Proceedings of the
SLGS Conference on

APPLICATIONS OF GEOSYNTHETICS IN CONSTRUCTION AND GROUND IMPROVEMENT TECHNIQUES

Wednesday, 14th March 2007

Organised by

SRI LANKAN GEOTECHNICAL SOCIETY

At Hotel Galadari Colombo

SRI LANKAN GEOTECHNICAL SOCIETY
SLGS
1986

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SRI LANKAN GEOTECHNICAL SOCIETY (SLGS)
SECRETARIAT: NATIONAL BUILDING RESEARCH ORGANISATION
99/1, Jawatte Road, Colombo 5, Sri Lanka.
A Member Society of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE)

Co-sponsored by
FINCO GROUP OF COMPANIES
49/16 Galle Road, Colombo, Sri Lanka
and
NOVA ENGINEERING SOLUTIONS
221/5, Sarojini Estate, Hokandara South, Hokandara, Sri Lanka
& EMAS KIARA GROUP OF COMPANIES, MALAYSIA

Venue: Hotel Galadari, Colombo

Sri Lankan Geotechnical Society

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Celebrates 20Yrs of service to the Nation

First Sri Lankan Geotechnical Society (SLGS) International Conference on Soil and Rock Engineering

Colombo, Sri Lanka -August 5~ 11, 2007

will present

- Technical Sessions covering over 200 Technical Papers from 35 countries and 20 Special Lectures,
- **Pre-conference Programme offering 7 Short Courses & Workshops,**
- **Technical Field Trips and Exhibition**

Venue: Galadari Hotel, Colombo

Tentative Programme

Date	Morning	Afternoon	Evening
August 5 (Sun)	Short Courses and Workshops		
August 6 (Mon)			
August 7 (Tue)			
August 8 (Wed)	Conference *Inauguration Session Main Session & Technical Sessions*		
August 9 (Thu)			Welcome Banquet
August 10 (Fri)			Cocktail / Tea Party
August 11 (Sat)	Technical Field Trips		

Pre-conference Short Courses
(5th ~ 7th August, 2007)

***One** day **courses** (Registration fee :* Rs. 4500 each)

- Soil and Sediment Remediation Technologies* (6th August)
- Geotechnical Design to Eurocodes 7 & 8 and Geotechnical Risk (6th August)
- Introduction to Geotechnical Earthquake Engineering (6th August)
- Design, Construction and Monitoring of Landfills (7th August)
- Unsaturated Soils: Latest Developments in Testing, Analysis and Design (7th August)

***One and half day courses** (Registration fee :* Rs. 7000 each)

- Rock Slope Stability Analyses(5th & 6th August)
- Block Theory and Applications for Surficial and Underground Rock Excavations (6th &7th August)

Note : * Short Course on Soil and Sediment Remediation Technologies will be sponsored and offered free of charge to selected applicants.

Technical Sessions (8th ~ 10th August, 2007)

Over 200 will be presented in 4 Parallel Technical Sessions on Tentative Themes of

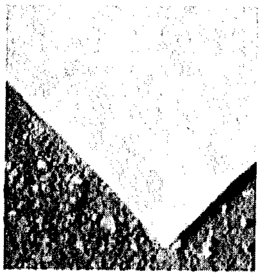
- Soil Properties
- Laboratory & Field Tests on Soils
- Unsaturated Soils
- Application of Geophysical Techniques
- Application of Statistics & Probability
- Analytical & Numerical Modeling
- Constitutive Models
- Fluid Flow in Geo Materials
- Soil-Structure Interactions
- Ground Improvement
- Soil Stabilization
- Shallow Foundations
- Pile Foundations
- Underground Excavations in Soils
- Dams and Embankments
- Geotechnical Construction
- Geo-environmental Engineering
- Geologic Hazards
- Soil Dynamics & Earthquake Engineering
- Landslides & Soil Slope Stability
- Rock Properties
- Rock Dynamics & Rock Cutting
- Rock Joint & Mass Properties
- Rock Foundations
- Surface Excavations in Rock
- Underground Excavations in Rock
- Tunnels and Shafts in Rock

Please see the **Conference Website** at URL **www.slgssr2007.org**

For more information and Application Forms, please write to :-Secretariat, **Sri Lankan Geotechnical Society**: c/o National Building Research Organisation 99/1, Jawatte Road, Colombo 5, Sri Lanka Tel. 2588946 ext. 228/224
or contact the Conference Co-chairs;
Dr. Athula Kulathilaka at University of Moratuwa Tel. 2650567ext. 2003 e-mail: sas@civil.mrt.ac.lk or
Mr. Kirthi Sri Senanayake at National Building Research Organisation Tel. 2588946 ext. 228 e-mail: senanayakeks@hotmail.com

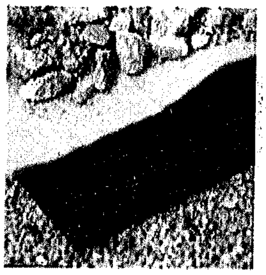
TENCATE

Polyfelt



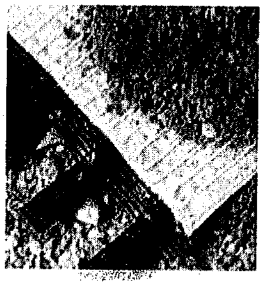
Polyfelt TS Non Woven Geotextiles

- High mechanical properties due to endless fibre manufacturing process
- High water flow and good soil retention property due to special needling technique
- High UV stability and durability



