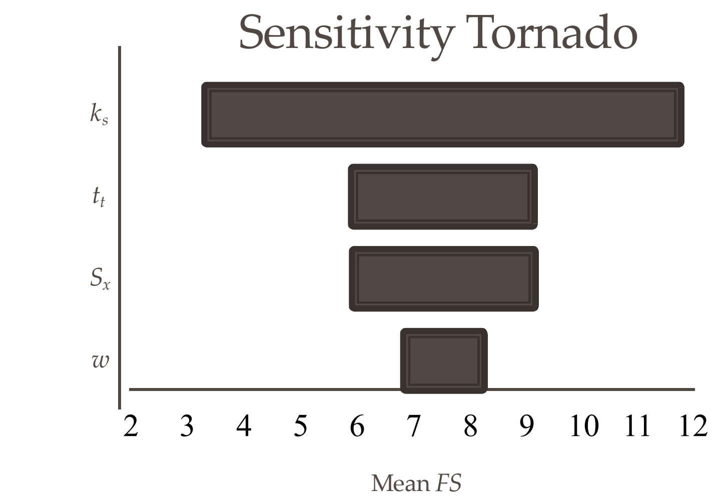
Table 4- Monetary values of the consequences (Shailendra and Veeraragavan, 2010)

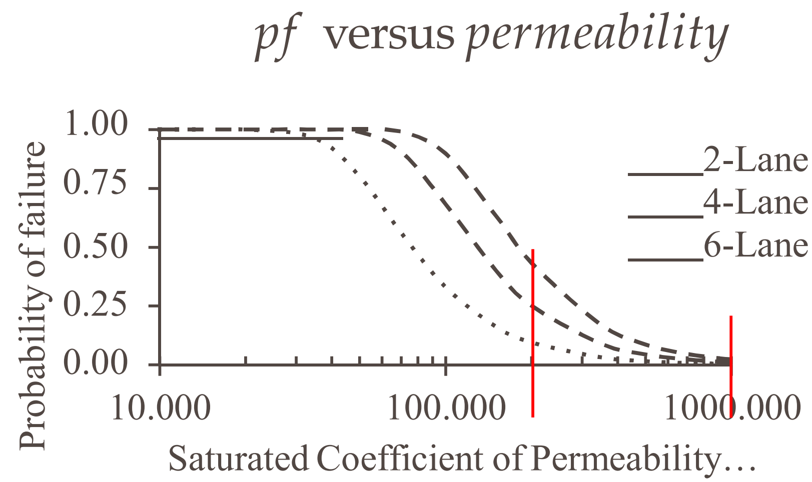
Subsurface drainage Type	**Consequences (lakhs Rupees)**		
	Overlay cost (/km/lane)	VOC	Total
Poor	67.35	6039	6106.35
Good	32.77	5940	5972.77

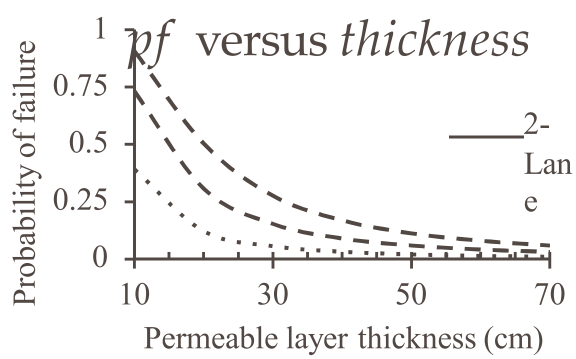
Table 5- Economic Risk for Indian highways (lakhs Rupees/km/lane)

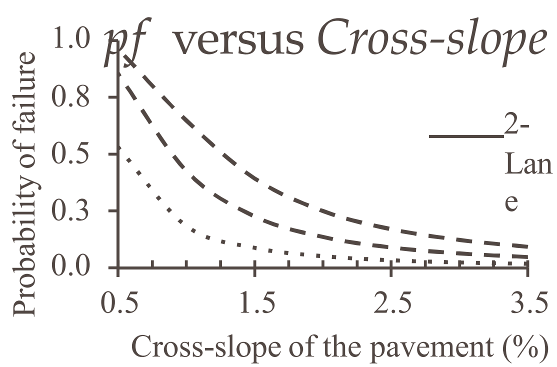
Gradation	**1**	**2**	**3**	**4**	**5**	**6**
2-Lane	6106.35	5880.41	2027.30	191.13	35.84	23.89
4-Lane	6106.35	6106.35	4176.74	537.55	95.56	77.65
6-Lane	6106.35	6106.35	5471.29	1009.40	185.16	149.32

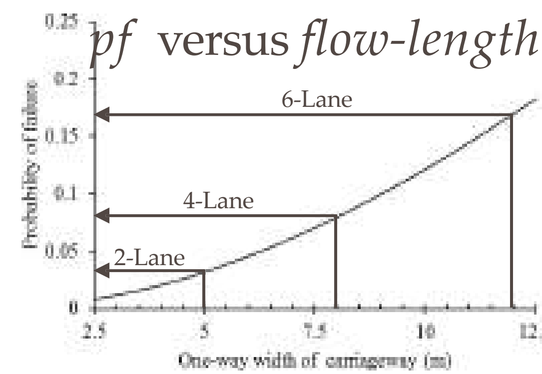
Risk Assessment for Indian Condition











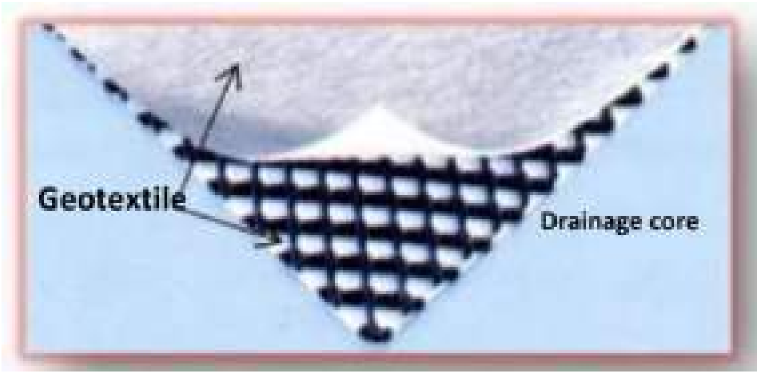
Risk Assessment for Indian Condition

Table 6- Required saturated coefficient of permeability (k_s) for India at desirable reliability

Zone	Reliability $R = 1\text{-}p_f$	Thickness (cm)	Camber (%)	Required k_s as per type of highway (m/day)		
	R	t_t	S_x	2-Lane	4-Lane	6-Lane
Northern	0.95	25	2.5	295	490	690
	0.90	25	2.5	205	350	490
	0.85	25	2.5	165	280	385
	0.80	25	2.5	140	240	330
Central	0.95	25	2.5	435	725	950
	0.90	25	2.5	310	510	705
	0.85	25	2.5	245	410	570
	0.80	25	2.5	210	350	490
Western	0.95	25	2.5	450	750	1050
	0.90	25	2.5	315	530	715
	0.85	25	2.5	250	420	590
	0.80	25	2.5	215	365	510
Eastern	0.95	25	2.5	465	785	1085
	0.90	25	2.5	330	550	760
	0.85	25	2.5	260	445	610
	0.80	25	2.5	225	375	520
Southern	0.95	25	2.5	370	620	865
	0.90	25	2.5	255	435	600
	0.85	25	2.5	260	350	485
	0.80	25	2.5	210	300	415

Geocomposites

- Tri planar composite
- EXTERNAL FILTER - GEOTEXTILE Structure: Needle punched or thermally bonded non-woven geotextile

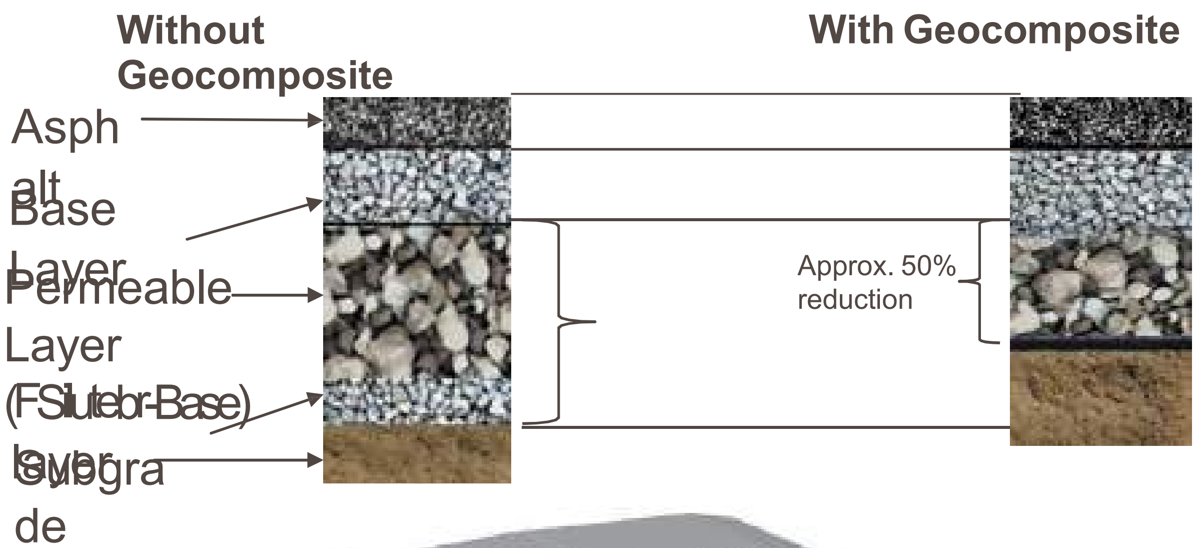


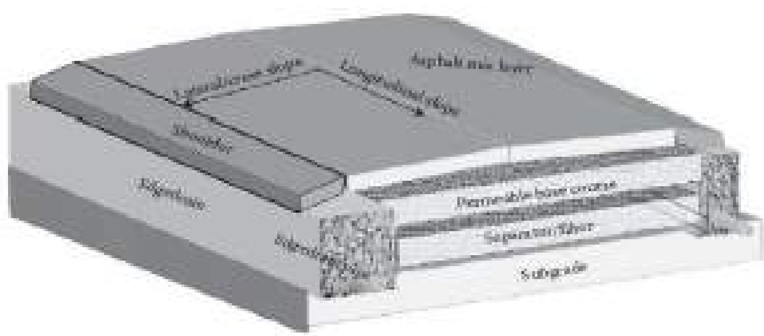
- DRAINAGE CORE - GEONET
  - Structure: Sets of parallel ribs overlaid and integrally connected having a rhomboidal shape
  - Raw Material: High density polyethylene, stabilized by carbon black, black colour

Advantages of use of Geocomposite in Pavements

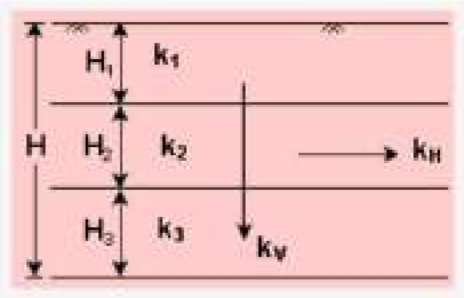
- Reduction in thickness of GSB considering both
  - Structural strength/stability
  - Drainage
- Act as Filter: Greater reliability compared aggregate filter
- Erosion control
- The in-plane permeability plays an important role due to relative permeability differences in Subgrade and Geocomposite
- Cost effective in terms of life cycle cost

Pavement Subsurface Drainage System

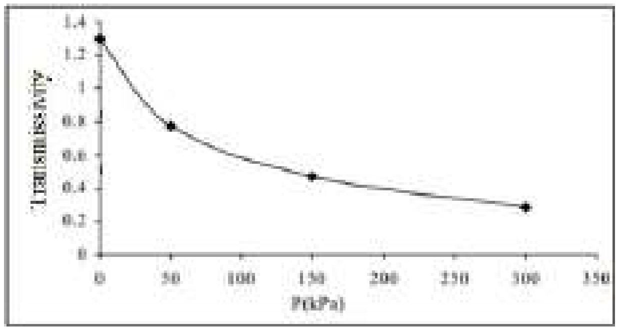




Equivalent Permeability



$$k_H = \frac{[k_1H_1 + k_2H_2 + k_3H_3]}{H_1 + H_2 + H_3}$$



$$k_{eq} = \frac{1}{H_{gsb} + H_{gt}} [r_{gtr} + k_{gsb}H_{gsb}]$$

where, k_{eq} is the equivalent permeability;

H_{gsb} is the height of granular subbase;

H_{gt} is the thickness of geotextile;

k_{gsb} is the saturated coefficient of permeability of granular subbase;

r_{gtr} is the residual transmissivity of the geotextile (at 250 kPa normal stress);

SUMMARY

- From the studies it was observed that the geosynthetic reinforcement in the unbound granular layers effectively reduces the permanent deformation compared to the unreinforced section.
- For the same thickness, it was noted that the permanent deformation per cycle was much higher for unreinforced section compared to the reinforced section
- The resilient deformation for the reinforced section was much lower when compared to the resilient deformation of the unreinforced section
- Field studies clearly showed the effectiveness of geosynthetics in pavements

REFLECTIONS ON PAVEMENT DESIGN

Use of resilient modulus and permanent deformation as function of load repetitions may be considered as design basis as these parameters can be evaluated both in the laboratory and field using standard procedures

Field studies clearly showed the effectiveness of geosynthetics in pavements. Hence, road construction agencies should be able to use them to have cost effective/better performing roads

Drainage design of pavements needs to be considered along with strength design and the issues of pavement deterioration need to be examined.

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Thank you for your attention

SLOPE STABILITY ANALYSIS
AND
DESIGN OF POSONALLAH GUIDE BUND ALONG
DIBANG RIVER, ARUNACHAL PRADESH

Prof. R. Sundaravadivelu, F.N.A.E
Prof. Nilanjan Saha
Department of Ocean Engineering,
Indian Institute of Technology, Madras

Contents

Introduction

- Dibang River, also known as Talo in Idu, is a tributary river of the Bramhaputra that originates and flows through the Mishmi Hills and northeast India from the state of Arunachal Pradesh.

- Between Bomjir (Nizamghat) and Sadiya, the Dibang has a steep river gradient and exhibits braided channel morphology with its width varying from 4 to 9 kilometres.

- It often changes its course resulting in flooding and destruction of cultivable land and forests along its banks.

- Due to heavy rains, the flow of water in the Dibang river had increased considerably causing severe damages to the existing guide bund along the Dibang River.

Scope of Work

Phase-I (Preparation of Preliminary Project Report)

i. To carry out review of desktop studies on hydrology of the Dibang river.

ii. To provide a suitable design and shore protection methodology using gabion boxes for construction of the guide bund.

iii. To perform numerical analysis using PLAXIS 2D for the proposed shore protection methodology and check stability against sliding for the proposed model.

iv. Submission of preliminary project report including the details of the analysis and shore protection methodology.

Phase-II (Preparation of Detailed Project Report)

i. The detailed engineering of the shore protection methodology selected in the PPR stage.

ii. List of drawings and list of designs.

iii. Methodology of construction and its sequence.

iv. Submission of Detailed Project Report including Good for Construction Design and Drawing, Bill of Quantity and Cost Estimate.

Site Location

•Coordinates of Dibang Stretch:

✓Start: 28° 06'54.1"N 95° 49'10.9"E

✓End: 28° 09'50.8"N 95° 39'38.1"E

Site location of the Dibang stretch
(Source: Google Earth)

Proposed Location of Guide Bund

Site Photos (Damage of Guide Bund Near A6 towards U/s Side, on 7th June 2020)

Site Photos


Proposed Guide bund location
Proposed Repelling spur location

Site Photos (Further damages of Guide Bund Near A6 towards U/s Side between June and July 2020)

Site Photos (Further damages of Guide Bund Near A6 towards U/s Side between June and July 2020)

Site Photos (Further damages of Guide Bund Near A6 on 20th and 21st July 2020)

Site Photos

Site Photos

Site Photos

Review of Hydrology Study

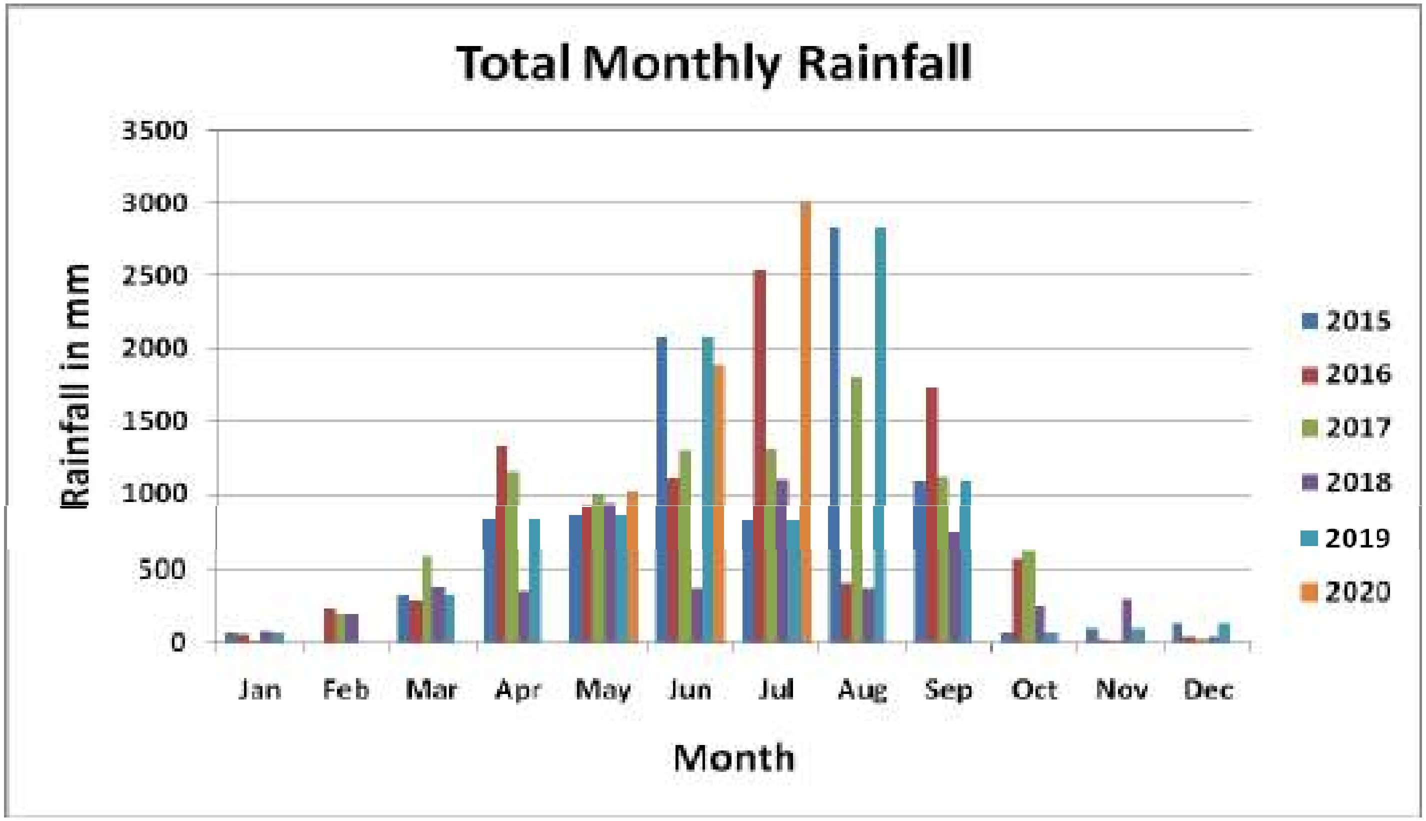
Catchment Area (Source - Invent Grid India (P) Ltd)

- The Dibang valley district is the major catchment area of river Dibang, accounting to nearly 80% of the catchment area.
- The measured catchment area of River Dibang up to Poshanalla is 11273.066 Sq. KM as per ISRO Bhuvan Geo Portal Data.

Monthly Rainfall Data
(Source - Water Resources Division, Roing, Arunachal Pradesh)

	Total Rainfal in mm					
	2015	**2016**	**2017**	**2018**	**2019**	**2020**
Jan	64.77	54.61	11.43	79.24	26	-
Feb	0	233.68	196.85	192.41	282	-
Mar	327.18	292.3	584.2	378.46	310.3	-
Apr	841.14	1330.99	1156.2	357.14	349	-
May	869.55	925.75	990.78	929.29	653.58	1022.5
Jun	2082.15	1107.44	1306.85	372.04	647.21	1883.65
Jul	834.39	2543.54	1312.63	1101.41	2095.9	3015.6
Aug	2833.37	402.59	1802.39	372.04	606.3	-
Sep	1092.32	1737.09	1114.84	757.05	714	-
Oct	69.85	572.77	623.29	252	370.1	-
Nov	101.6	15.24	10.16	300	11.2	-
Dec	134.59	52.07	25.4	49	19.2	-
Total Yearly Rainfall	**9250.91**	**9268.07**	**9135.02**	**5140.08**	**6084.79**	**5921.75**

Monthly Rainfall Data

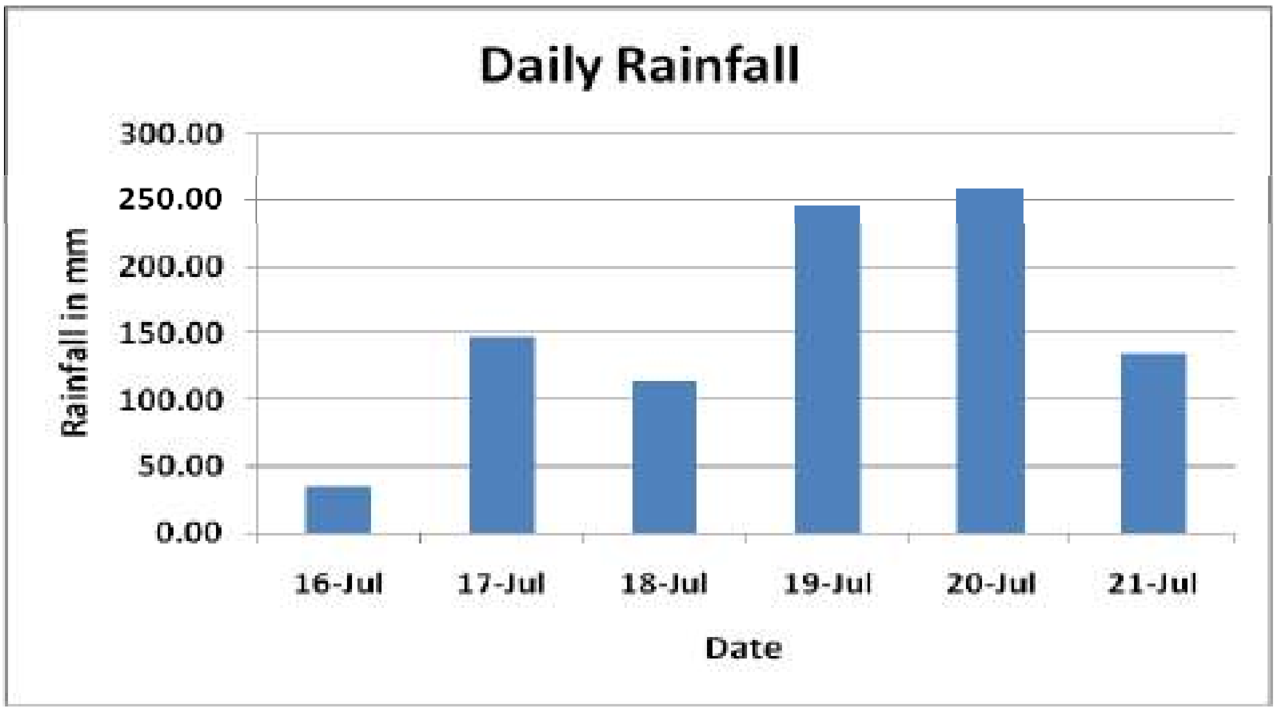


It is clear that the total monthly rainfall since January 2015 has been maximum in the month of July 2020.

Daily Rainfall Data

(Source - Water Resources Division, Roing, Arunachal Pradesh)

- Figure shows the daily rainfall from 16th July 2020 to 21st July 2020 for Dibang Valley district which is the major catchment area of river Dibang.
- Approximately more than 224mm rainfall was recorded on 19th and 20th July 2020, which is much more than the actual rainfall as per IMD.



Discharge considered for Posonallah Major Bridge

As per “Feasibility Report on Construction of Road Bomjur – Meka (NH-52) including bridges across Dibang river system, 2009”

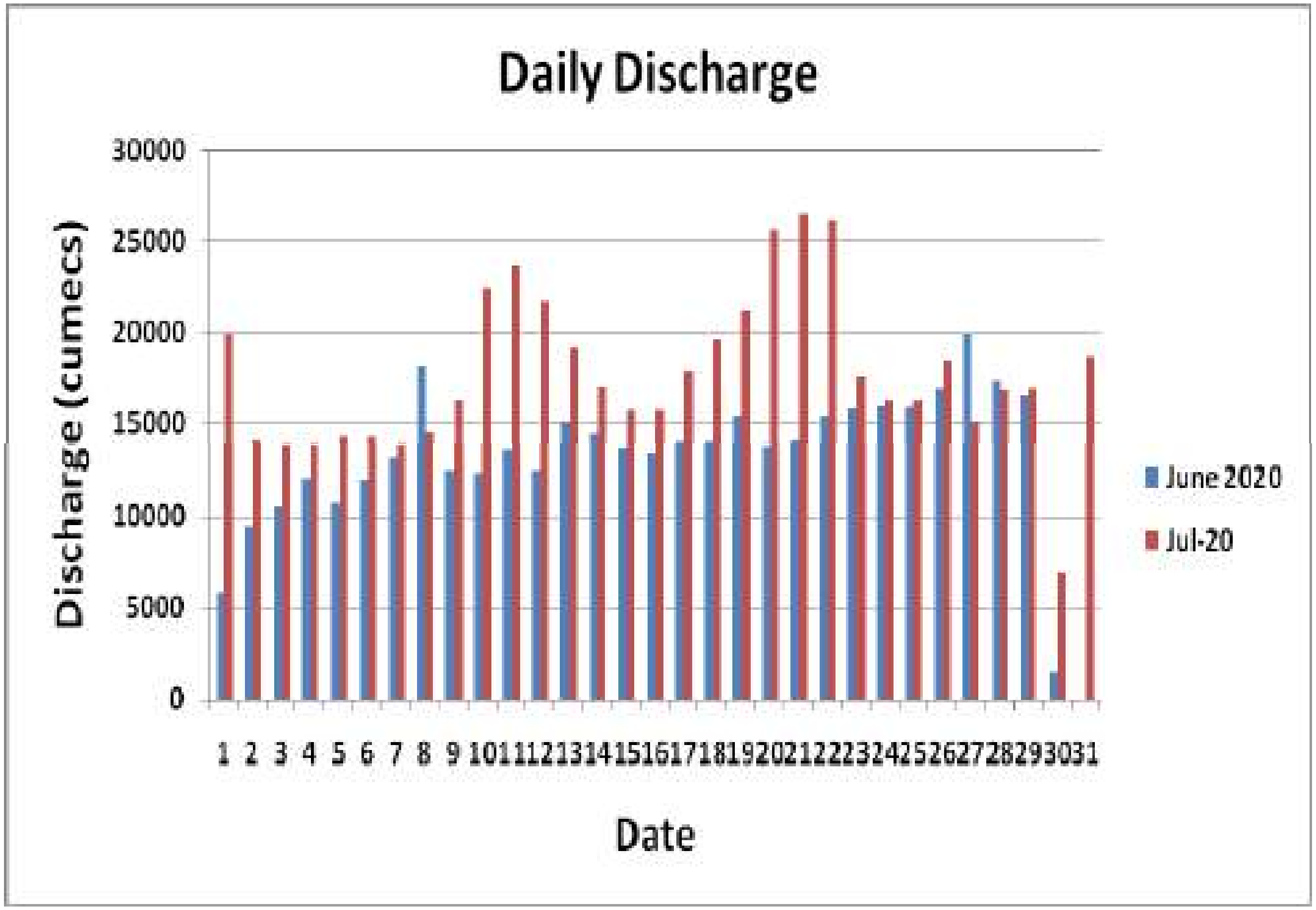
Q_{50} = 22,000 Cumecs
Q_{100} = 29,700 Cumecs

Discharge Data – June 2020 & July 2020

(Source - Water Resources Division, Roing, Arunachal Pradesh)

June	Discharge (cumecs)	July	Discharge (cumecs)
June 1st	5827.33	July 1st	19939.45
2	9435.55	2	14064.3
3	10590.3	3	13950.22
4	12074.83	4	13950.22
5	10724.02	5	14235.42
6	12011.95	6	14235.42
7	13197.4	7	13950.22
8	18148.42	8	14520.62
9	12518.81	9	16231.83
10	12319.49	10	22423.88
11	13607.73	11	23737.22
12	12570.68	12	21767.21
13	15035.24	13	19140.54
14	14490.84	14	16960.89
15	13747.11	15	15716.56
16	13479.22	16	15716.56
17	14000 (assumed)	17	17827.2
18	14000 (assumed)	18	19608.18
19	15374.44	19	21207.2
20	13793.17	20	25605.26
21	14047.57	21	26513.79
22	15331.02	22	26210.95
23	15818.32	23	17586.43
24	16027.68	24	16238.58
25	15866.91	25	16238.58
26	16938.71	26	18476.93
27	19939.45	27	15092.92
28	17372.64	28	16777.72
29	16517.03	29	16912.51
30	1452.62	30	6913.5
		31	18597.31

Discharge Data – June 2020 & July 2020



Discharge Data – June 2020 & July 2020

- It is clear that the Dibang river had recorded exceptionally heavy discharge of water on 20 July, 21 July and 22 July 2020, the maximum being 26513.79 cumecs on 21.07.2020, and this may be considered close to the 1 in 100 peak discharge (29700 cumecs).
- The discharge values have breached the 1 in 50 years discharge (22000 cumecs) on the following days:

10 July 2020 – 22423.88 cumecs
11 July 2020 – 23737.22 cumecs
20 July 2020 – 25605.26 cumecs
21 July 2020 – 26513.79 cumecs
22 July 2020 – 26210.95 cumecs

Highest Flood Level
(Source - Water Resources Division, Roing, Arunachal Pradesh)

- Water level observed on 20.07.2020 = 200.1m.
- Highest flood level (Maximum observed on 21.07.2020) = 200.3m.

Temperature and Precipitation

- The average of past 15 years of weather data has shown that the time period from the month of april to september is most vulnerable for the Dibang River.
- Both the temperature and precipitation are high in the said period.
- This causes heavy flow of water in the Dibang river because of glacier melting due to high temperature and higher precipitation resulting heavy rainfall.
- This natural weather conditions are resulting in flood like situation in the Dibang river during the above-mentioned period.

Satellite Data Analysis - (Imagery Nov 2011 and Feb 2017)

(Source: Google earth)

Satellite Data Analysis

- The Dibang river is currently showing a trend of continually shifting towards right side of the flow direction (in the line of and even going behind the longitudinal alignment of A6 Guide Bund).
- The river is very steep (slope ~ 1/250) and has tendency for scour formation in the right bank.
- A deep channel like formation is noted in February 2017 indicating scour of right bank.

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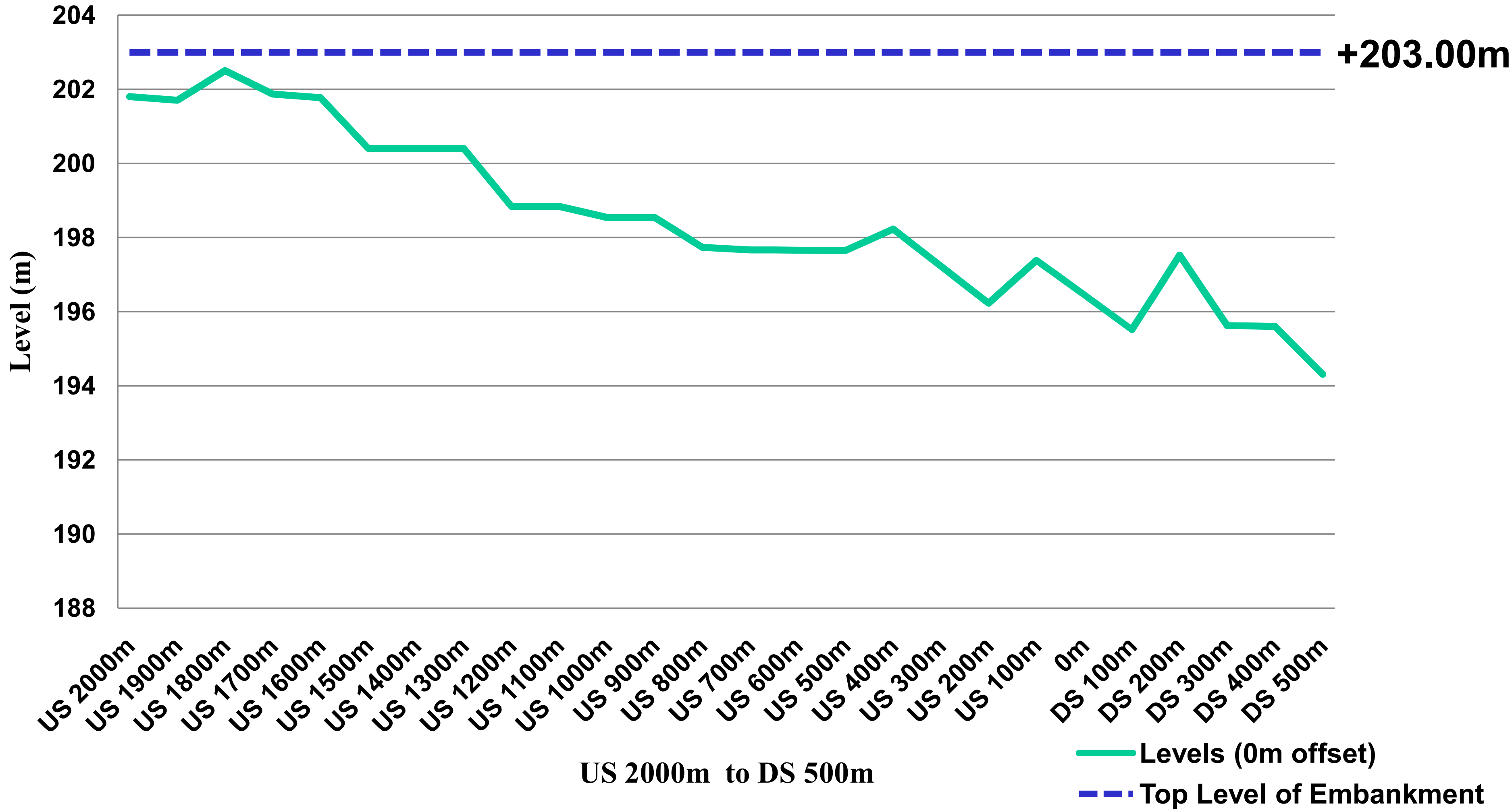
Levels plotted from Upstream to Downstream



Slope ~ 1/250

Webinar on Geo-synthetics for Coastal Protection and Port works

10th and 11th Dec 2020

NATIONAL TECHNOLOGY CENTRE FOR PORTS WATERWAYS AND COASTS NTCPWC

Prof. R.Sundaravadivelu, IITM

Summary of Hydrology Study & Probable Causes for failure of Previous Bund

- The catchment of Dibang river has water inflow due to higher Precipitation resulting in heavy rainfall along with additions from glaciers melting from Himalayan range.
- This year, the Dibang river has recorded heavy rainfall since June and the total monthly rainfall since January 2015 has been maximum in the month of July 2020.
- The previous guide bund was designed for Q_{50} = 22,000 cumecs which was breached many times in July 2020. Moreover, a discharge of 26513.79 cumecs was recorded on 21.07.2020.
- The previous guide bund was designed for a high flood level = 200.03m. However, on 20th and 21st July 2020, the high flood level recorded was 200.1m and 200.3m respectively.
- The Dibang river is currently showing a trend of continually shifting towards right side of the flow direction (in the line of and even going behind the longitudinal alignment of A6 Guide Bund).
- The river is very steep (slope ~ 1/250) and has tendency for scour formation in the right bank.
- A deep channel like formation is noted in February 2017 indicating scour of right bank.

Design Data

Design Discharge

- Design discharge considered : **Q_{100} = 29,700 Cumecs** as per 2009 feasibility report for construction of bridge.

- Observed maximum discharge during the recent floods is **26,513.79 cumecs** (Source - Water Resources Division, Roing, Arunachal Pradesh)

Design High Flood Level

- Highest flood level observed : **HFL = 200.3m**
(Source - Water Resources Division, Roing, Arunachal Pradesh)

Design Velocity

•For all the available cross-sections for the proposed length of the guide bund (550m upstream and 150m downstream), the flow velocity corresponding to 30,000 cumecs are calculated using **the U.S. Army Corps of Engineers' River Analysis System (HEC-RAS) software.**

•The HEC-RAS model runs were performed for a discharge of 30000 cumecs (design discharge = 29,700cumecs).

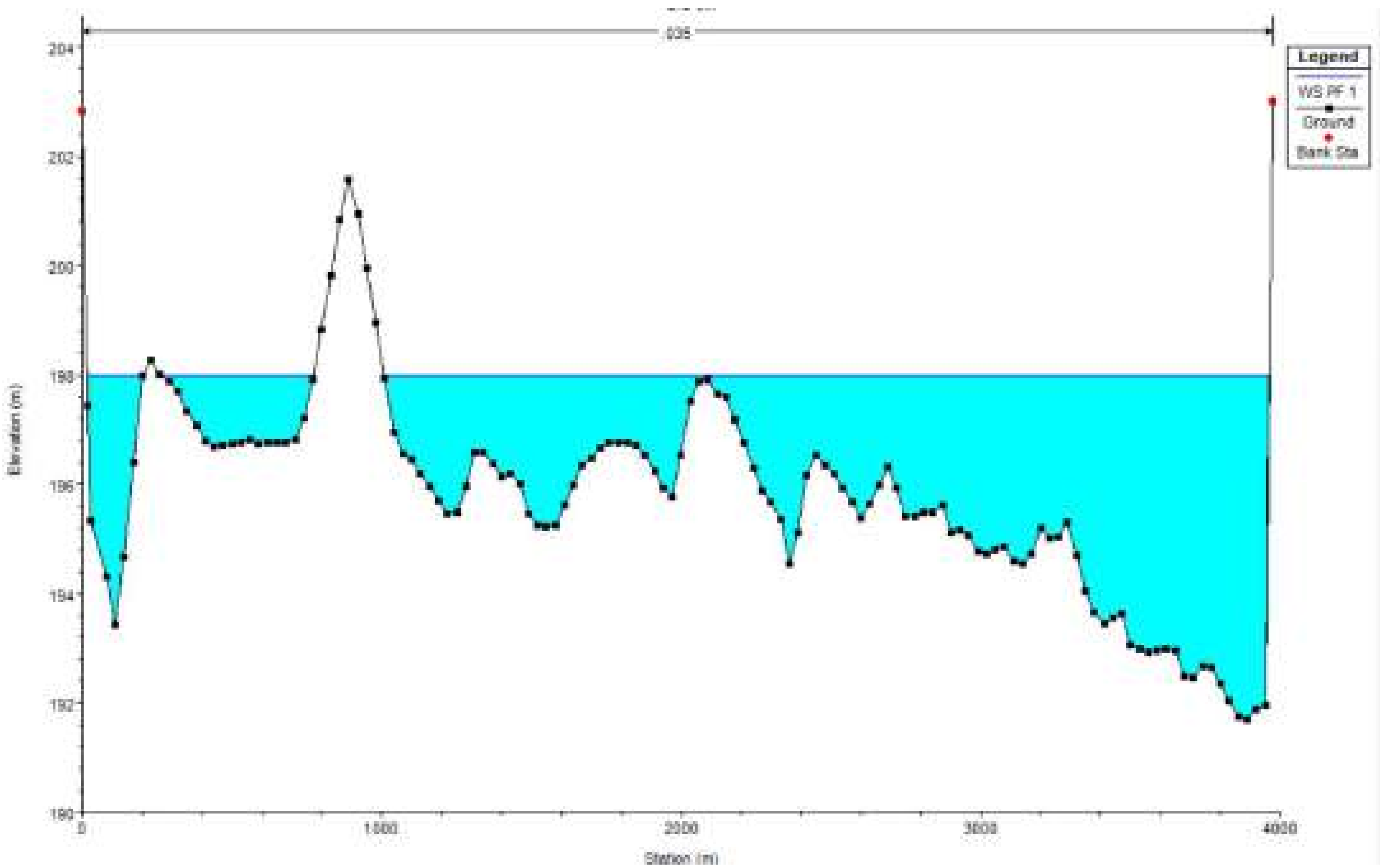
Design Velocity

Results of HEC-RAS software

Cross-section	Flow Area A(m^2)	Discharge Q (m^3/s)	Velocity V (m/s)
US 550m	9980.60	30000	3.01
US 500m	9955.76	30000	3.01
US 450m	9863.80	30000	3.04
US 400m	9871.44	30000	3.04
US 350m	9408.89	30000	3.19
US 300m	9407.79	30000	3.19
US 250m	9380.69	30000	3.20
US 200m	9183.37	30000	3.27
US 150m	9275.86	30000	3.24
US 100m	10031.84	30000	2.99
US 50m	9584.86	30000	3.13
0m (Bridge)	9841.11	30000	3.05
DS 50m	9887.56	30000	3.03
DS 100m	8775.76	30000	3.42
DS 150m	9052.20	30000	3.31