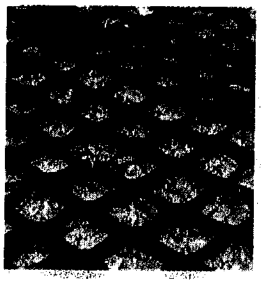
Enviromat GCL / Heavy Duty Polyfelt TS Geotextiles / Geonets / Megadrain

- High puncture resistance geotextile for geomembrane protection
- Low permeability GCL as primary and secondary liner system
- High chemical and UV resistance
- High permeability drainage of leachate and rainwater run off



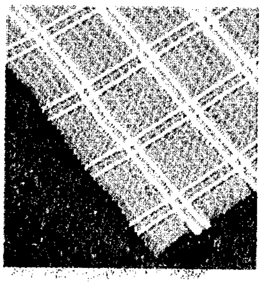
Polyfelt PEC / GX Geogrids / WX

- Suitable for granular and poor draining soils
- High tensile modulus at low strain
- Low tensile creep property for design > 100 years
- Quality product - British Board of Agrement (BBA) certified



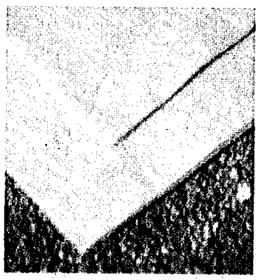
Envirocell / Polymat EM Series

- Retention of soil against steep slopes
- Soil erosion protection
- Enhance vegetation growth
- Vegetation on hard surface



Polyfelt PGM / PGM-G / Paving Fabrics

- Excellent sealing property for permanent adhesion of new pavement
- High tensile strength for pavement reinforcement



Polyfelt F Filter Geotextiles

- 2 layer system geotextile for optimum filtration performance
- Suitable for filtering fine silty soils
- Quality product - BAW (Germany) certified
- Designed to withstand high rock drop energy

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SLGS Conference on Applications of Geosynthetics in Construction and Ground Improvement Techniques

Wednesday, 14th March 2007 at Hotel Galadari

PROGRAMME

8.45–9.00	**Registration**
9.00–9.10	**Welcome Address** Mr. Kirthi Sri Senanayake, President-SLGS
9.10–10.30	**Geosynthetical Applications in Civil Engineering Designs and Constructions** Dr. G.P. Karunaratne, Consulting Geotechnical Engineer, PCI, Southern Transport Development Project and Former Associate Professor of National University of Singapore.
10.30–10.45 .	Tea
10.45–11.45	**Geosynthetical Applications in Civil Engineering Designs and Constructions (*cont....*)**, Dr. G.P. Karunaratne.
11.45–12.45 .	**Advantages of Paving Fabric for Road and Runway Construction** Mr. Michael Chong, Engineering Manager (Export Market), TenCate Geosyntetics Asia Sdn. Bhd, (formerly known as Polyfelt Asia Sdn. Bhd), Malaysia
12.45–1.45	Lunch
1.45–2.30	**Use of Heavy Tamping in Highway Construction – Experience in Southern Transport Development Project** Dr. Kamalnath Dissanayake, Chief Geotechnical Engineer, China Harbour Engineering Co., Southern Transport Development Project.
2.30–3.30	**A Review of Soft Ground Improvement by Electro Osmosis** Eng. Lee Eng Choy, General Manager (Technical), Emas Kiara Group of Companies, Malaysia. Manufacturers of KiaraTex Geosynthetic Products.
3.30–4.15	**Discussion**
4.15– 4.30	Tea

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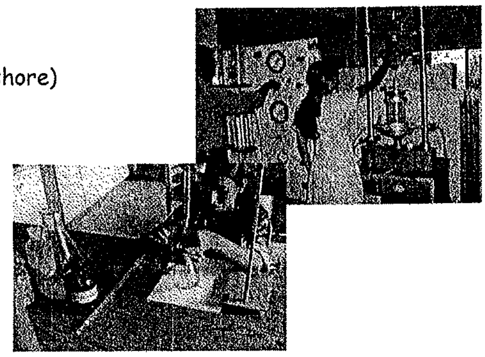
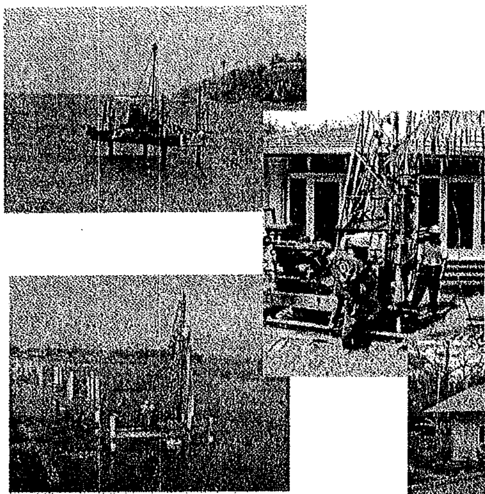
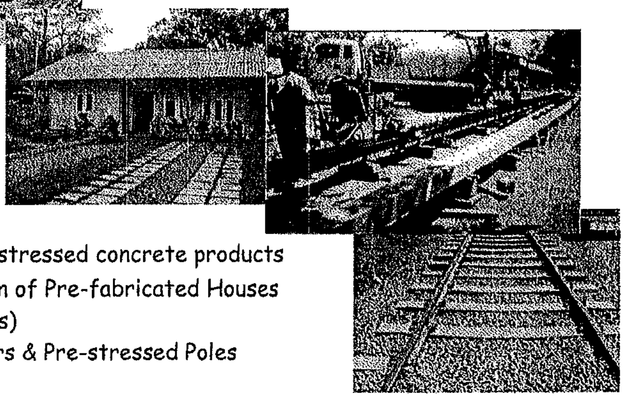


* Geotechnical Investigation (Offshore & Onshore)
* Piling (Offshore & Onshore)
* Laboratory Testing (Material)
* Laboratory Testing (Environmental)



* Specialized Foundation Systems
* Full Scale Load Test (Piles, ect)
* Micro Piling
* Pile Dynamic Load test
* Pile Integrity Testing.



* Construction of Tube wells
* Manufacture of Pre-cast & Prestressed concrete products
* Manufacture, Supply & Erection of Pre-fabricated Houses
* Low Cost Houses (Model Houses)
* Production of Concrete Sleepers & Pre-stressed Poles
* Concrete Ready Mix Plant

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Sri Lankan Geotechnical Society Conference on Applications of Geosynthetics in Construction and Ground Improvement Techniques

CONTENTS

Message from the President of the Sri Lankan Geotechnical Society

Dear Participant,

The Sri Lankan Geotechnical Society (SLGS) was founded on 27th February 1986 to provide the much needed forum for scientists, engineers and geologists for the objective of exchanging and enhancing the knowledge in the field of Geotechnical Engineering in Sri Lanka.

In way of achieving this objective SLGS has been successful in organising various activities and events such as the monthly Geoechnical Forum that offers public lectures on the latest developments in the field of geotechnical engineering, field visits that demonstrate various geotechnical engineering techniques, annual seminars & conferences, workshops and Project Day that encourages young engineers in research work and promoting their presentation skills. In addition, SLGS issues a quarterly SLGS Newsletter and yearly SLGS Geotechnical Journal.

Since its recognition in 1994 as a Member Society of the prestigious International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE) SLGS has broadended its vision and prospects internationally. This has enabled SLGS to organise events jointly with other National Geotechnical Societies. Affiliation with the ISSMGE also provides SLGS members the privilege to publish and participate in International and Regional Conferences organised by ISSMGE while opening the door to the young Sri Lankan Geotechnical Engineers to participate in the international Young Geotechnical Engineers Conferences.

SLGS has just completed 20 years of its service, contributing extensively in the national development programmes. Our success lies on the active participation of our members, well wishers and sponsors here and abroad and the devoted volunteers. To celebrate the Anniversary, SLGS has organized many events and programmes.

To start with, SLGS has organized this Conference on Application of Geosynthetics in Construction and Ground Improvement Techniques which is co-sponsored by M/s Finco Group of Companies and M/s Nova Engineering Solutions, Sri Lanka & Emas Kiara Group of Companies, Malaysia.

As the major event to mark the anniversary, SLGS is organizing the First Sri Lankan Geotechnical Society (SLGS) International Conference on Soil and Rock Engineering, Colombo, August 5-11, 2007 to be held at Hotel Galadari, Colombo. This event begins with a 3 day programme of Pre-conference Short Courses and Workshops followed by the main conference where over 200 papers from 35 countries will be presented in the Technical Sessions. Technical Exhibition and Field Visits will also be organized. ISSMGE, has extended its full cooperation and institutes such as the University of Arizona, USA, Tsin Hua University, China are collaborating with the organisers.

SLGS is organizing another workshop on Ground Improvement Techniques with the participation of Prof. Pedro S Pinto- President ISSMGE, Prof M R Madhav- Vice President Asia of ISSMGE, Prof E C Shin and Prof J Sangseom from South Korea and Prof G Herten from Germany on 17 th and 18 th of December.

On behalf of the SLGS, I warmly welcome your active participation to make all these events successful.

Thank you,
Sincerely,
Kirthi Sri Senanayake,
President, Sri Lankan Geotechnical Society.

Bio Data of Dr G P Karunaratne

Dr G P Karunaratne graduated from the University of Ceylon at Peradeniya, Sri Lanka and was awarded PhD in soil mechanics by the University of Manchester. He was associated with the University of Ceylon, Peradeniya and the National University of Singapore (NUS) for a combined period of 30 years. As an associate professor in Civil Engineering at the time he left NUS in 2002, he had contributed to research, teaching and the local and regional construction industry through designs, consultation and testing of geotechnological material such as rocks, soils and geosynthetics. He has been honoured by the continuing appointment as an Adjunct Associate Professor at the National University of Singapore during which he is being consulted by research students at the same university.

His contribution to NUS Patented Fibredrain, developed for the Changi International Airport Construction in Singapore, and associated development of geosynthetics research resulted in NUS granting spin-off status to the Geo5 R&D and Services Pte Ltd, a private consulting company in Singapore, in which he acts as a consultant and a director. His expertise in geotechnology brought him many interesting problems from construction industry ranging from land reclamation to ground improvement. These led to the development of layered clay-sand reclamation scheme, Fibredrain, geosynthetics and band drain strain-gauging schemes, laboratory and field testing, damage assessment of geotextiles and soft ground improvement schemes including heavy tamping. Recently his interests were directed towards electro-osmotic treatment of soft soils, blast resistant reinforced soil structures, embankment piles and slope repairs.