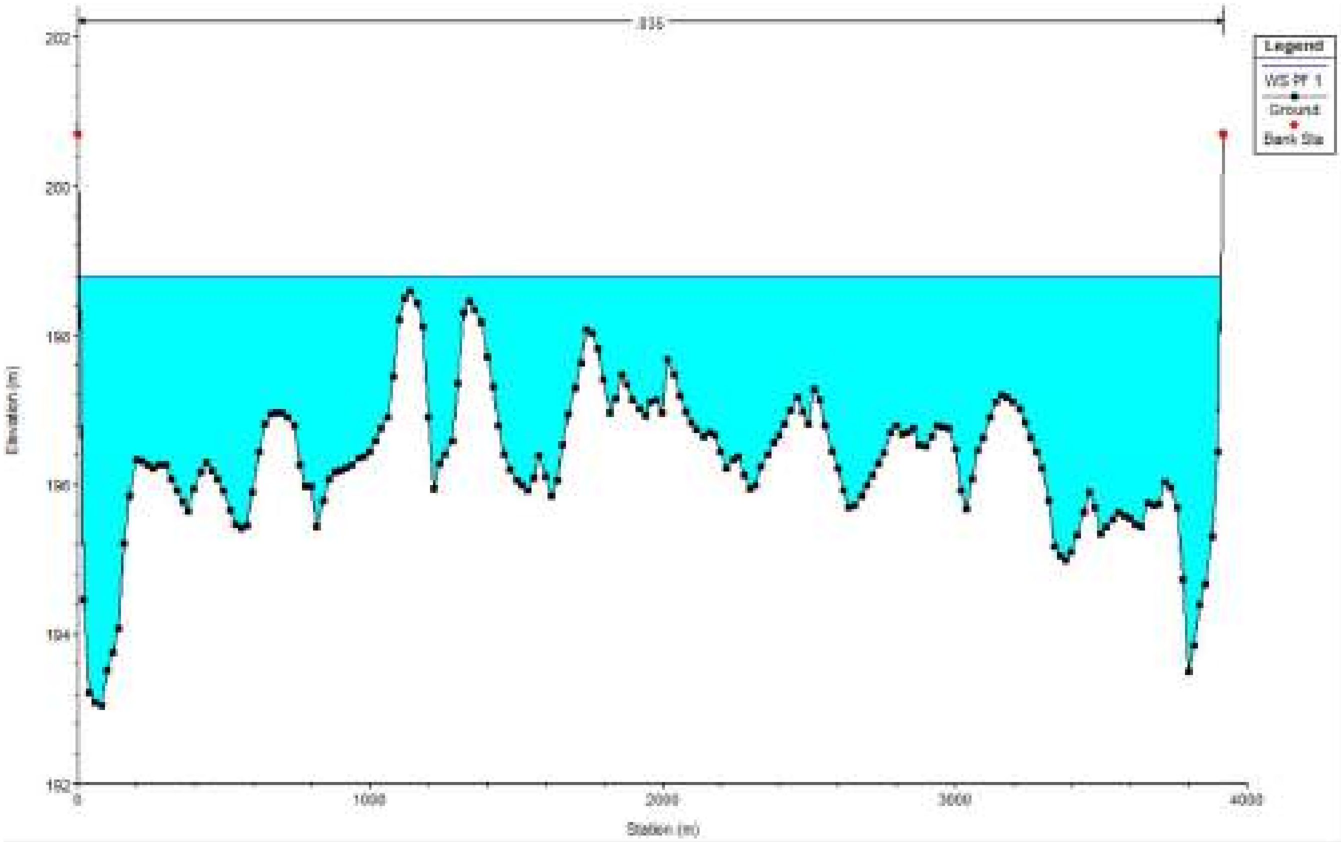
Results of HEC-RAS software

Cross-section (DS 150m)



Results of HEC-RAS software

Typical Cross-section (US)



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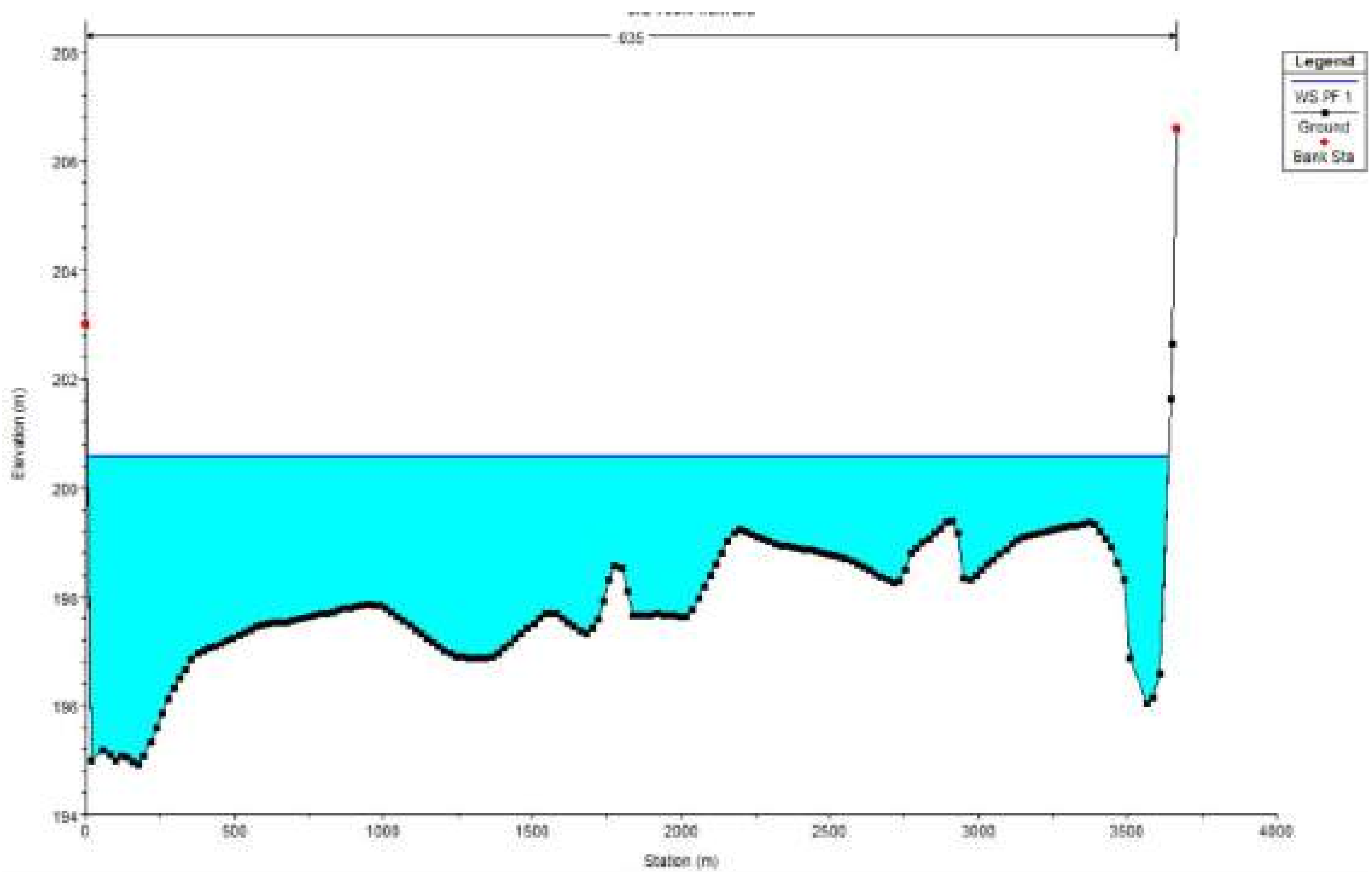
Results of HEC-RAS software

Cross-section (US 550m)

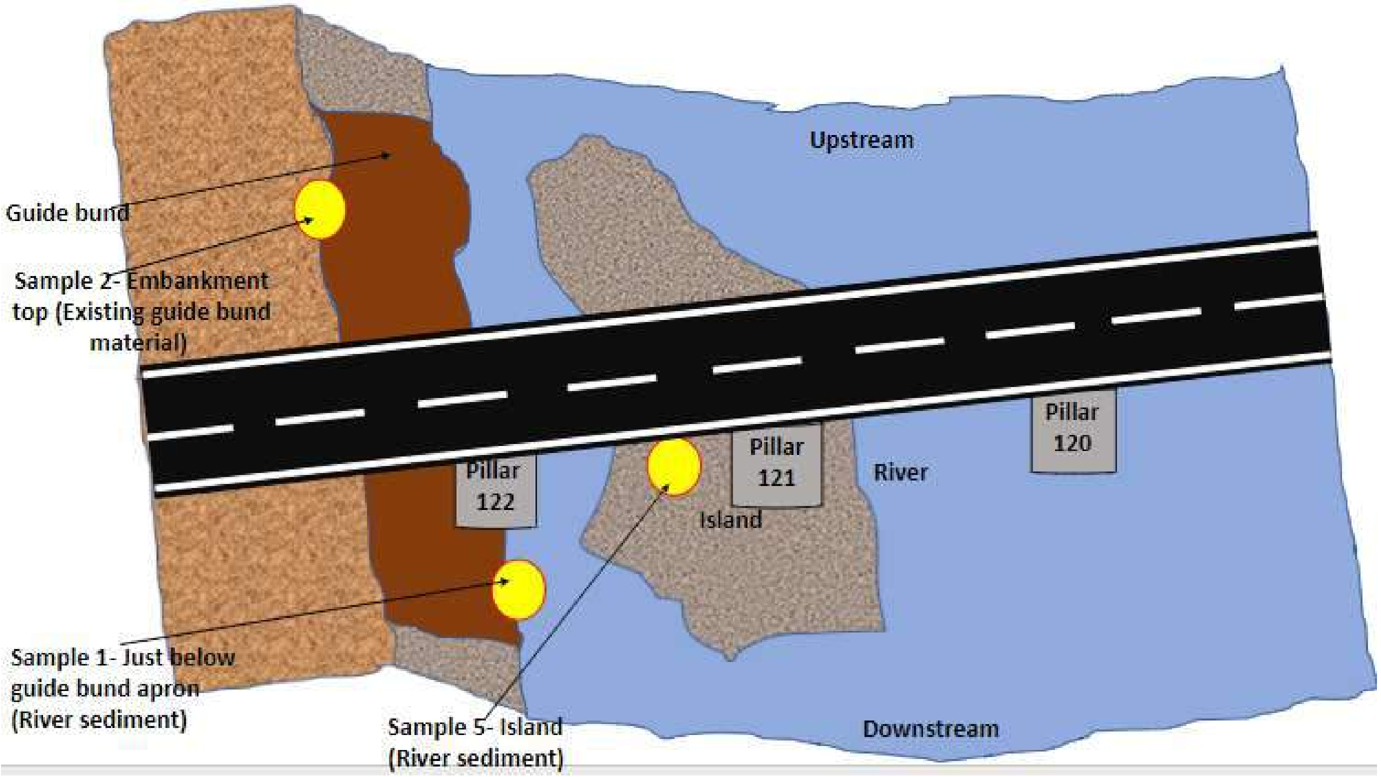


Design Velocity

- The river carries a discharge of 30000m^3/s without overflowing and the flow is subcritical.
- The **average velocity is 3m/s** for 30000m^3/s discharge.
- However, since the exact flow velocity will differ at each location based on various site conditions, **instantaneous velocity = 4m/s** at site has been used for the analysis and design of the Guide bund.

Geotechnical Data

The soil test report for sample 1 – river sediment just below guide bund apron has been considered.



(Source: Dibang Infra Projects Limited)

Geotechnical Data

1	Details of Sample	
	Description of Sample	Sample No.1
	Date of Collection	12.09.2020
	Chainage No.	16+400
	Location	Possonallah River (River Sediment)
	Soil and Rock	Clay and Soil
2	Wet Seive Analysis	
	Sieve Size	% of Passing
	75.00 mm	100
	19.00 mm	92.6
	4.75 mm	69.22
	2.00 mm	50.16
	425mic	16.22
	75 mic	8.02
3	Atterberg Limits	
	Liquid Limit (%)	21.80%
	Plastic Limit (%)	Nil
	Plasticity Index (%)	Non-Plastic
4	Grain Size Analysis	
	Gravel	30.78
	Sand	61.2
	Silt + Clay	8.02
5	Particle Size (mm)	
	D_{10}	0.17
	D_{30}	0.90
	D_{60}	3.00
	D_{15}	0.40
	D_{85}	11.00
	D_{50}	2.00
6	Co-efficient of Uniformity (Cu)	17.65
7	Co-efficient of Curvature (Cc)	1.59
8	Type of Soil	GW-SW

Geotechnical Data

The weighted mean diameter of the bed material as mentioned in the soil test report is 2mm which corresponds to heavy sand as per **Table 1 of IRC 78:2000.**

Type of bed material	d_m	K_{sf}
Coarse silt	0.04	0.35
Silt/fine sand	0.081 to 0.158	0.5 to 0.7
Medium sand	0.233 to 0.505	0.85 to 1.25
Coarse sand	0.725	1.5
Fine bajri and sand	0.988	1.75
Heavy sand	1.29 to 2.00	2.0 to 2.42

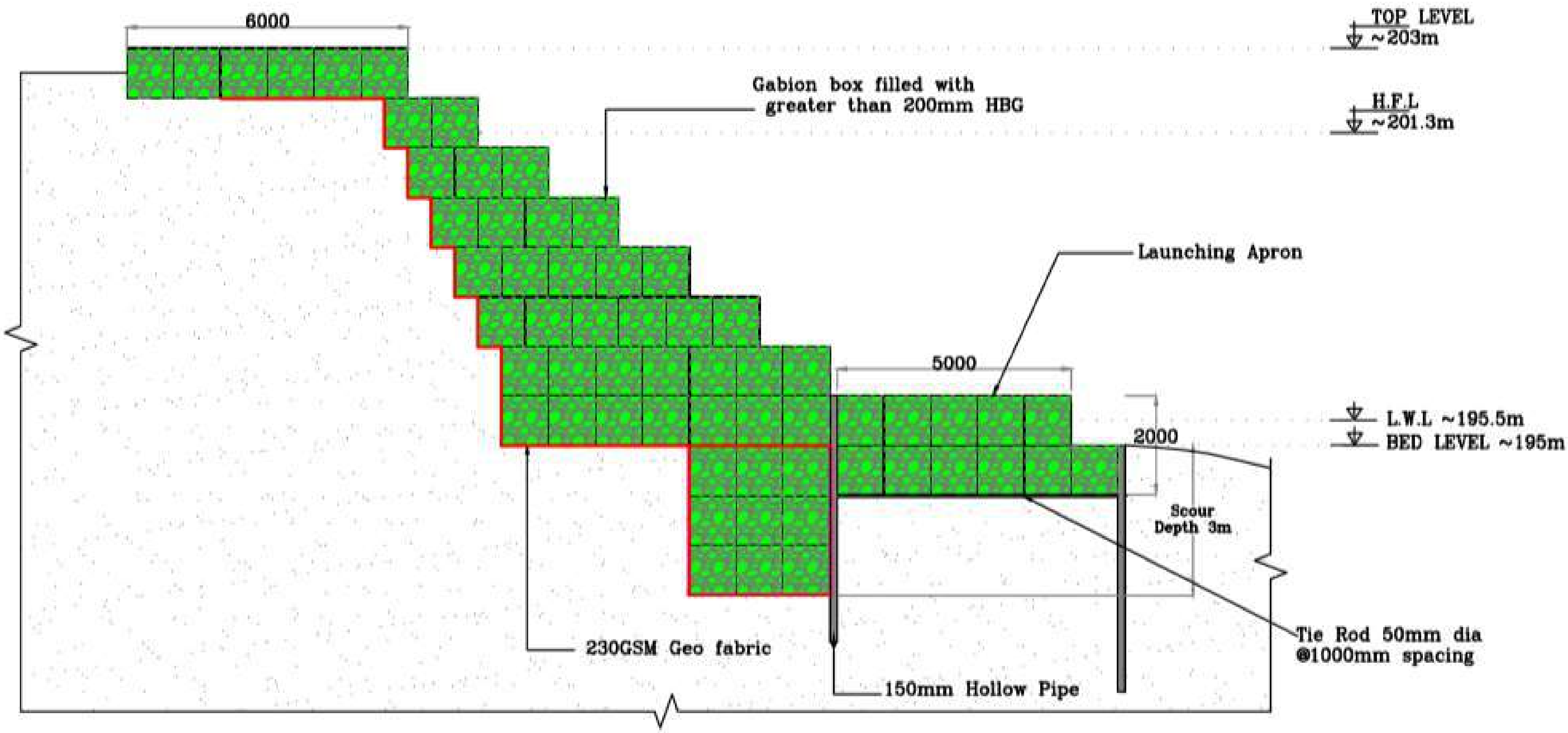
General Arrangement

- **Recorded maximum HFL = +200.3m**
- **HFL considered for fixing top level of embankment = +201.3m**
- **Free Board = 1.5m**
- **Top level of embankment = +203.00m**

There is varying bed level along the 700m stretch, the lowest bed level being approximately +195.85m. Hence to attain a uniform bed level, the lowest bed level i.e. say +195.00m is considered as the design bed level for the entire length of the guide bund to prevent any differential settlement.

- **Design Bed Level = +195.00m**
- **Considering water depth = 0.5m, LWL = +195.50m**

Typical Cross-section



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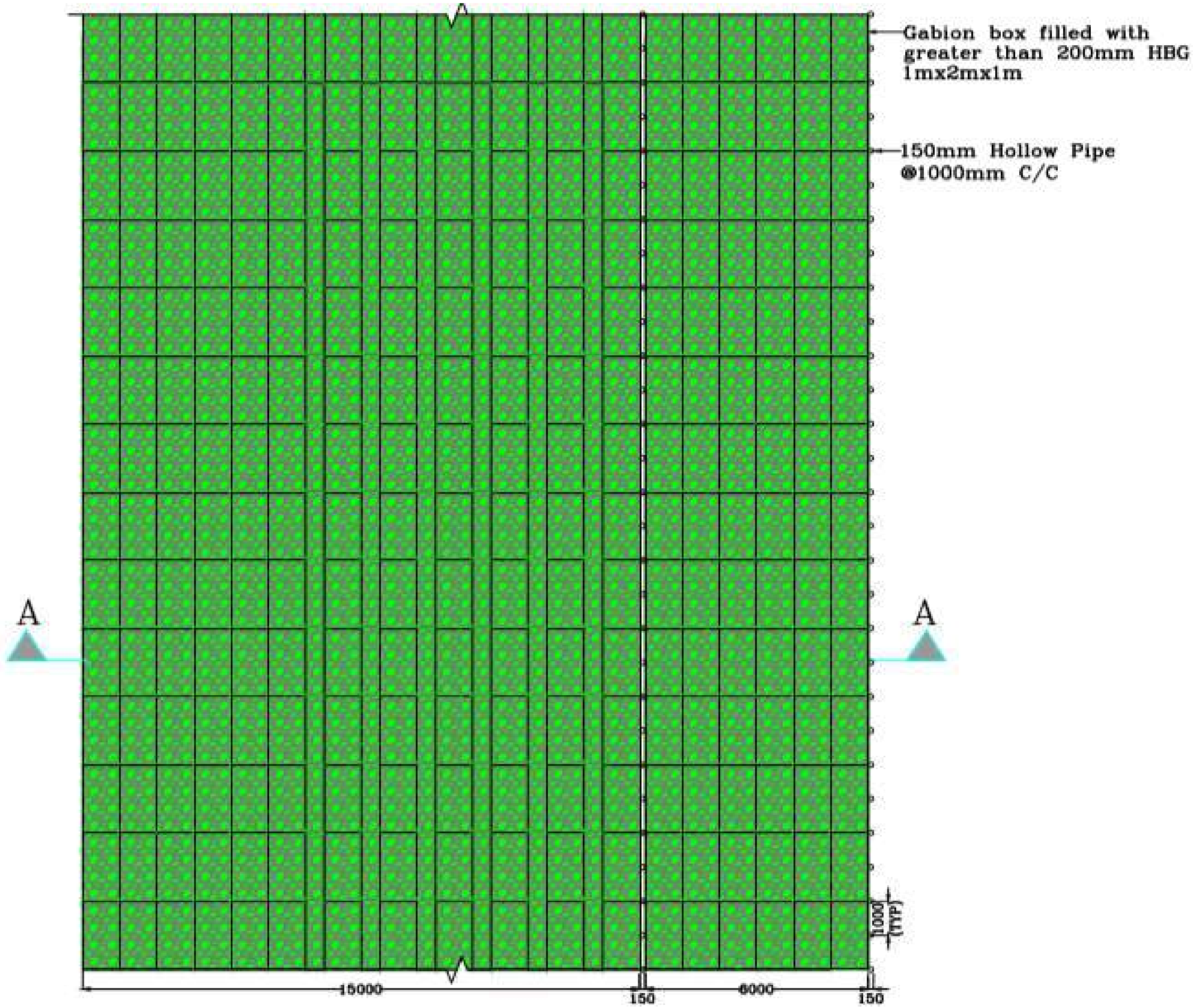
Plan View (Typical)



General Arrangement

- To protect the toe of the embankment from scouring, a launching apron will be provided on the riverside.
- 150mm diameter hollow circular steel pipes will be provided for anchorage, the depth of anchorage and spacing between the pipes will depend on the site conditions.
- For ease of driving, the steel pipes shall be welded together using a tie rod of 50mm diameter and the spacing of the tie rods will be same as that of the pipes.
- A layer of non-woven geotextile filter shall be placed, to prevent leaching out of foundation soils. The filter is easy to lay, durable, efficient and quality control is easy.

DESIGN OF GUIDE BUND

Revetment Thickness

As per, **clause 5.3.5.2 of IRC : 89 – 1997**, thickness of gabion revetment is

$$t = \frac{v^2}{2g(S_m - 1)}$$

where,

t = Thickness of revetment (m)

v = Maximum velocity = **4m/s**

g = Ground acceleration

S_m = Specific gravity of mattress = S_s(1-e)

S_s = Specific gravity of stone = 2.65

where,

d_{50} = Mean diameter of the stone filled in the gabions = 200mm

$$e = \frac{0.245 + 0.0684}{(d_{50})^{0.21}}$$

$$e = \frac{0.245 + 0.0684}{200^{0.21}}$$

Hence, e = 0.1

S_m = 2.65 (1-0.1) = 2.39

Revetment Thickness

Substituting values,

$$t = \frac{4^2}{2*9.81(2.39-1)} = 0.6\text{m}$$

According to 6.1.1.1, of FHWA HEC 11, the minimum thickness of mattress is 0.6m. As per 4.3 of HEC11, thickness should be **increased by 1.5 times** as the mattress is placed underwater to provide for uncertainties associated with placement.

t = FOS X t = 1.5 × 0.6 = 0.9m

Hence revetment thickness required = 0.9m.

Size of Gabion box = size 2m x 1m x 1m.

Scour Depth Calculation for HFL

Lowest Bed Level = 195.85m
HFL = 200.3m

Hence, depth of water = 4.5m
Considering maximum discharge as 29700 cumecs (1 in 100 years)
and width of river = 3500m,

The scour depth is estimated using **Lacey's formula, Clause 19.1 of IS 6966 - Part I: 1989**

R = 0.473 $(Q/f)^{1/3}$ when looseness factor more than 1.
R = 1.35 $(q^2/f)^{1/3}$ when looseness factor less than 1.
where,
R = Depth of scour below HFL in m
Q = High Flood Discharge in m^3/s
q = Intensity of flood discharge in m^3/s per metre width
f = silt factor $=1.76\sqrt{m_r}$
m_r = weighted mean diameter of the bed material = 2mm

Scour Depth Calculation for HFL

$q = 29700/3500 = 8.5\ m^3/s$ per metre width

$m_r = 2mm$ (**for heavy sand as per IRC 78:2000**)

$f = 1.76\sqrt{2} = 2.5$

$R = 1.35\ (8.5^2/2.5)^{1/3}$

$R = 4.14m$

Likely extent of scour as per **clause 19.2 of IS 6966 (Part 1) : 1989**, **Table 1**

Range = 1.25R to 1.75R

Mean = 1.5R = 6.2m

Mean depth of scour, below bed level, considering water depth 4.5m = 1.7m.

Maximum anticipated scour depth = FOS X Mean scour depth = 1.5 x 1.7 = 2.55m.

For analysis, the scour depth shall be considered as 3m.

Scour Depth Calculation for HFL

Table 1 Likely Extent of Scour at Different Places Along a Barrage/Weir

(*Clause* 19.2)

Sl No.	Location	Range	Mean
(1)	(2)	(3)	(4)
i)	Upstream of impervious floor	1·25 to 1·75 *R*	1·5 *R*
ii)	Downstream of impervious floor	1·75 to 2·25 *R*	2·0 *R*
iii)	Noses of guide banks	2·00 to 2·50 *R*	2·25 *R*
iv)	Noses of divide wall	2·00 to 2·50 *R*	2·25 *R*
v)	Transition from nose to straight	1·25 to 1·75 *R*	1·50 *R*
vi)	Straight reaches of guide banks	1·00 to 1·50 *R*	1·25 *R*

Scour Depth Calculation for LWL

Bed Level = 195m

LWL = 195.5m

Depth of water = LWL – Bed level = 195.5 – 195 = 0.5m

The scour depth is estimated using **Lacey's formula,**

R = 0.473 $(Q/f)^{1/3}$ when looseness factor more than 1.
R = 1.35 $(q^2/f)^{1/3}$ when looseness factor less than 1.
where,
R = Depth of scour below HFL in m
Q = High Flood Discharge in m^3/s
q = Intensity of flood discharge in m^3/s per metre width
f = silt factor = $1.76\sqrt{m_r}$
m_r = weighted mean diameter of the bed material = 2mm

Typical Scour Depth Calculation for LWL

q = 4 X 0.5 = 2m^3/s per metre width

m_r = 2mm (**for heavy sand as per table 1 of IRC 78:2000**)

f = $1.76\sqrt{2}$ = 2.5

R = 1.35 $(2^2/2.5)^{1/3}$

R = 1.57m

Likely extent of scour as per **clause 19.2 of IS 6966 (Part 1) : 1989**, **Table 1**

Range = 1.25R to 1.75R

Mean = 1.5R = 2.4m

Mean depth of scour, below bed level, considering water depth 0.5m = 1.9m ~ 2m.

Maximum anticipated scour depth = FOS X Mean scour depth = 1.5 x 2 = 3m.

For analysis, the scour depth shall be considered as 3m.

Launching Apron for Protection of Toe on Riverside

As per clause 5.3.7.5 of IRC 89: 1997,

Width of Lauching Apron = 1.5 times maximum anticipated scour depth
= 1.5 x 3 = 4.5m ~ 5m.

Max. Thickness of Launching Apron required = 2.25t = 2.25 x 0.6 ~ 2m.

Launching apron of width 5.0m at top and 6.0m at bottom and thickness 2.00 m is proposed at the toe.

Slope Stability Analysis

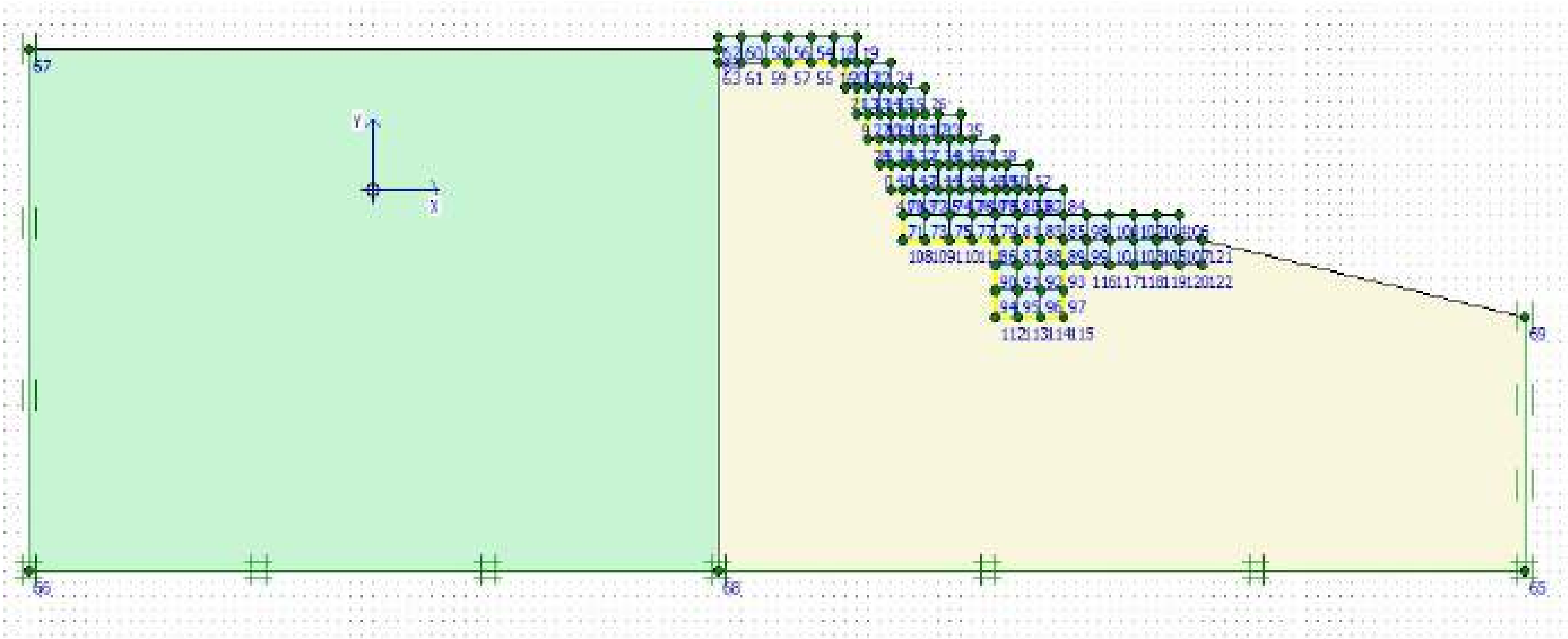
- The slope stability analysis is carried out using PLAXIS 2D software. The analysis is carried out considering different cases namely

1. High Flood Level with scour
2. Low Water Level with scour

- The stability of the river embankment will be checked for the above cases.

- As per **clause 4.7 of IS 12094: 2000**, the factor of safety (for the embankment to be stable under all stages of construction and conditions of saturation and drawdown) should be **1.3 or greater.**

PLAXIS 2D Analysis Model



Properties – Gabion box

Material Name	= Gabion Box
Material Model	= Mohr- coulomb
Material Type	= Drained
Saturated unit weight γ_{sat}	= 22 kN/m^3
Permeability at horizontal direction k_x	= 864 m/day
Permeability at vertical direction k_y	= 864 m/day
Young's Modulus E	= 150 $\times 10^3$ kN/m^2
Undrained cohesion C	= 200 kN/m^2
Poisson's Ratio	= 0.25
Angle of internal friction φ	= 38°

Properties – Sand

Material Model	= Mohr- coulomb
Material Type	= Drained
Unsaturated unit weight γ_{unsat}	= 16 kN/m^3
Permeability at horizontal direction k_x	= 1 m/day
Permeability at vertical direction k_y	= 1 m/day
Young's Modulus E	= 30×10^3 kN/m^2
Undrained cohesion C	= 0 kN/m^2
Poisson's Ratio	= 0.30
Angle of internal friction φ	= 32°

Properties – 230 GSM Geofabric

Material Type	= Elastic
EA	= 68 kN/m

PLAXIS 2D Analysis for HFL

Calculation information

Multipliers | Additional Info | Step Info

Step Info

Step	113 of 113	Extrapolation factor	2.000
Plastic STEP		Relative stiffness	0.000

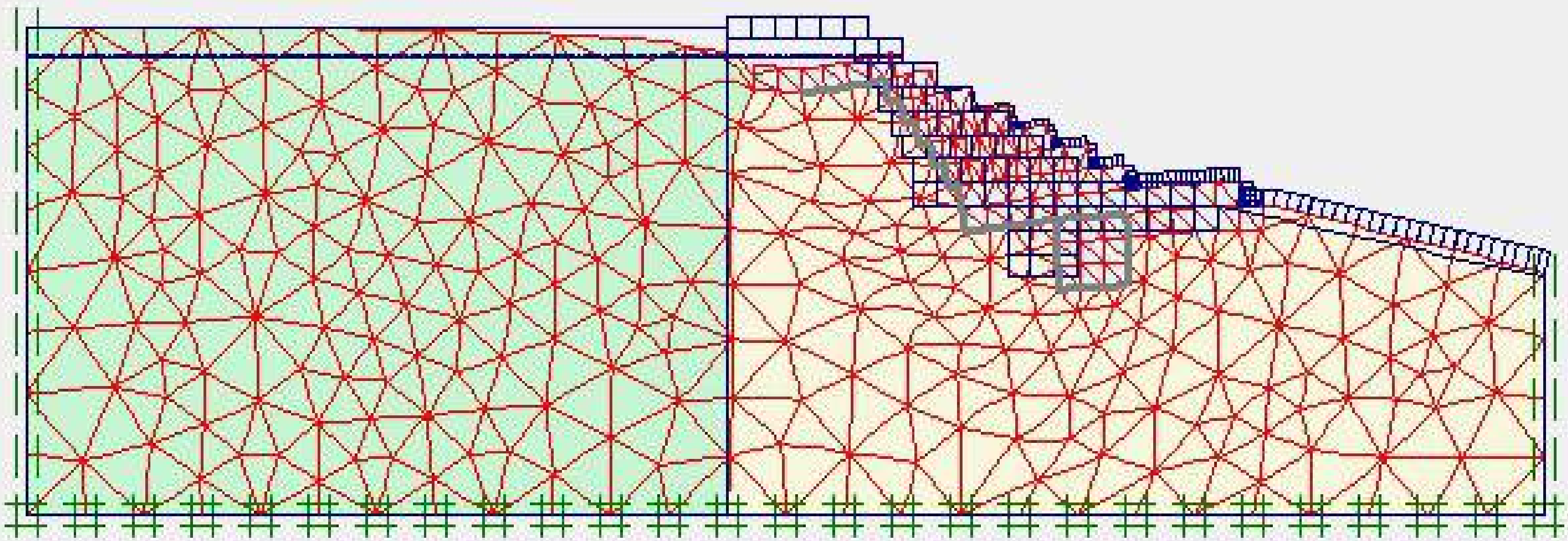
Multipliers

	Incremental multipliers		Total multipliers	
Prescribed displacements	Mdisp:	0.000	Σ-Mdisp:	1.000
Load system A	MloadA:	0.000	Σ-MloadA:	1.000
Load system B	MloadB:	0.000	Σ-MloadB:	1.000
Soil weight	Mweight:	0.000	Σ-Mweight:	1.000
Acceleration	Maccel:	0.000	Σ-Maccel:	0.000
Strength reduction factor	Msf:	0.000	Σ-Msf:	1.893
Time	Increment:	0.000	End time:	0.000
Dynamic time	Increment:	0.000	End time:	0.000

Print | OK

FOS = 1.893 > 1.3

PLAXIS 2D Analysis for HFL



Deformed mesh

Extreme total displacement $12.40*10^{-3}$ m
(displacements scaled up 200.00 times)

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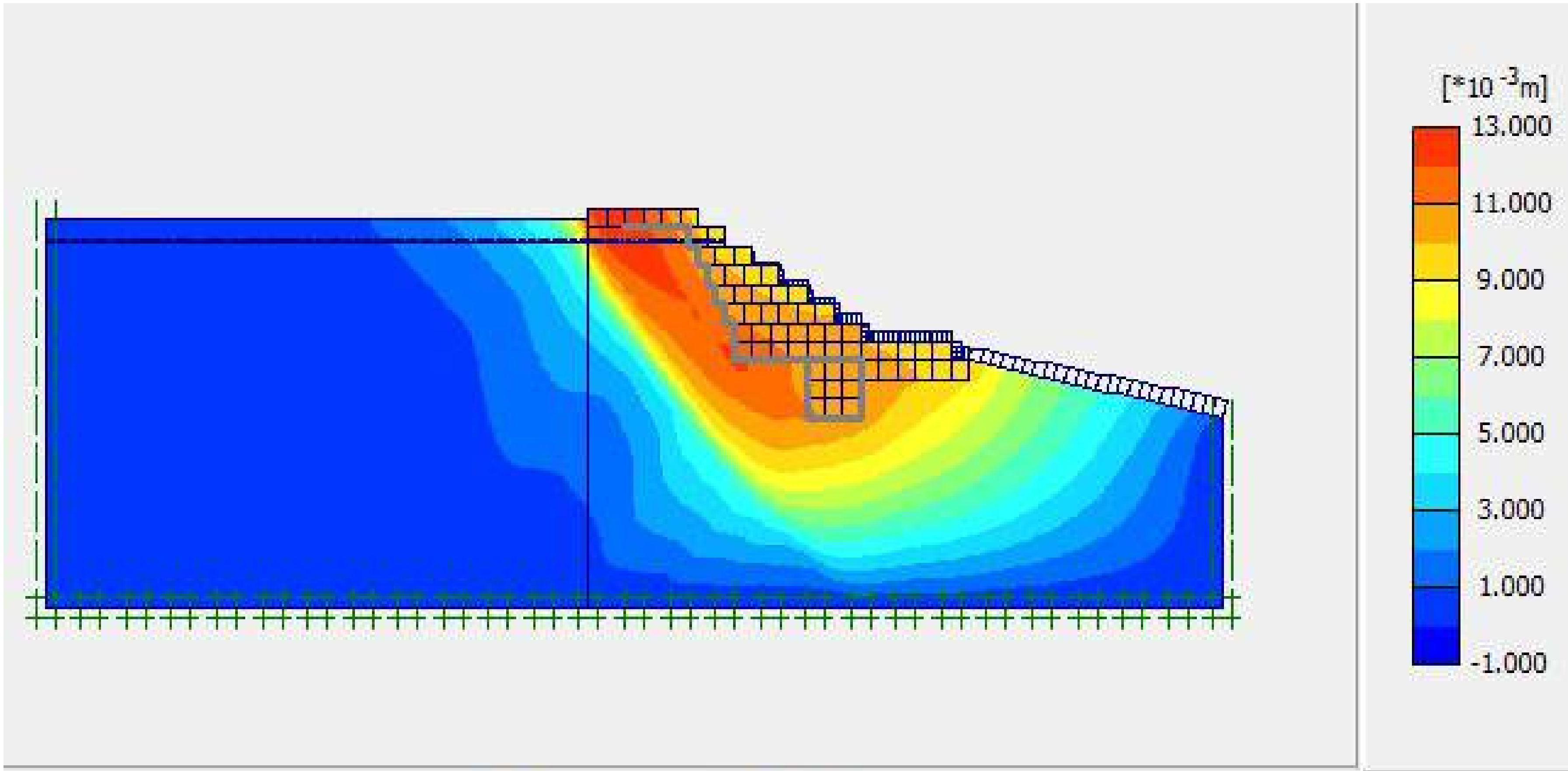
PLAXIS 2D Analysis for HFL



Total displacements (Utot)

Extreme Utot $12.40 * 10^{-3}$ m

PLAXIS 2D Analysis for LWL

Calculation information

Multipliers | Additional Info | Step Info

Step Info

Step	118 of 118	Extrapolation factor	1.000
Plastic STEP		Relative stiffness	0.000

Multipliers

	Incremental multipliers		Total multipliers	
Prescribed displacements	Mdisp:	0.000	Σ-Mdisp:	1.000
Load system A	MloadA:	0.000	Σ-MloadA:	1.000
Load system B	MloadB:	0.000	Σ-MloadB:	1.000
Soil weight	Mweight:	0.000	Σ-Mweight:	1.000
Acceleration	Maccel:	0.000	Σ-Maccel:	0.000
Strength reduction factor	Msf:	0.000	Σ-Msf:	1.600
Time	Increment:	0.000	End time:	0.000
Dynamic time	Increment:	0.000	End time:	0.000