In geosynthetics, he has been instrumental with his colleagues in conducting successfully the 5th International Conference Geosynthetics in Singapore in 1994. He served as a council member of the International Geosynthetics Society (IGS) and a committee member of the Southeast Asia Chapter of IGS. Recognition of his work internationally resulted in a joint Key Note speech presentation at the 8th International Geosynthetics Conference held in Yokohama in September 2006. He is also in the editorial board of the Geosynthetics International Journal since its inception. He has well over 200 publications in Journals and International Conferences.

He is a member of Australian, British, and Singapore institutions of Civil/Engineers and a Fellow of American Society of Civil Engineers. He is also a professional or chartered engineers of the former three countries.

Course Organized by

Sri Lanka Geotechnical Society

GEOSYNTHETICS
- Introduction & Applications

G P Karunaratne
BSc(Eng), PhD (Manch)
PE(Sing), CPEng(Aust), CEng(UK)
FASCE, MICE, MIES, MIE(Aust)
Pacific Consultants International

Application of Geosynthetics in Design and Construction in Civil Engineering - 14 Mar 2007

Outline of Presentation

- Introduction and background
- Type of Geosynthetics
- Basic Polymers
- Functions and various applications
- Reinforced Soil - Concepts and Design Considerations
- Filtration and Drainage Function
- Landfill and other applications

Background

- Geosynthetics: Products manufactured from polymeric material (synthetics and natural) used with soil, rock, or other geotechnical engineering related material as an integral part of a man-made project, structure, or system (ASTM, 1998; IGS, 1994)

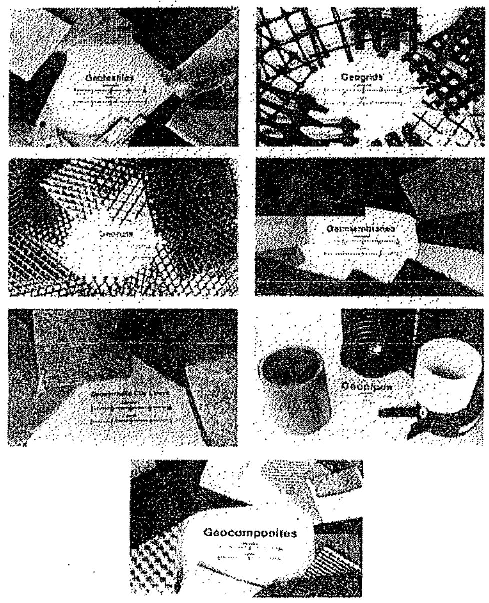
User Requirements

Engineers need to know:

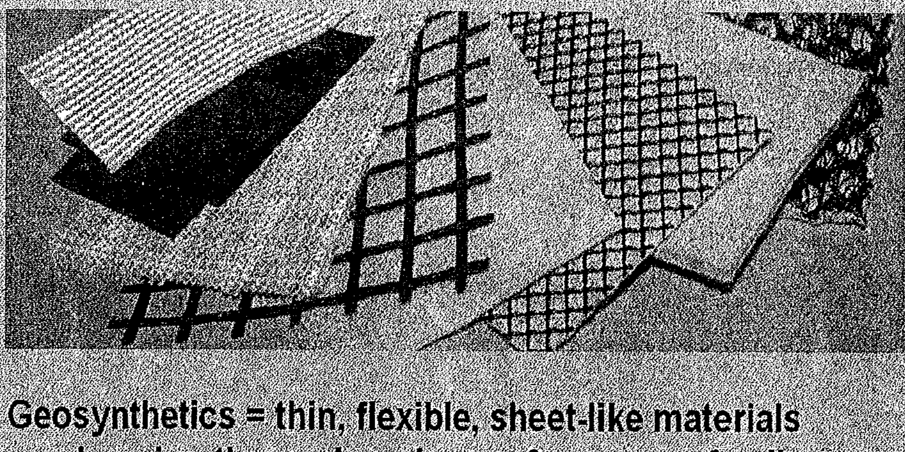
- Product types {What are geosynthetics?}
- Applications {Where can they be used?}
- Design methods {How can they be used?}
- Reliability {Safety, long-term effects?}
- Cost {$$$, fast construction?}
- Construction control {How to ensure proper control?}
- More information {Where to get?}

Geosynthetic Products

- Geotextiles
- Geogrids
- Geonets
- Geomembranes
- G.Clay Liners
- Geopipes
- Geotubes
- Geocomposites
- Others (geo-***)
- Erosion control



Products and Raw Materials



Geosynthetics = thin, flexible, sheet-like materials enhancing the engineering performance of soils

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Types of Geosynthetics

- $k_{Geosynth} << k_{Soil}$
  Geomembranes & Geomembrane Related Products
- $k_{Geosynth} \sim k_{Soil}$
  Geotextiles & Geotextile Related Products
- **Geocomposites**