Print | OK

FOS = 1.6 > 1.3

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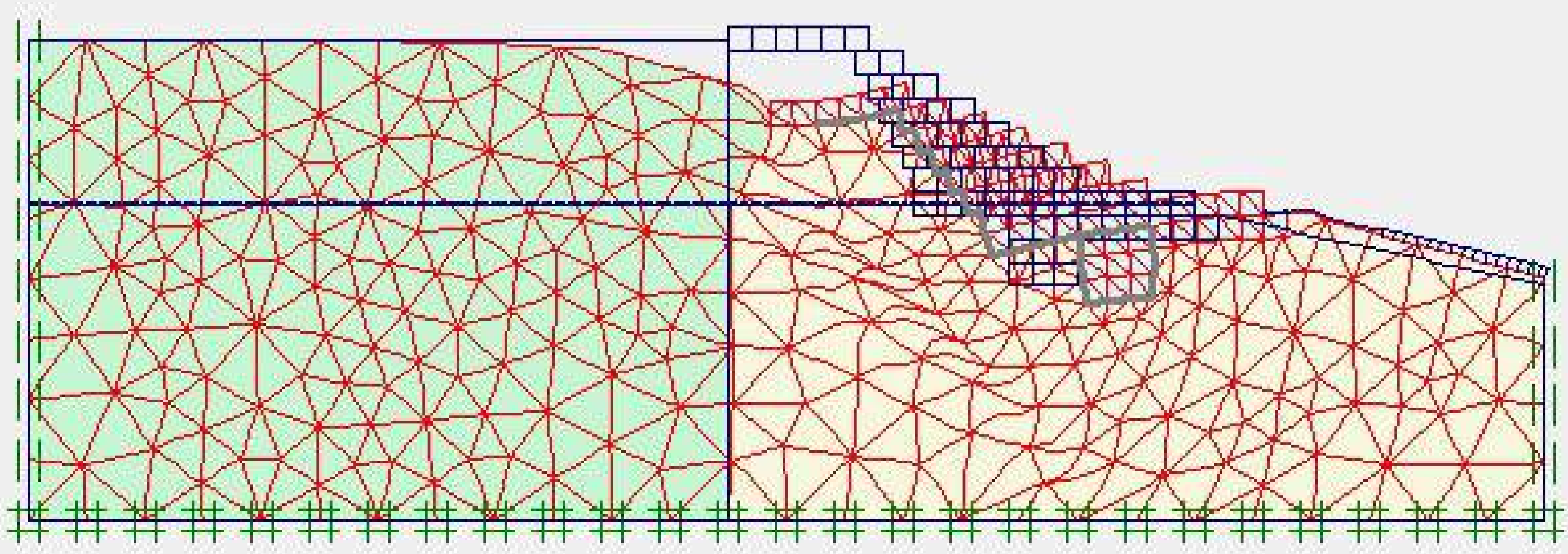
PLAXIS 2D Analysis for LWL

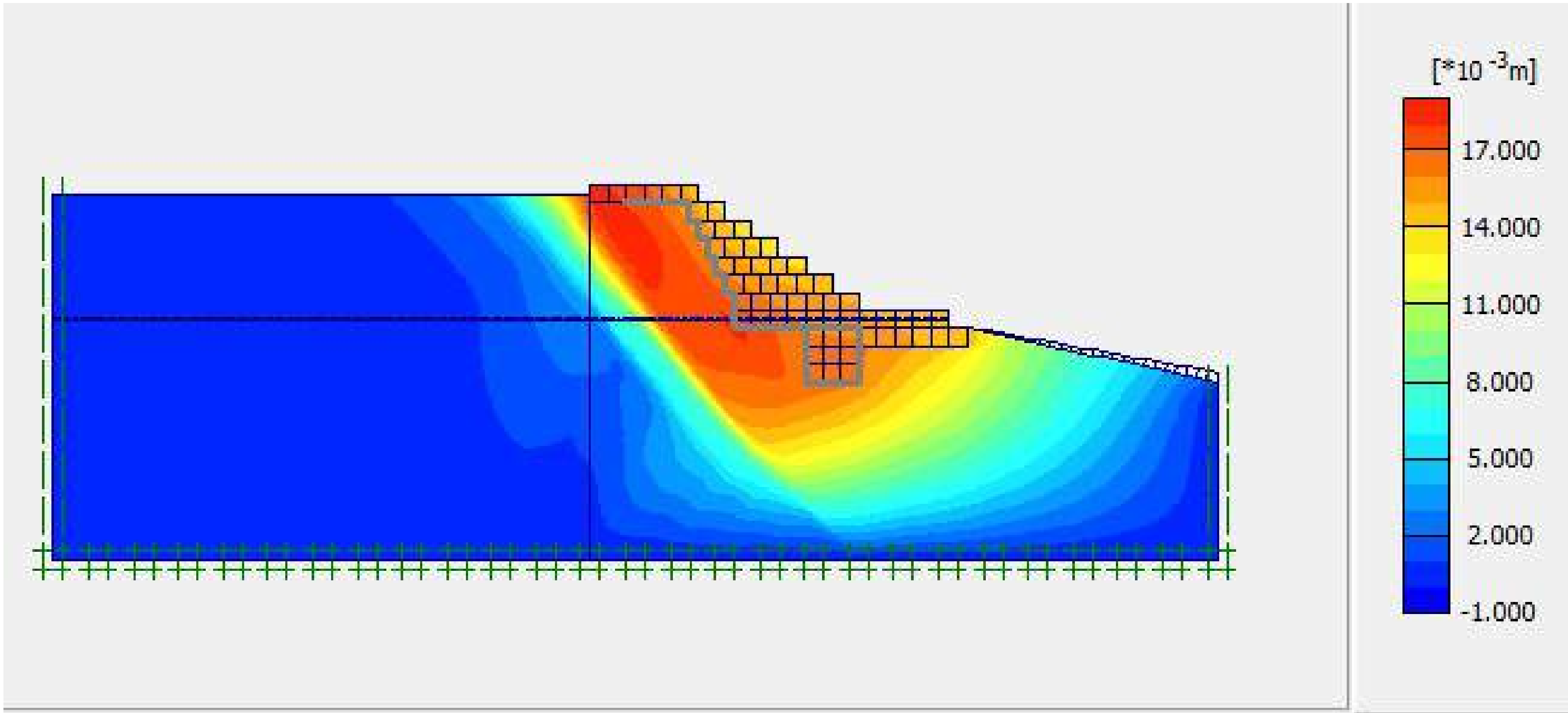


Deformed mesh

Extreme total displacement 18.42*10 $^{-3}$ m

(displacements scaled up 200.00 times)

PLAXIS 2D Analysis for LWL



Total displacements (Utot)

Extreme Utot 18.42*10 $^{-3}$ m

Detailed Cost Estimate

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Sl. No:	Description	Unit of Measure ment	Quantity	Rate per unit (Rs.)	Total Amount (Rs.)	Total Amount (Crores.)	Total Amount (Per RM)
1	Earth work in excavation of foundation/ trench/ preparatory area in all kinds of soil (this shall include ordinary soil, sand, silt, slusy soil, gravelly soil, stoney earth mixed with pebbles isolated stone boulders not exceeding 0.028 cum by volume, stiff heavy clay, hard shade, compact muroom and isolated rocks not requiring blasting) both in dry, wet and submerged condition by manually or mechanically means including dewatering, jungle clearance, stump removals, rough dressing and leveling as per design section and depositing the excavated soil in layers not exceeding 22.5 cm in depth, breaking clods maximum 5 cm to 7 cm size and depositing the excavated material away from outside as specified with spreading and leveling to required profile by mechanical or manual means with all leads, lifts and delifts, loading, unloading, handling, rehanding with all cost of labour and transportation etc, with all taxes, royalities, T&P etc complete with all unforeseen items & incidental cost to work site as per specification & direction of Engineer-in-charge.(Measurement of the item will be in level section)	Cum	74500	500	37250000	3.725	53214.286
2	Collecting, supplying & spreading good quality coarse river sand & compacting with watering by vibrator roller for filter layer over compacted surface including hire & running charges of roller, all machineries, all cost of sand conveyance, dewatering, dewatering charges, taxes & royalties etc. complete as directed by Engineer-in-Charge.	Cum	9500	1500	14250000	1.425	20357.143
3	Collecting, supplying and placing ISO Certififed Rope Gabion Box (9mm, 4 strand PP tarred rope) of size 2000 mm x 1000 mm x 1000 mm with mesh size 150 mm x 150 mm filled with hard granite stone not less than 200 mm size placing along the proposed alingment of embankment in the desired location including cost of all gabion Boxes, cost, carriage of material for tying & labour for filling, placing including hire & running charges of machinery required etc. complete as directed by Engineer-in-charge.	Nos.	21000	5200	109200000	10.920	156000.0
4	Non - woven geotextile filer - 230 GSM Geofabric, as per material specification 592.	Sq.m	17150	110	1886500	0.189	2695.0
5	Supplying and laying of 150 mm diameter of hollow circular pipe with a length of 5m long placed @ 1m horizontal spacing for anchorage. Cost of material with transportation, royalties hire & running charges of all machineries deployed, all cost of labour & all cost of dewatering charges till completion of the work etc. complete as directed by Engineer-in-Charge.	MT	105	69000	7245000	0.725	10350.0
6	Tie rod 50mm dia with a length of 6m placed @ 1m horizontal spacing, for welding the 2 hollow pipes together.	MT	66.5	69000	4588500	0.459	6555.0
				Sub Total	**174420000**	**17.44**	**249171.43**

Webinar on Geo-synthetics for Coastal Protection and Port works
10th and 11th Dec 2020

Detailed Cost Estimate

	Total Amount (Rs.)	Total Amount (Crores)	Total Amount (Per RM)
Sub Total	174420000	17.44	249171.43
Contingency 5%	8721000	0.87	12458.57
GST 12%	20930400	2.09	29900.57
Grand total	204071400	20.41	291530.57

IITM Recommendation for Repelling Spur

A

Satellite Data Analysis (Source - Invent Grid India (P) Ltd) - Flow of Posonallah River – 2019 & 2020.

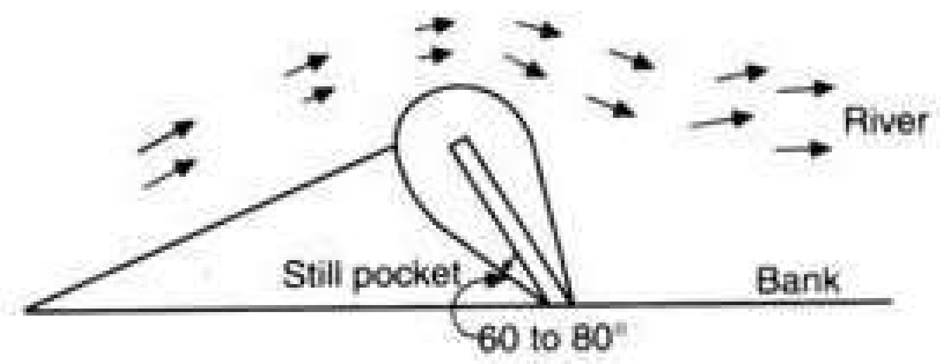
IITM Recommendation for Repelling Spur

Currently, the Client have proposed construction of Guide Bund for 700m (550m upstream and 150m downstream). But beyond this, for 900m it can be seen the river is even going behind the longitudinal alignment of A6 Guide Bund and hence needs to be arrested to prevent further meandering of river behind the line of A6 Guide bund.

Hence IITM strongly recommend to provide shore protection measures in the form of repelling spur for the remaining 900m.

Design of Repelling Spur

- Repelling spur pointing u/s is to be provided to repel the flow away from the bank.
- On the u/s of the repelling spur, a still pocket is formed, where the sediments carried by the river water get deposited.
- The head of the repelling spur cause disturbance in the flow at its nose.
- The head of the repelling spur need strong protection, as the head is subjected to direct attack of whirling current.
- The repelling spur will be designed as per **IS: 8408 – 1994 and IRC: 89 – 1997.**



Design of Repelling Spur

1. Length of spur

As per clause 5.2 of IS: 8408 – 1994, the effective length of the spur shall not exceed 1/5th of width of flow and not less than 2.5 times the maximum anticipated scour depth. The length of the proposed spur is 60m.

2. Orientation of spur

As per clause 6.2.2 of IRC: 89 – 1997, for repelling spur the angle upstream varies from 60° to 80° with the bank. The proposed spur shall be inclined at an angle 60 degrees from the bank and facing upstream side.

3. Spacing of spur

As per clause 5.2 of IS: 8408 – 1994, the spacing of spur is normally 2 to 2.5 times of effective length. The spacing of spur of effective length 60m shall be kept as 120m.

Design of Repelling Spur

4. Top level of Spur

Recorded maximum HFL = +200.3m

HFL considered for fixing top level of spur = +201.3m. Considering 1.5m as free board as per **clause 5.3 and 5.5 of IS: 8408 – 1994**, the top level of the spur shall be fixed as +203.00m MSL.

5. Width of spur

As per clause 5.4 of IS 8408 – 1994, the top width shall be taken as 3m at +203.00m MSL.

6. Size of Gabions

As per clause 6.3.4 of IRC: 89 – 1997, the size of stone shall be same as that for the guide bund. Hence Gabion boxes of size 2m x1m 1m shall be used.

Design of Repelling Spur

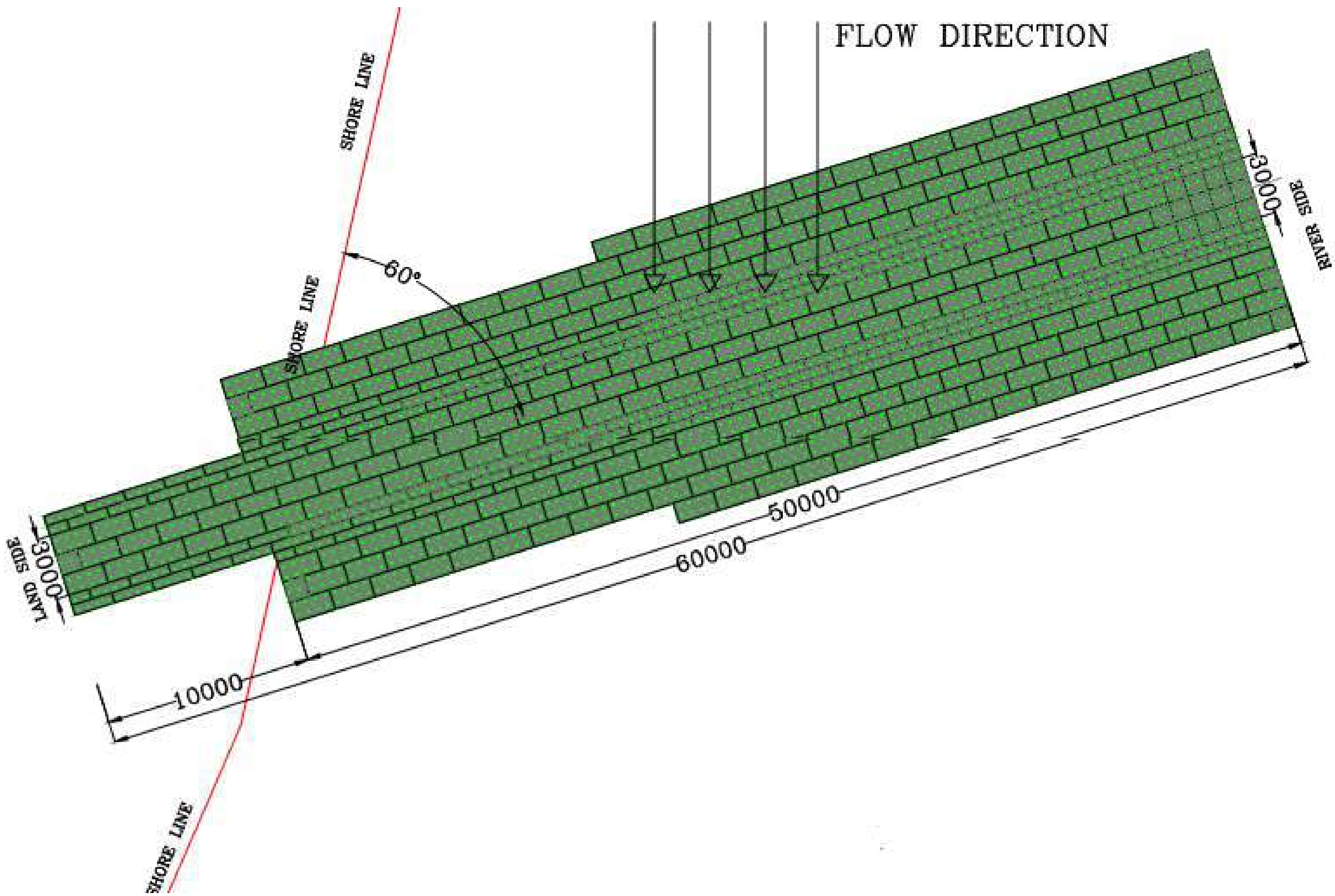
7. Launching Apron

As per clause 6.3.7.3 of IRC: 89 – 1997, a Launching apron of width 5.0m and depth 3.0m is proposed at the nose.

As per clause 5.9.5 as per IS 8408 - 1994, the width and depth of the launching apron will be reduced gradually, depending on the bed profile at each location.

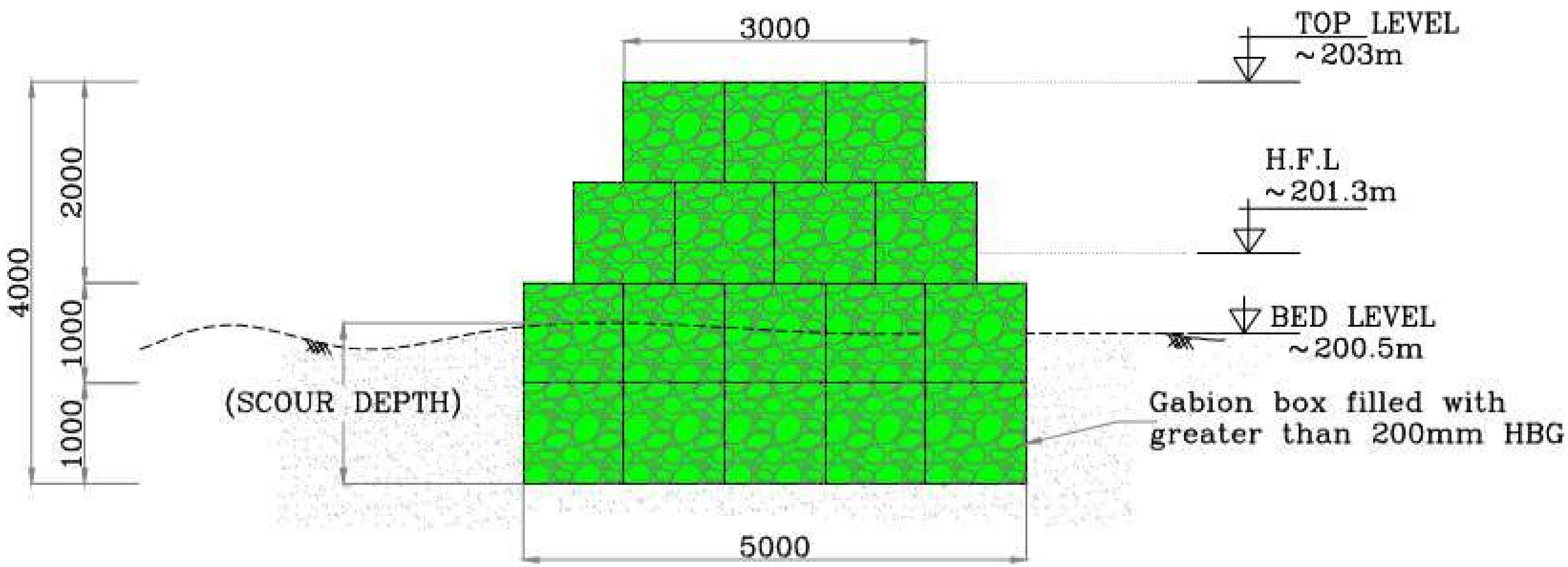
Prof. R.Sundaravadivelu, IITM

Plan View of Proposed Repelling Spur (Typical)