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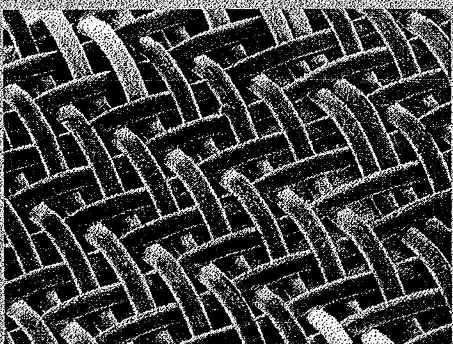
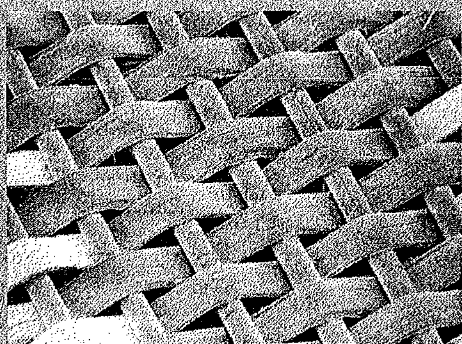
Geotextiles 1 – Woven Geotextiles



Monofilament-on-monofilament



Monofilament-on-tape

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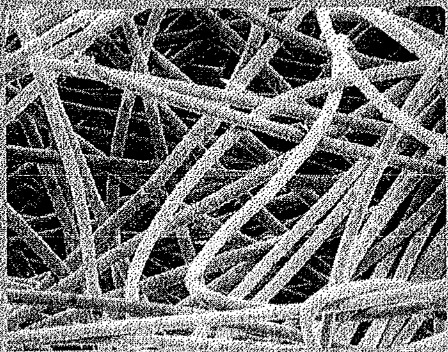
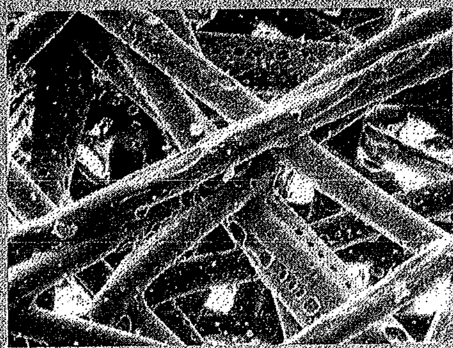
Geotextiles 2 – Non-woven Geotextiles



Mechanically bonded non-woven



Thermally bonded non-woven

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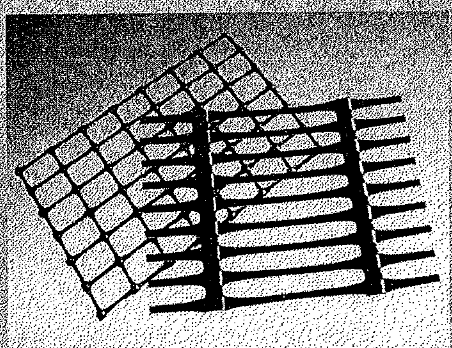
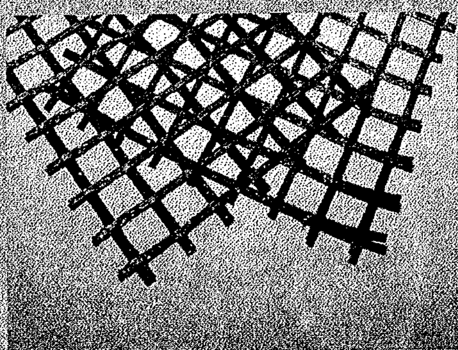
Geogrids



Integral junction geogrids



Woven junction geogrids

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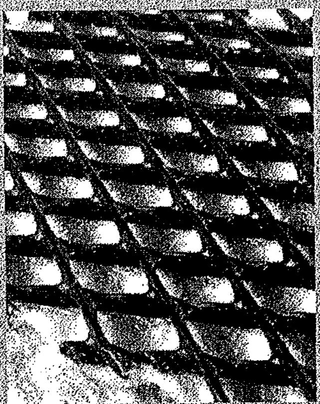
Other Geotextiles/Geogrid related Products

- **Geogrids** - 2 sets of orthogonal load bearing elements
- **Geostrips** - 50 to 100 mm wide, high strength
- **Geonets** - typically diamond shaped aperture, low strength
- **Geomeshes** - large aperture, low strength
- **Geomats** - e.g. extruded monofilament mats
- **Geocells** - interconnected cells filled with soil in-situ
- **Geospacers** - high transmissivity, used for composite drains
- **Geocontainers** - tubes, bags, gabions etc.

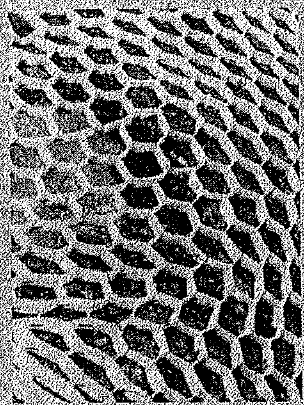
Application of Geosynthetics in Design and Construction in Civil Engineering - 14 Mar 2007