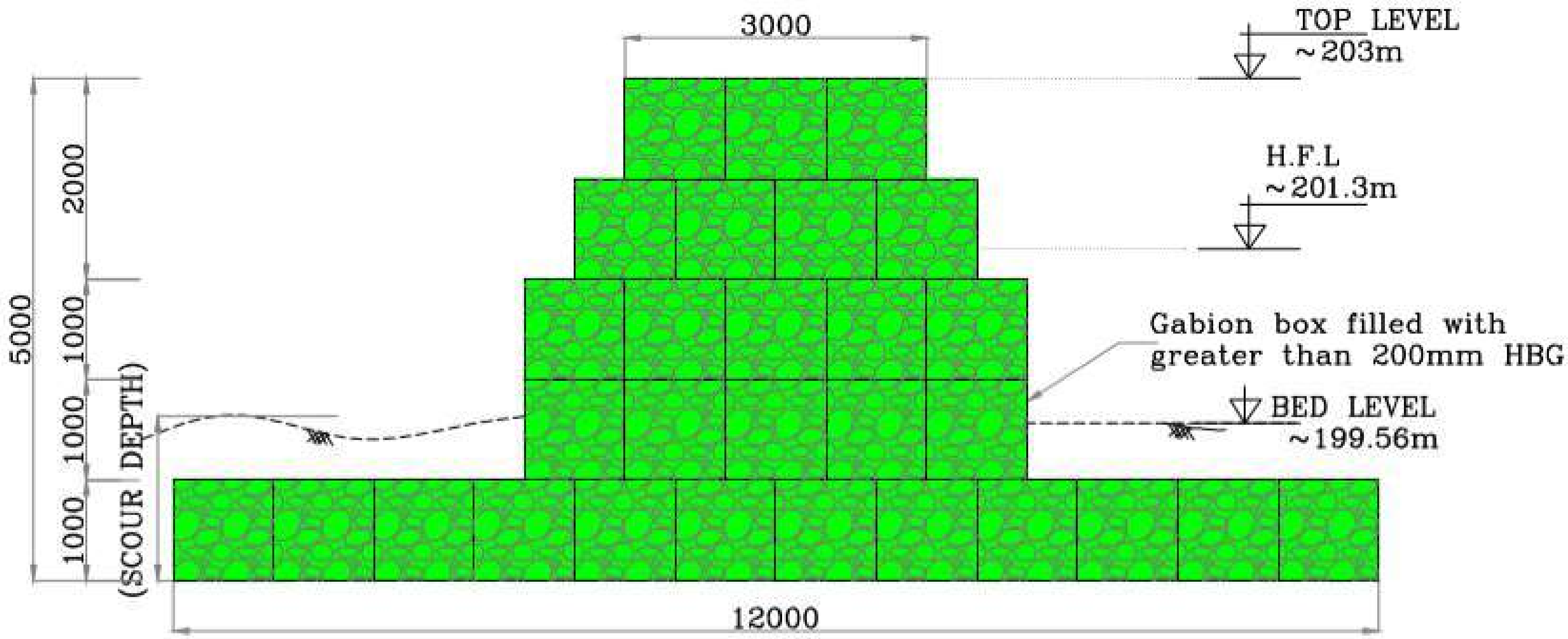
Prof. R.Sundaravadivelu, IITM

Cross-section of Proposed Repelling Spur (Typical)



CROSS SECTION AT 0 TO 10m for SPUR-1

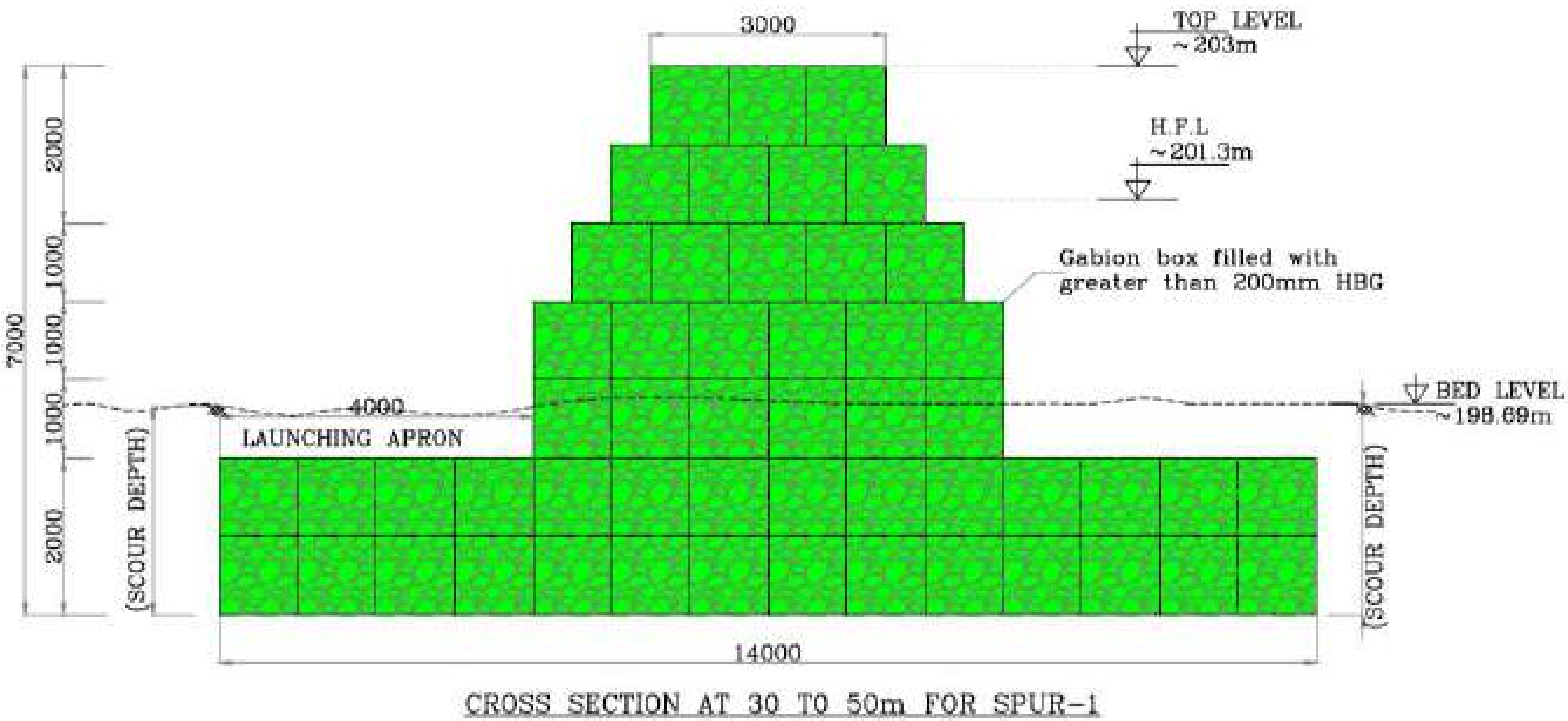
Cross-section of Proposed Repelling Spur (Typical)



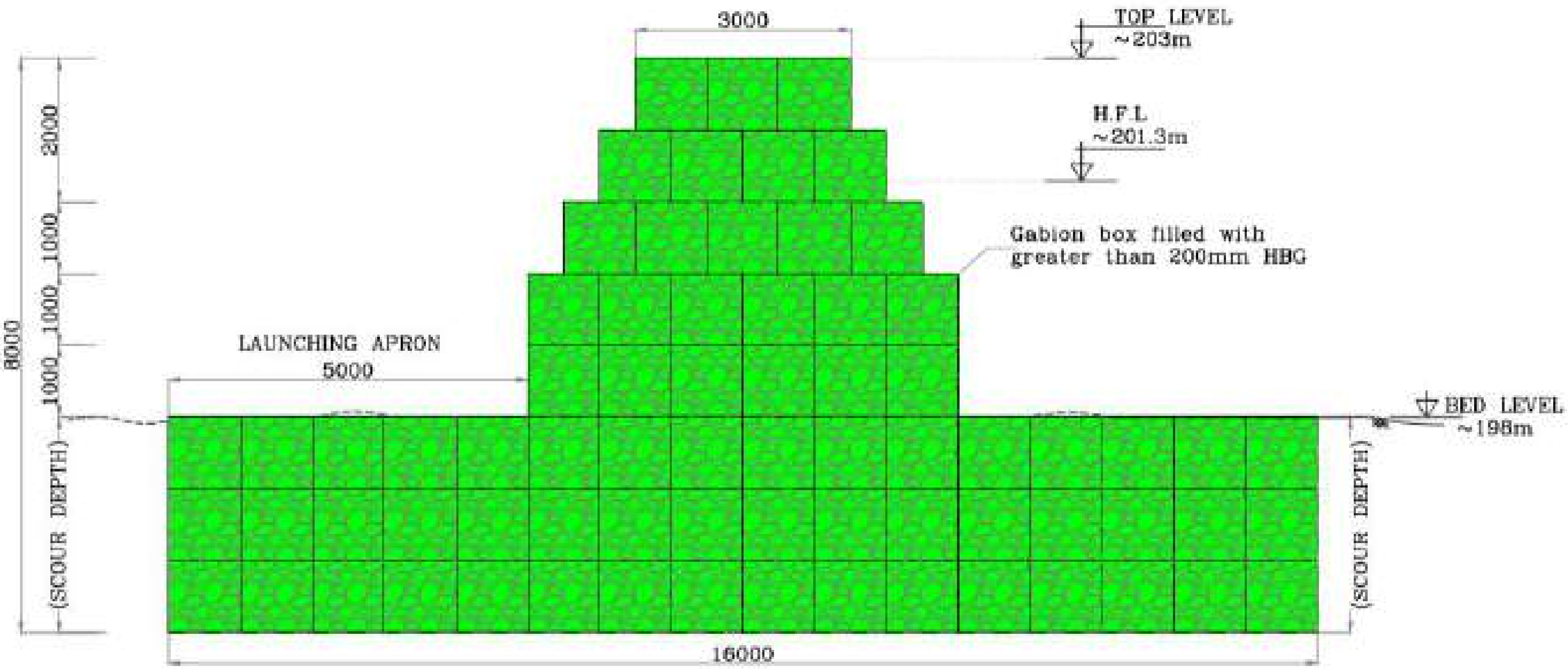
CROSS SECTION AT 10 TO 30m FOR SPUR-1

Prof. R.Sundaravadivelu, IITM

Cross-section of Proposed Repelling Spur (Typical)



Cross-section of Proposed Repelling Spur (Typical)



CROSS SECTION AT 50 TO 60m FOR SPUR-1

Design of Repelling Spur

- A total of 7 spurs is proposed in the 900m stretch upstream of the guide bund.
- The spurs will be designed for varying depth.
- Hence the longitudinal sections and cross-sections for each of the spurs will vary based on the location of the spur, bathymetry and site conditions.
- The spurs provided will effectively deflect the flow current to the central portion of the river.
- Since the river is very steep in the upstream portion for the proposed location, repelling spurs may be required for further upstream as per site conditions, in order to protect the banks from erosion.

Google Location of Proposed Repelling Spurs

Conclusion

- The slope stability analysis for design of Posonallah guide bund along the Dibang river at Arunachal Pradesh was carried out for the cross section proposed using gabion boxes.
- The finite element analysis shows a factor of safety greater than the minimum required 1.3. This clearly indicates that the proposed slope is stable.
- The Repelling spurs provided will effectively deflect the flow current to the central portion of the river.
- Since the river is very steep in the upstream portion for the proposed location, repelling spurs may be required for further upstream as per site conditions, in order to protect the banks from erosion.

THANK YOU

Webinar on
Draft Guidelines on Geo-synthetics for Coastal Protection and Port Works
10-11 Dec. 2020

Coastal erosion
and
Role of Geosynthetics

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Coastal Processes

- **Typical beach profile and coastal zone**

- Beaches dissipate wave energy and are constantly adjusting to the wave environment (shoaling, wave breaking, sand bar & surf zone)

- **Littoral Transport (sediment transport)**

- Long shore transport (parallel to the shoreline, long shore current)

- Offshore-onshore transport (perpendicular to the shoreline)

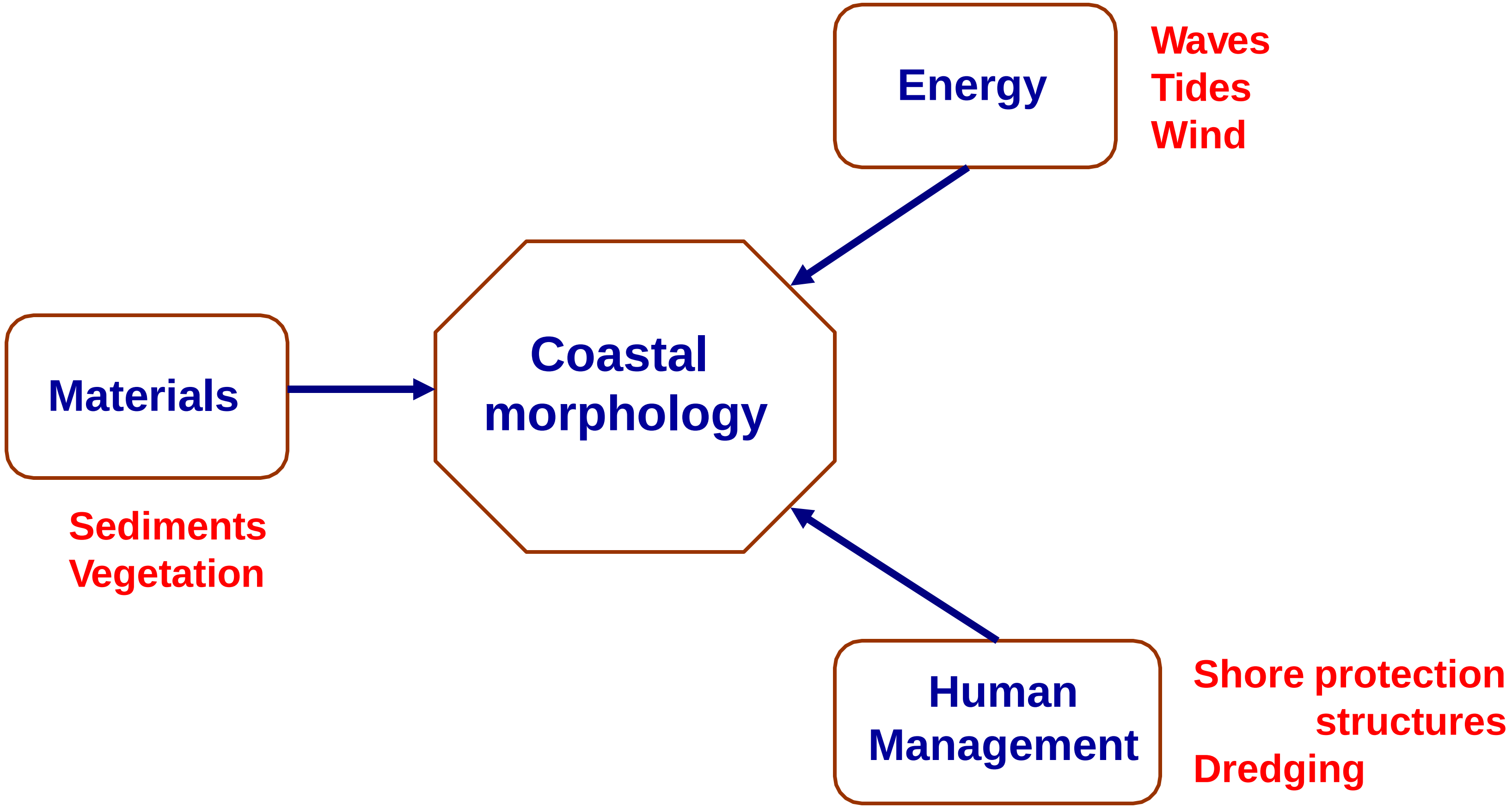
Indian Peninsula :

- About 6500 km long shoreline
- Settlement along Coastal line is large
- Several Industries
- Major Cities like Chennai, Mumbai
- Important historical Monuments

Adverse Environmental Forces

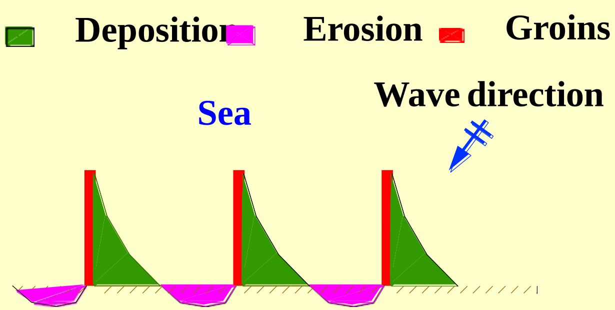
- Waves
- Storms
- Tidal level variation

Basic controls of a coastal system



REMEDIAL MEASURES FOR COASTAL EROSION

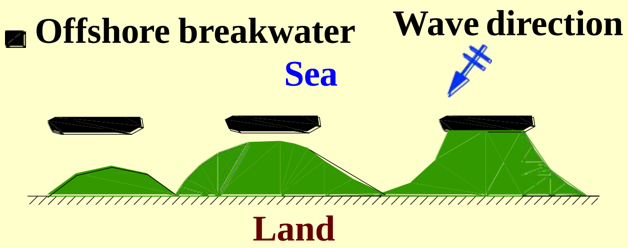
	NonStructural response	Natural !!
Normal to the coastline [groins]	Replenishment of coast with sand.	Bioshield
Parallel to the coastline on shore [seawalls]	At the landward side of the dune [A]	
Parallel to the coastal offshore break waters	At the seaward side of the dune, landward of the dune base [B]	
	At the seaward side of the dune, seaward of the dune base [C]	

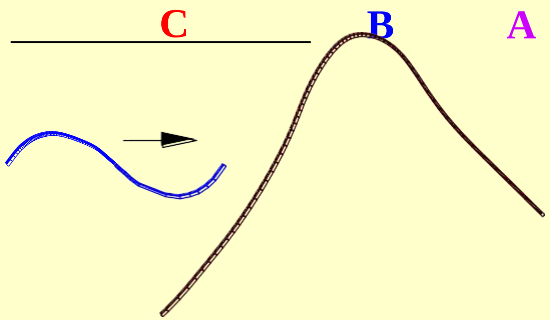


Sea

Seawall

Land





Type of "sections" as a part of Structural Response

Conventional structures using

- Rubble Stones
- Masonry/ Concrete blocks of different shape

In addition,

- Geosynthetic materials – gabions, geosynthetic bags

What is the preferred Choice?

Simple solution by a Field Engineer is, To

replicate an older (!!) design

Most times, it fails

Site specific studies

Every decade one can see environmental changes – Climate Change!

Thus, Increased Challenges being faced by the Engineer from

Environmental Forces (increases) Space

Constraints

Decision on Protection Scheme

Environmental Clearance

Further,

It takes more time for Obtaining "Clearance"

and the Engineer/ Contractor would be

asked to work in a „Short Time Schedule"

- Possible with max. use of local resources

Forces one to opt for different than Conventional materials like Rubble Stones & Concrete/ masonry

One of the Alternative: Geosynthetic Systems

!!Claim!!