Other Geotextile Related Products



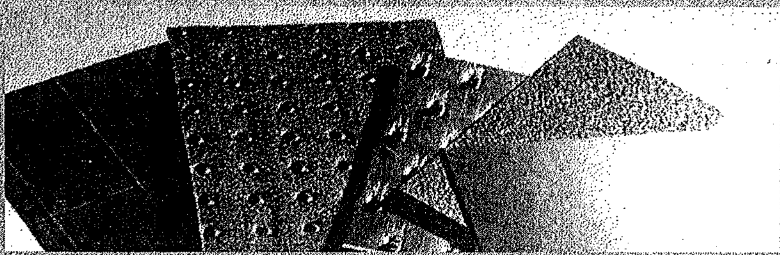
Geonet

Geomat



Geocell

Geomembranes



- Impermeable sheets acting as barriers to fluids
- Factory produced using thermoplast polymers, elastomers, rubber compounds or bitumen
- Grid reinforcement or special surface possible

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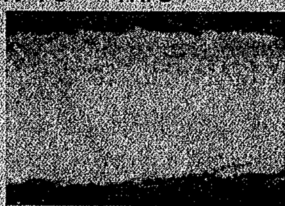
Application of Geosynthetics in Design and Construction in Civil Engineering - 14 Mar 2007

Geomembrane Related Products

- Geosynthetic Clay Liners (GCL) / Clay Geosynthetic Barriers (CGB):
  - Bentonite sandwiched between two geotextiles
  - Swells on wetting to form a low permeable layer
  - Permeability in the range of 1 to 5 x 10^{-11} m/s



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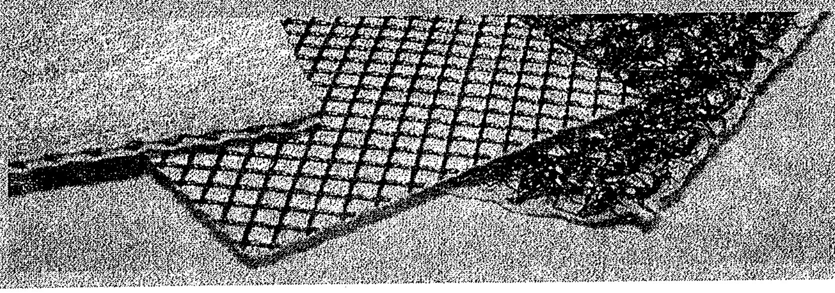
Geocomposites

- Two or more different types of geosynthetics
- Combined in-situ or prefabricated
- E.g. sheet drains = geospacer with geotextile filter on one or both sides



Polymeric Material

- Long chain polymers from plastic industry
- Polymers from natural materials

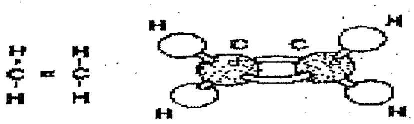


Fig. 1. Ethylene monomer

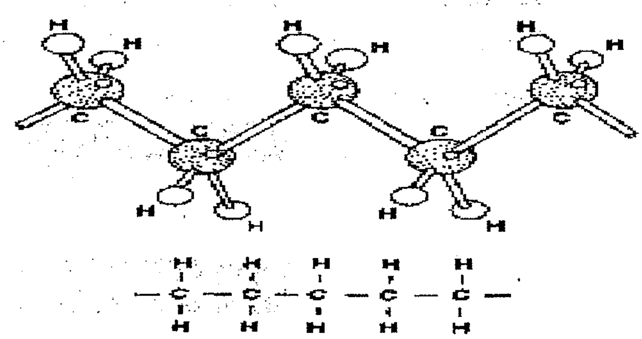
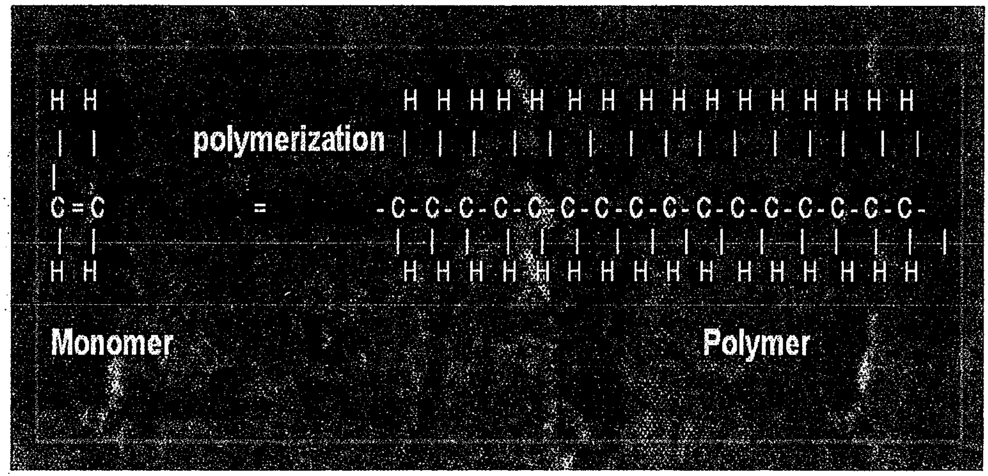