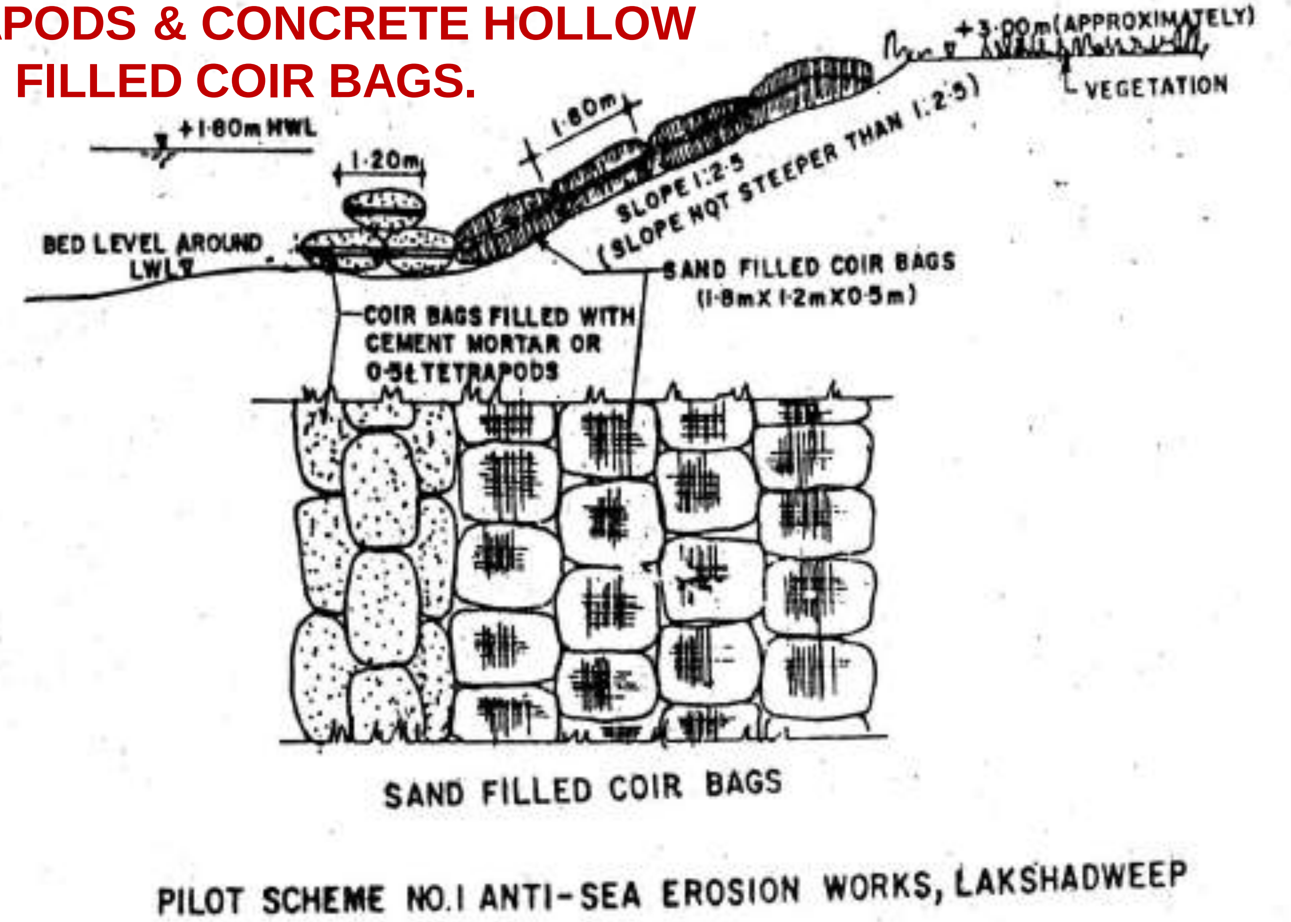
- **Least environment impact**
- **Long term performance & higher Cost benefit ratio**

- **In India, it is not proven yet in Coastal Environment!!**
- **But, we have the concept of BAG and TUBES from the historical times earlier than the geosynthetics.**

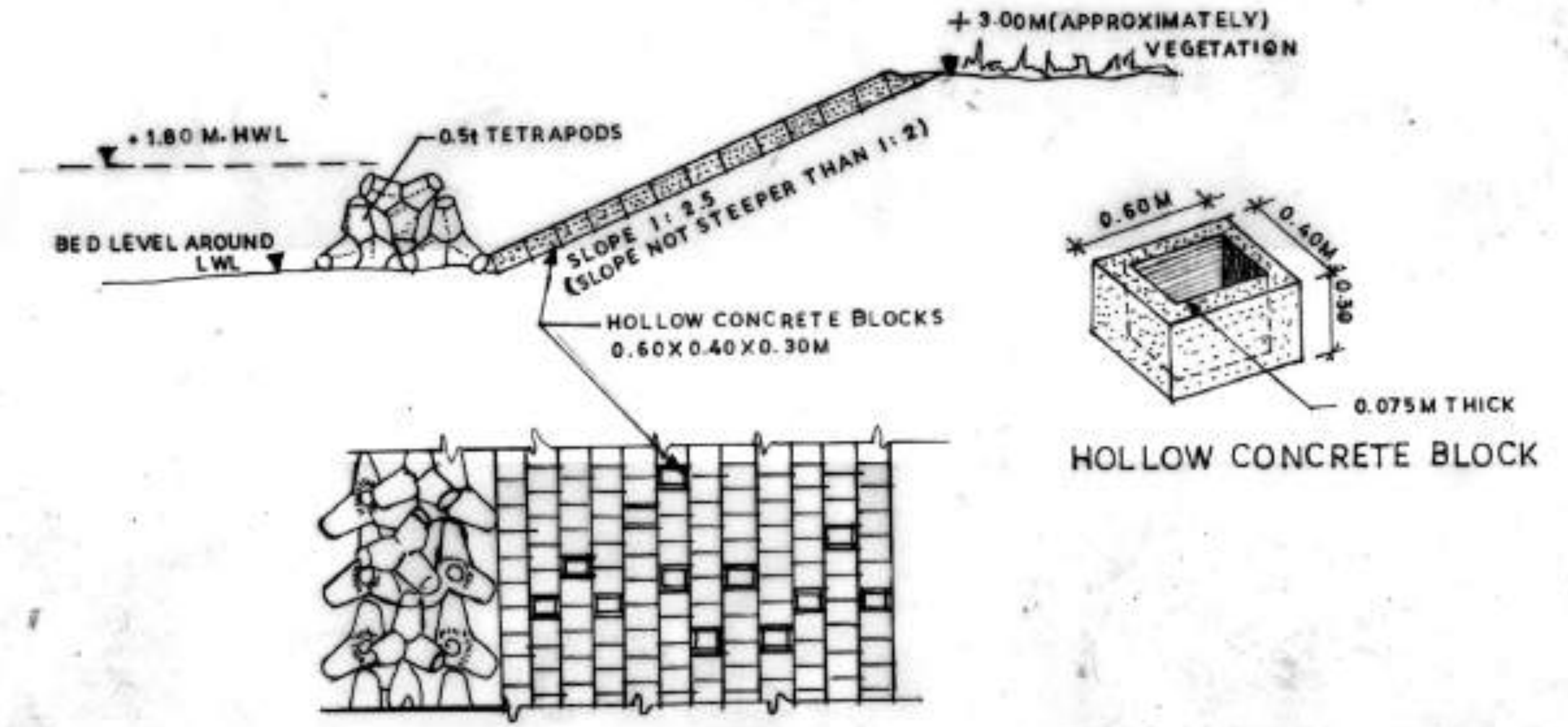
PROTECTION WORK AT PADUTHOMSE RASID, KARNATAKA

LAKSHADWEEP ISLANDS
132 km long shoreline
along 36 islands

SEAWALL USING TETRAPODS & CONCRETE HOLLOW BLOCKS, MORTAR FILLED COIR BAGS.



SOURCE : CWPRS, PUNE



PILOT SCHEME NO.2 ANTI-SEA EROSION WORKS, LAKSHADWEEP

SOURCE: CWPRS,PUNE

Geosynthetics for hydraulic structures since 1980s

Geosynthetic Systems

- Flexible — *ok*
- Porous — *ok (to drain but not dissipate energy)*
- Economical — ??
- Faster Installation — *ok*
- Allows use of Local Materials for Construction — *ok / ?where is source?*

Design Principles

Wave Energy Dissipation

- Gabions – rubble stone filled
  -- Geotextile bags filled
- Sand Filled Geotextile Tubes
- Sand Filled Geotextile Bags / Containers / Mattresses

Filtration

- Allows water to pass, Retains soil and Prevents erosion

TECHNICAL SPECIFICATIONS

Properties to be concentrated by a design engineer (?Standardised Physical & Mechanical properties?)

- **Hydraulic Properties**
  - **Porous**
- **Survivability**
  - **Installation Practices**
  - **UV Resistance**

Presently, irrespective of application, technical specification includes all the properties irrespective of the function to be satisfied

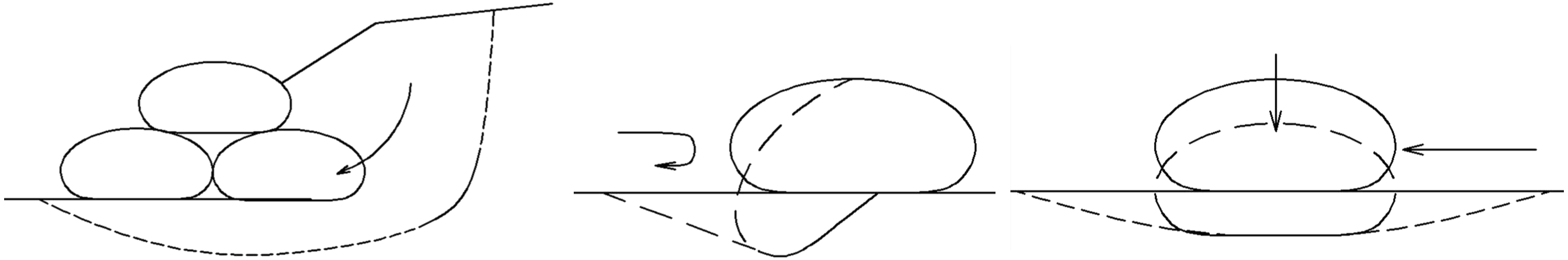
Physical Stability Criteria

1. External Stability

2. Internal Stability

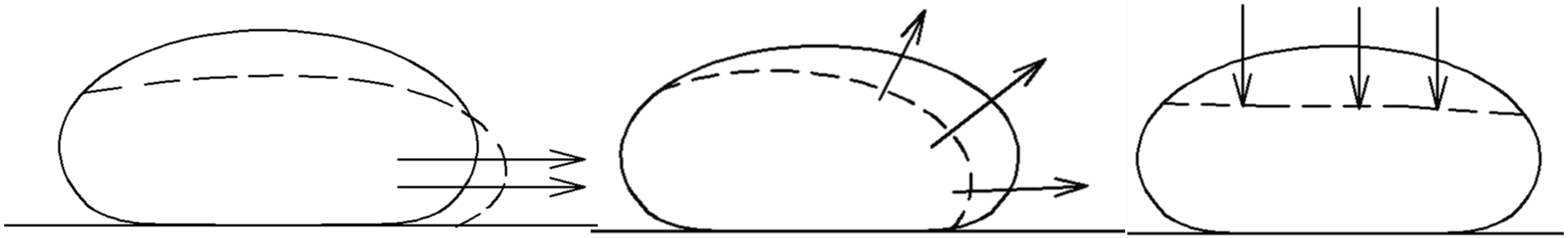
External Stability

Sliding **Overturning** **Bearing Failure**



Slope failure **Scour** **Foundation Settlement**

Internal Stability

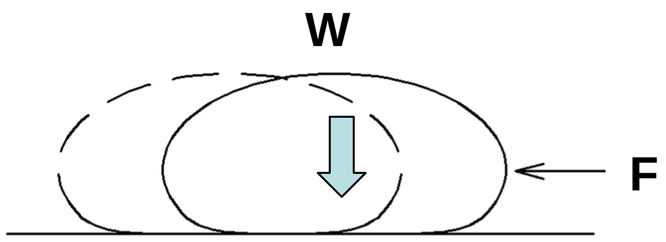


Geotextile Rupture

Erosion of Fill

Deformation of Fill

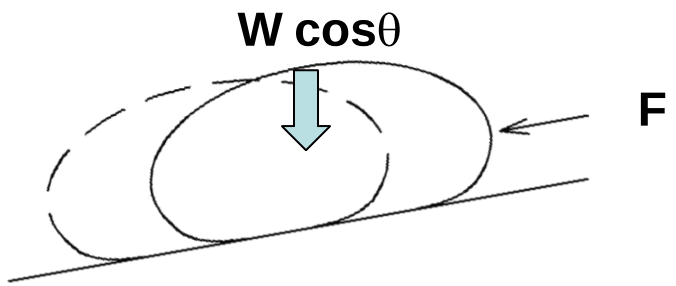
External Stability

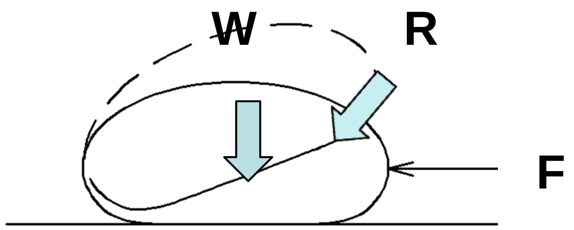


Sliding

$\mu W > F$

Factors: Smooth fabric

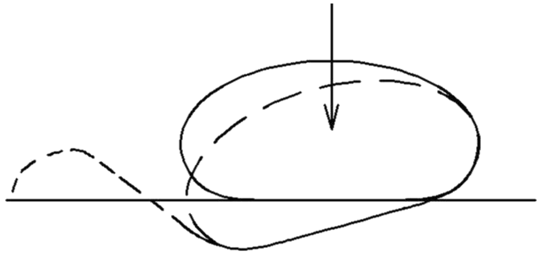




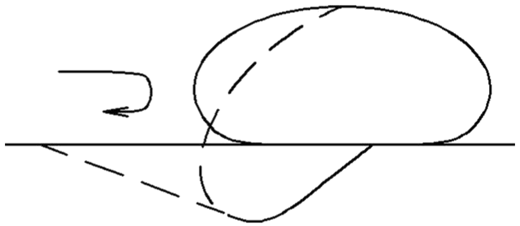
Overturning

$F\,h > W\,x$

External Stability



Bearing failure

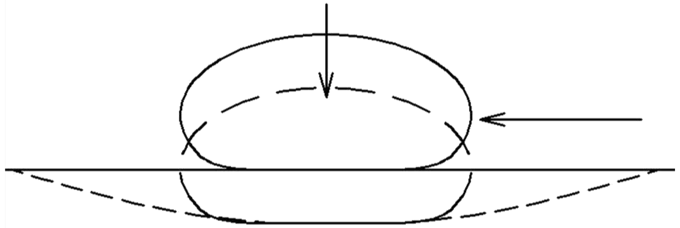


Scour

Foundation Characteristics (Seabed)

Characteristics due to wave action

Failure occurs while filling with Foreign soil with higher density



Foundation stability

Design of Sections in the Marine Environment

Governing Factors

Stability

Environmental Force (wave/ current)

Weight of tube/ bag

Surface Smoothness

Settlements (Bearing/ scour/Foundation)

Bearing capacity – underlying seabed Scour – sediment type, wave action & size/type of bag

Foundation settlement: While foreign soil is used

Section should be such that it reduces the

wave impact on the face of the structure as well as

to provide more stability against sliding,

Stability

Environmental Force – site specific

Weight – governed by the size of tube & smoothness

Reduce wave impact by providing a berm to induce loss of wave energy

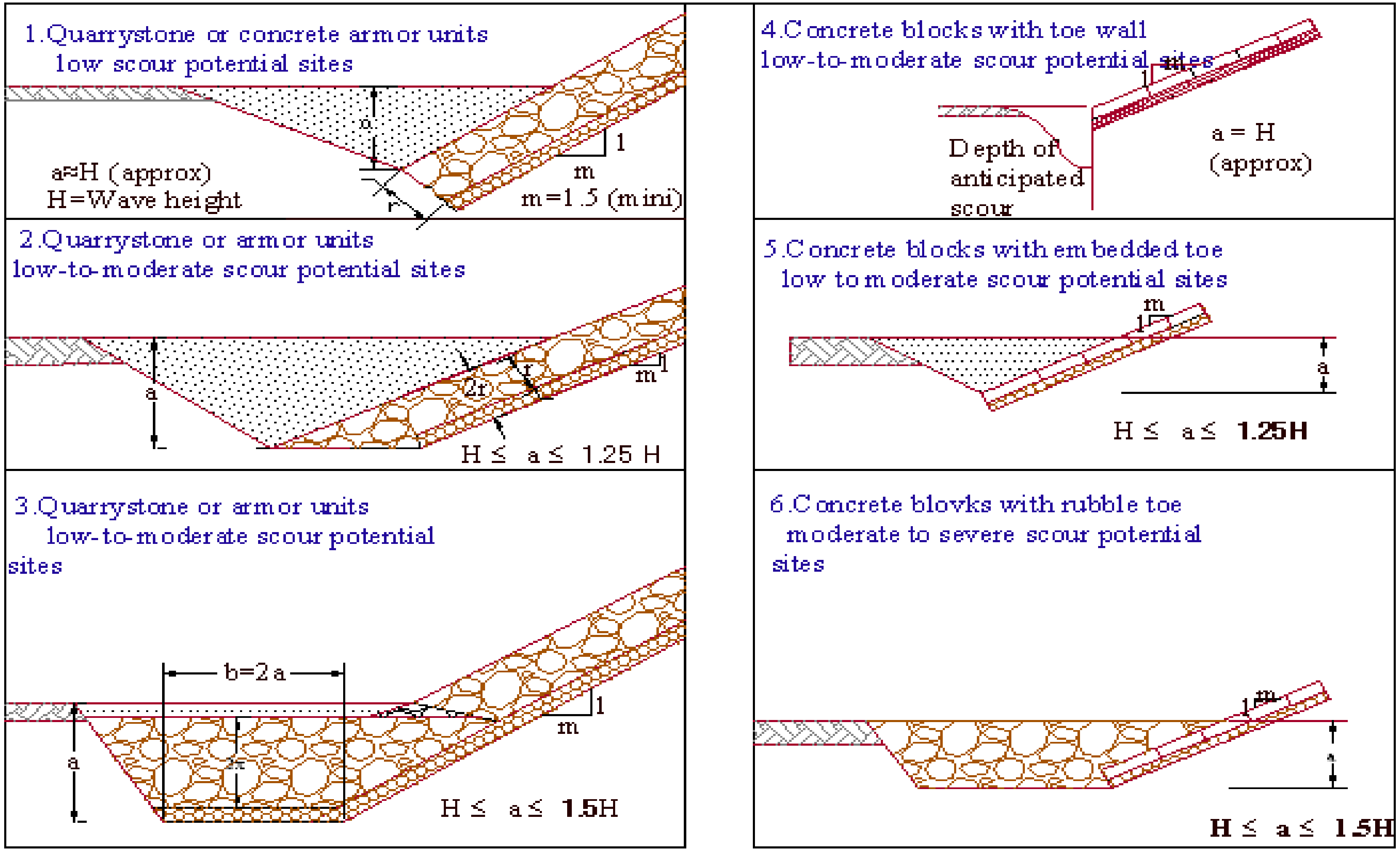
Settlement

A toe of sufficient width and height

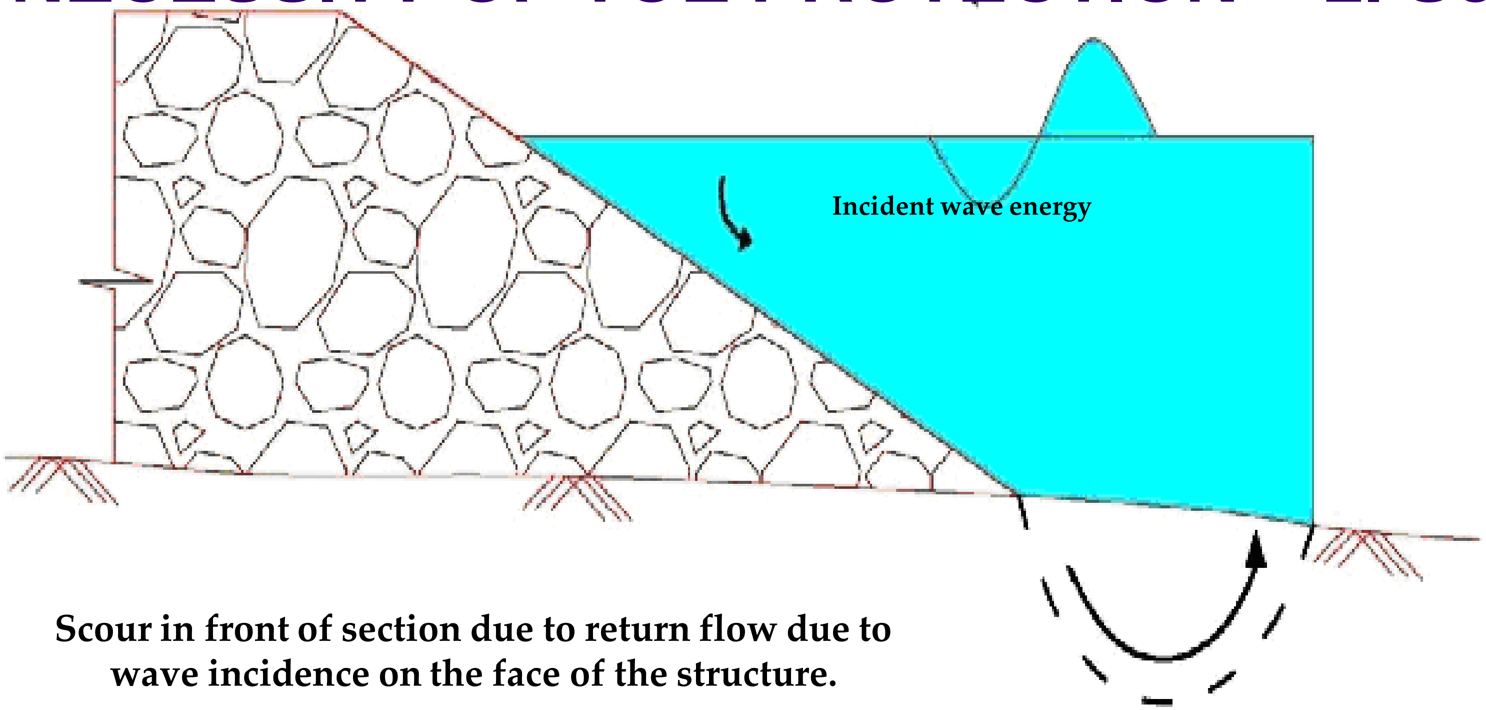
Address bearing and Scour

Also reduce the direct wave impact (if suitably chosen)

Toe Protection

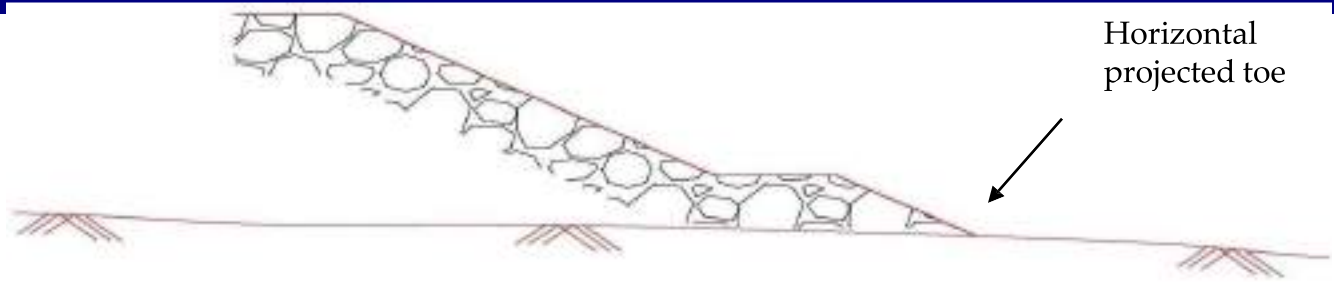


NECESSITY OF TOE PROTECTION – 1. Scour

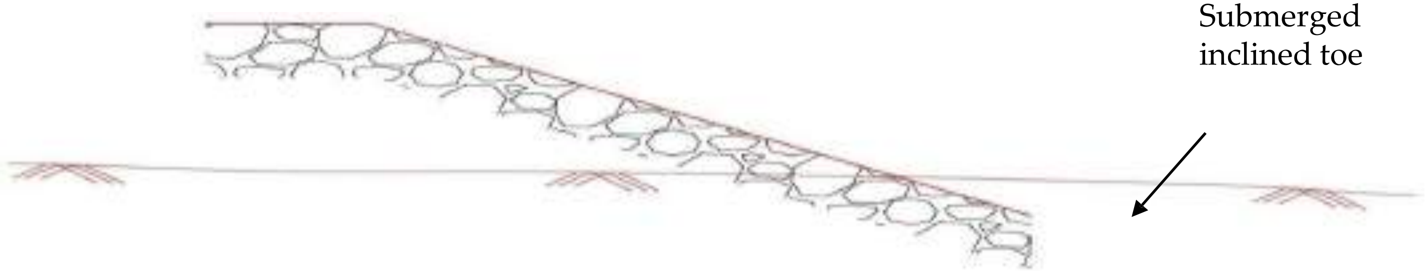


Scour in front of section due to return flow due to wave incidence on the face of the structure.

- Toe protection serves as a scour blanket -to prevent undermining - and reduces lateral stability.



Toe system: Projection of front inclined surface deep into seabed.



Toe system: Extension of sufficient length of front surface.

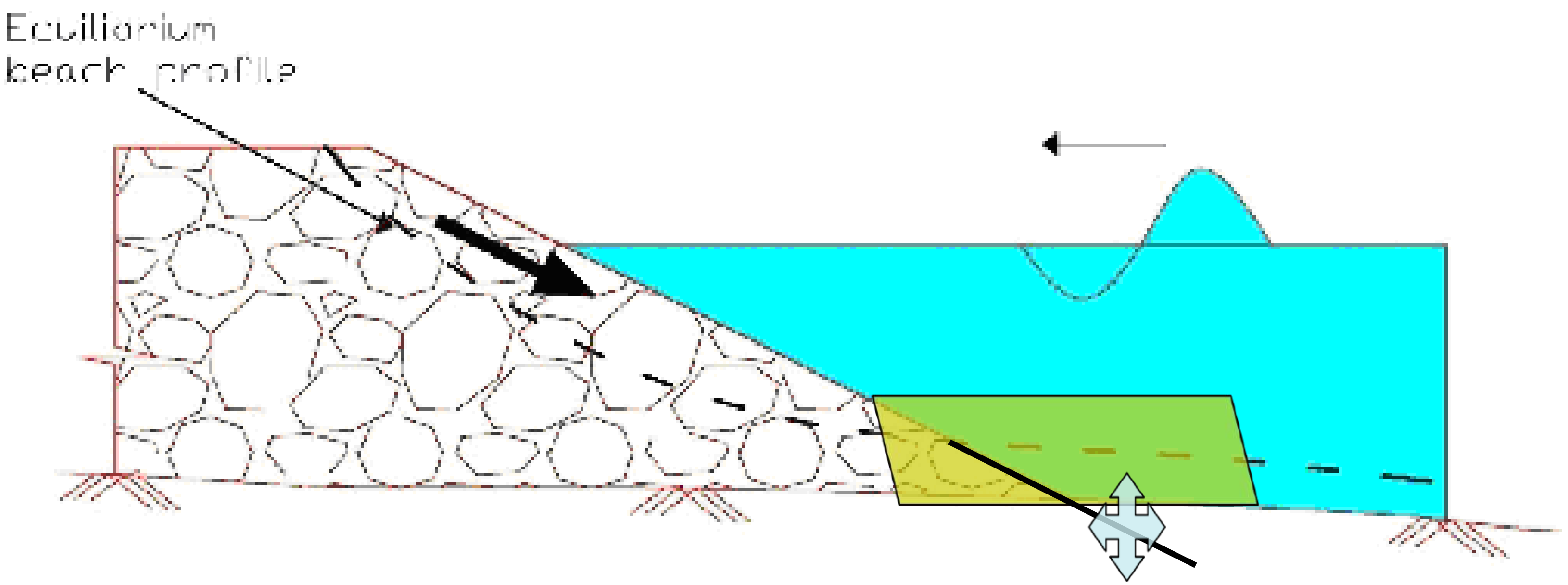
Principle of scour reduction

By dissipating the return flow over the horizontal or inclined top surface of the rubble mound toe.

Toe can be above seabed or submerged

NECESSITY OF TOE PROTECTION

2. Equilibrium profile

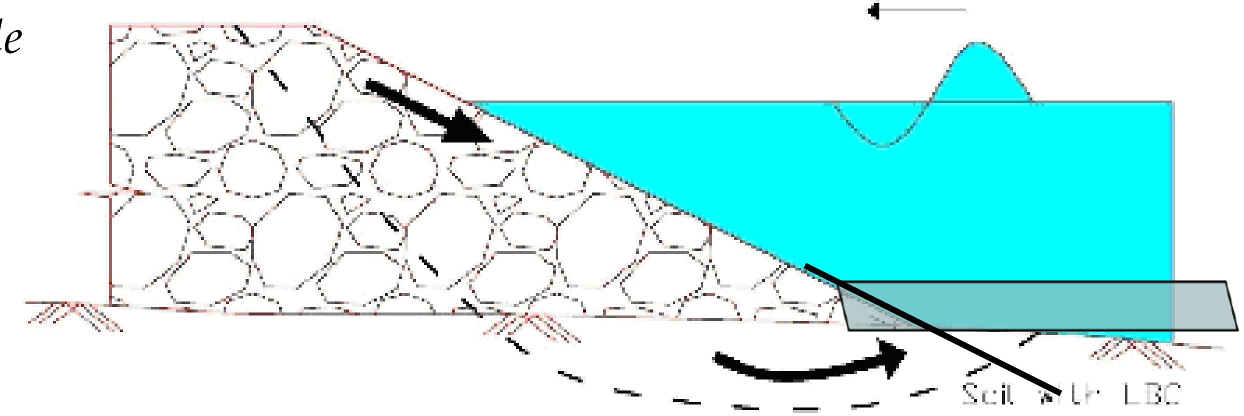


Reshape of rubble mound structure to its equilibrium profile.

NECESSITY OF TOE PROTECTION

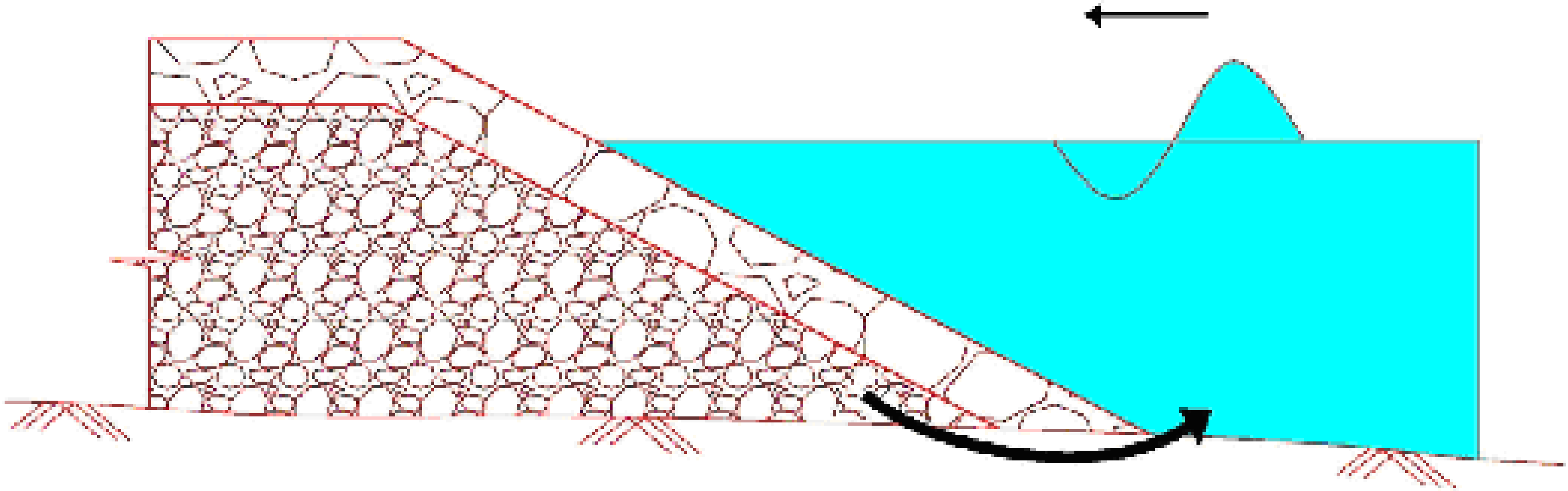
3. Breakwater on weak soil

Breakwater *-weak soil medium-* distribution of weight of the breakwater is not even into the sub-grade soil medium *-bulging of soil* – sliding of breakwater – *slip circle*

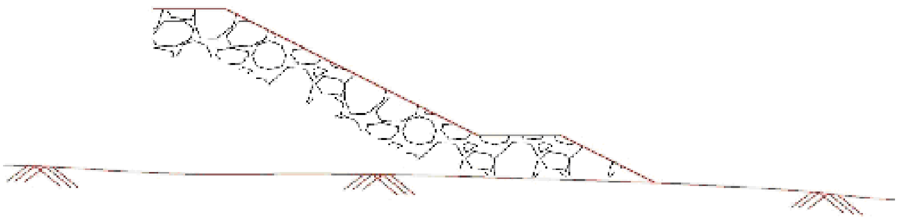


NECESSITY OF TOE PROTECTION

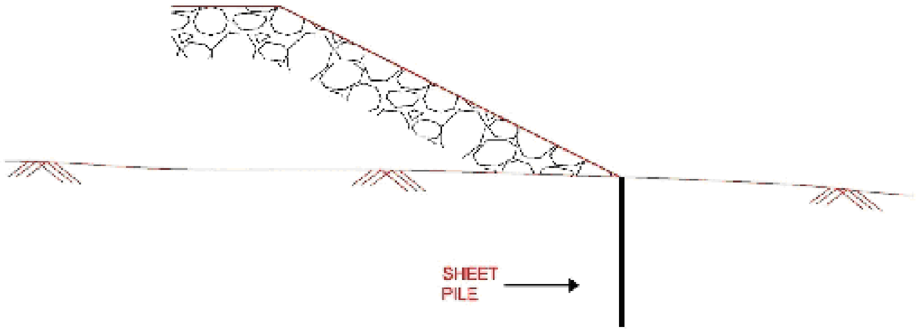
4. Leaching of core materials



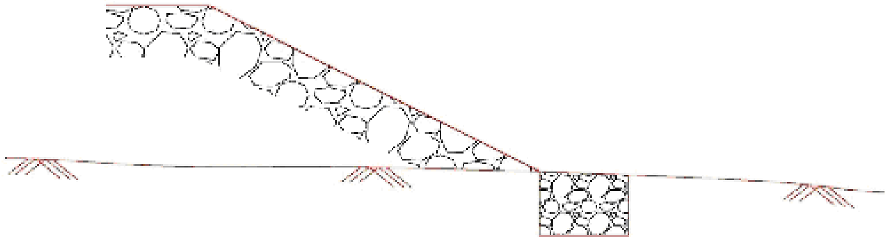
TYPES OF TOE PROTECTION



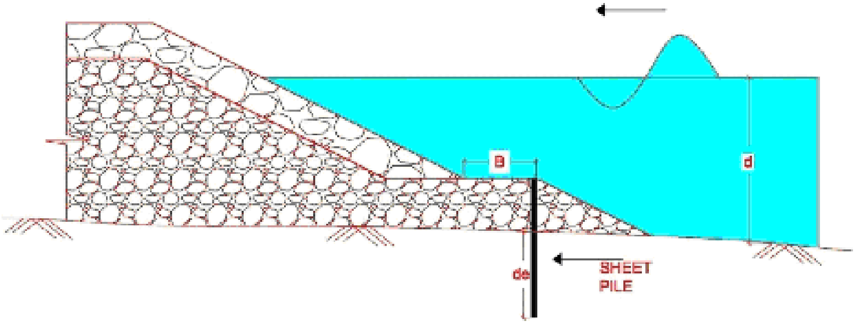
Horizontal projected toe above seabed.



Toe system with a sheet pile.

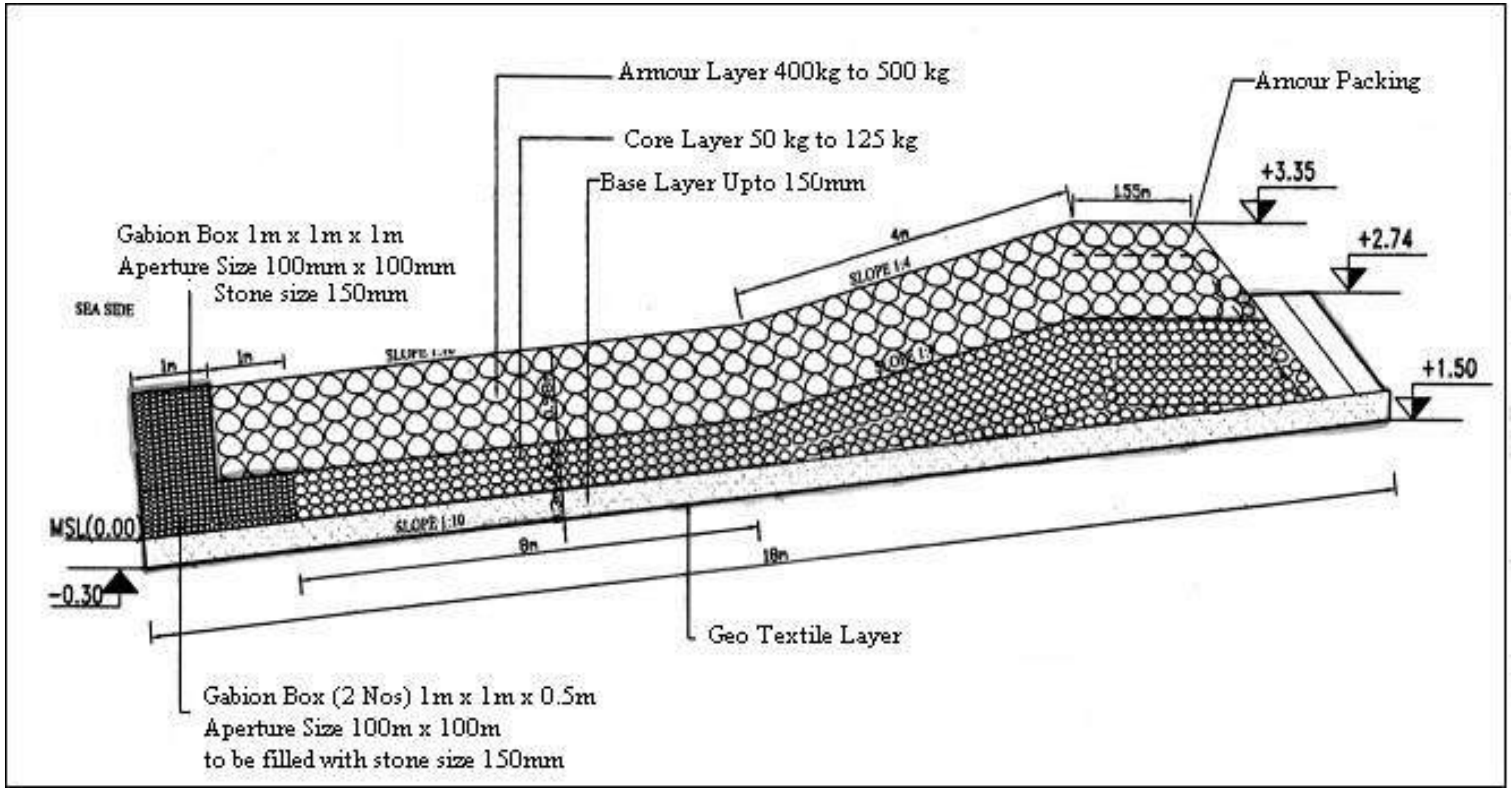


Submerged toe below seabed.



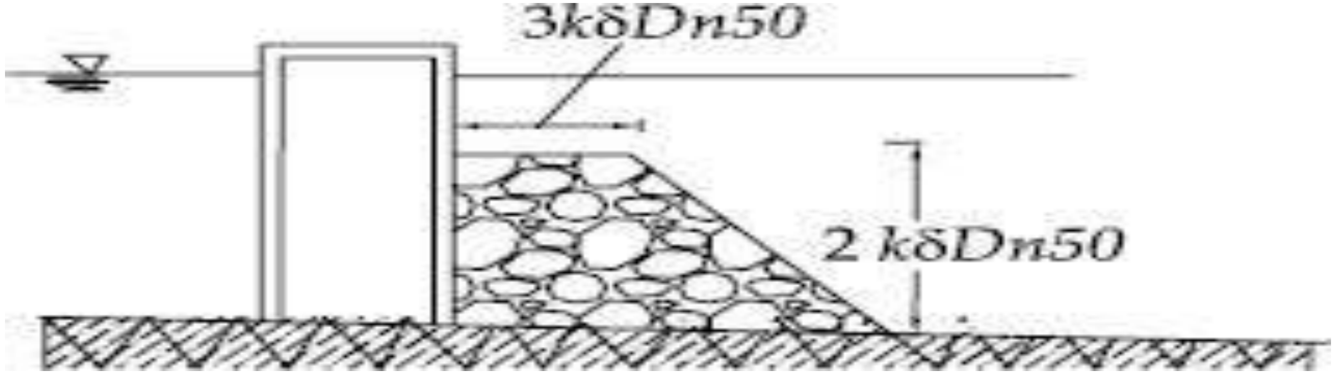
Combined sheet pile and project rubble mound toe.

Gabions as toe protection system for seawalls



Seawall section showing the toe protection system using gabions.

DESIGN ASPECTS OF TOE LAYER



W50 is the weight of armour stones
Dn50 is the mean diameter of toe stones and
kδ is the layer coefficient.

Design aspects of Toe Layer by Coastal Engineering Manual (CEM, 2002)

The general _thump rule_ is the *scour depth is about one wave height* or of the *order of the maximum size of the stone.*

Recommendation

Design of coastal structure to withstand environmental loads

- Wave climate
- Rock characteristics such as density and the size,
- Geo-systems, if any.

Overall stability of the structure (made up of geosynthetics)

- Armour protection of Geotextile tube using Gabions/ Rubble stones
- Toe plays a major role - irrespective of the flexibility of the geosynthetic system
- A submerged dissipative toe is important design element (to reduce the wave impact)

Thank you

Contact: sasraj@iitm.ac.in