Fig 2. Polyethylene molecule

Polymer Chains



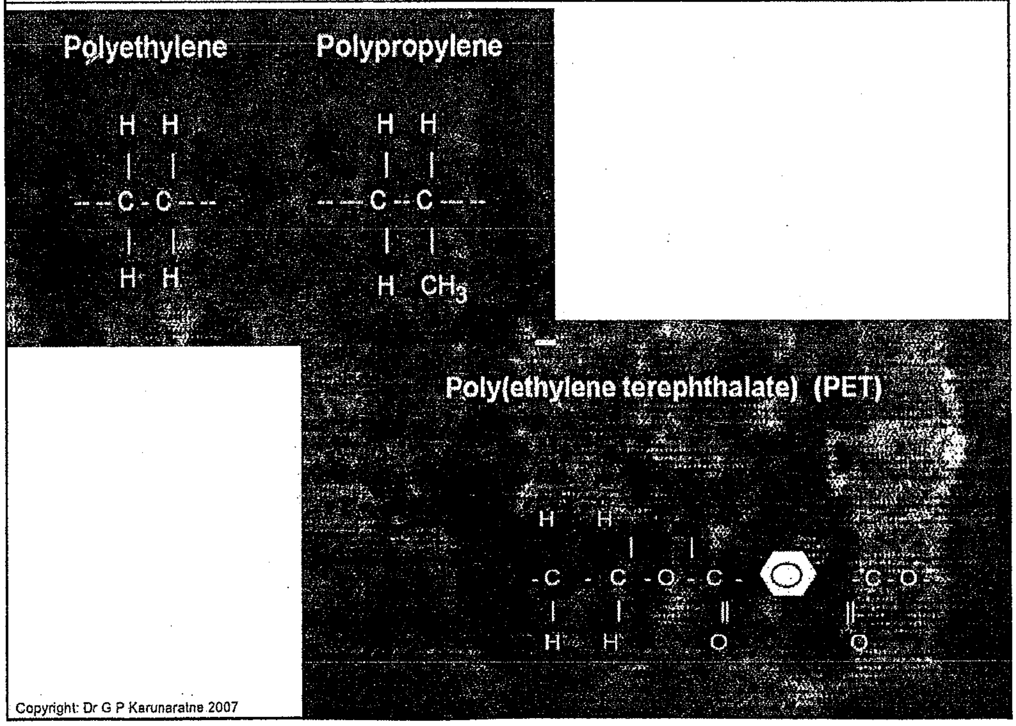
Polymer Action

- Polymer: 'Many parts' of monomer
- Monomers have the smallest Molecular weight
- Degree of polymerization (number of times a repeating unit occurs) determines molecular weight

Common (Synthetic) Polymer Types

- Polyethylene (PE)
- Polypropylene (PP)
- Polyvinyl chloride (PVC)
- Polyester (Polyethylene terephthalate) (PET)
- Polyamide (PA) (Nylon 6/6)
- Polystyrene (PS)

Application of Geosynthetics in Design and Construction in Civil Engineering - 14 Mar 2007



Polyester (Polyethylene terephthalate) PE
polystyre (...)

Grades Of Polyethylene (ASTM D 1248)

- Low Density Polyethylene (LDPE)
  0.910 < Density < 0.925
- Medium Density Polyethylene (MDPE)
  0.926 < Density < 0.940
- High Density Polyethylene (HDPE)
  0.941 < Density < 0.965

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Effect of Increased Polymerization

Increase in *average* Molecular Weight

→ increases
- Strength
- Elongation
- Impact strength
- Heat resistance

→ decreases
- Processability
- Flow behaviour

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Geosynthetic Market Growth

- The total market in North America in 1992 was US $1.3 billion

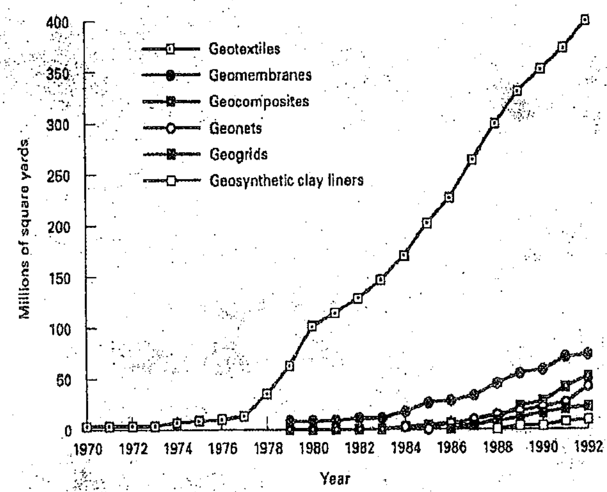

Figure 1.2 Estimated geosynthetic market in North America.

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Sales Pattern in 2001

- Geotextiles: Largest share of demand for geosynthetics in 2001, with roughly 75 percent of volume sales
- Geomembranes: Led geosynthetic sales in value terms as they cost more than geotextiles on a dollars per square yard basis.
- Geogrids: Smaller volume geosynthetic products are expected to achieve stronger demand growth through 2006, albeit from a small base.
- Geocomposites: Gains for these smaller volume products will be driven by the ongoing development of new applications

http://www.mindbranch.com/page/catalog/product

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Sales Pattern in 2001

- **Ground stabilization and reinforcement**: Continues to lead demand, accounting for 29 percent of sales in volume terms in 2001, among the various markets for geosynthetics.
- **Solid waste disposal, erosion control markets fastest growing:** Stronger annual gains are projected for markets such as solid waste disposal and erosion control, where geosynthetics benefit from the performance advantages of the materials relative to competing products.

http://www.mindbranch.com/page/catalog/product

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2007 January

- US demand for geosynthetics is projected to increase five percent per year through 2010 to more than 850 million square yards http://www.mindbranch.com/catalog/print

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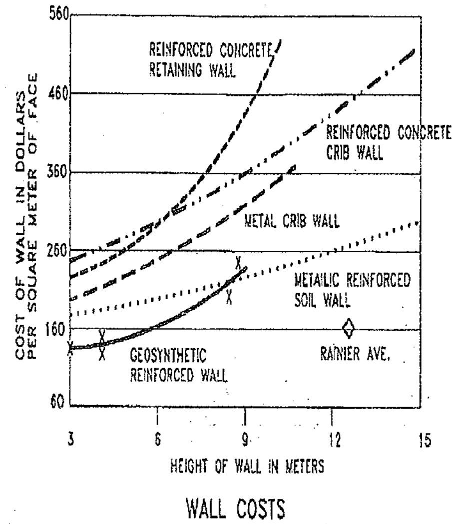
Cost Analysis

- Cost savings
- Compare cost-benefits between geosynthetics and other products generally employed



Production

- Polymer with additives
- Components
- Conversion into finished product

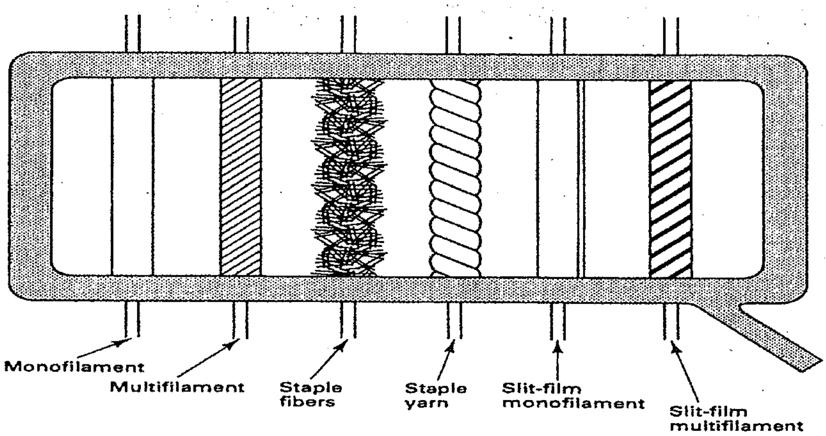


Figure 1.11 Types of polymeric fibers (or yarns) used in the construction of geotextiles.

Geosynthetics Functions and Applications

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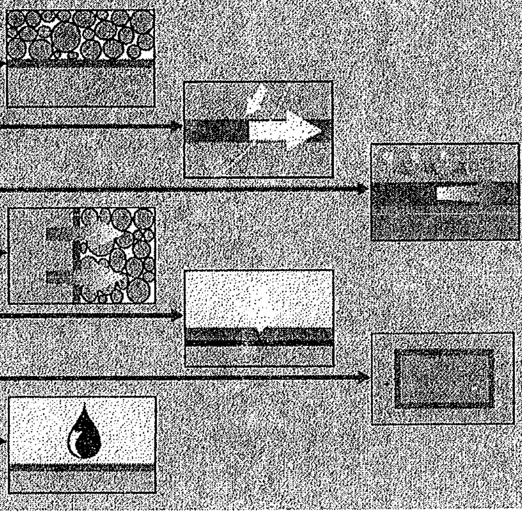
Functions

- Filtration
- Drainage
- Separation
- Reinforcement
- Fluid barrier } newly added
- Protection

Primary Function
Secondary Function
Tertiary Function

Roles of Geosynthetics

- Separator
- Reinforcement
- Drainage
- Filter
- Energy absorber
- Container
- Barrier



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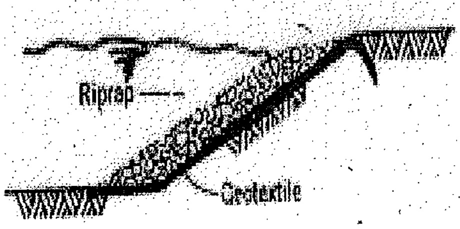
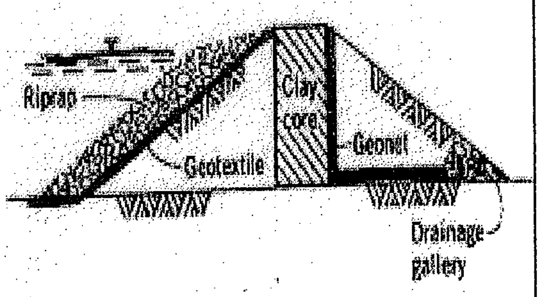
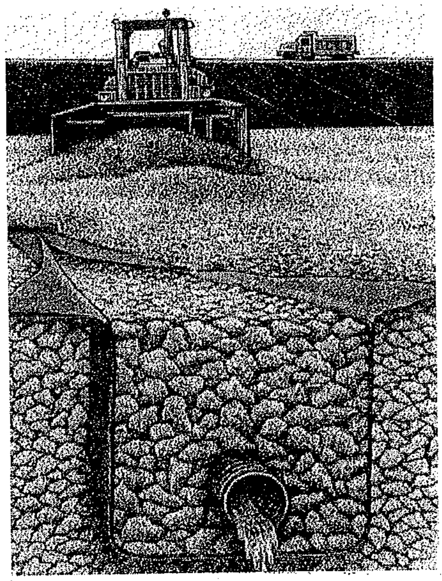
Filtration Functions





Drainage Functions



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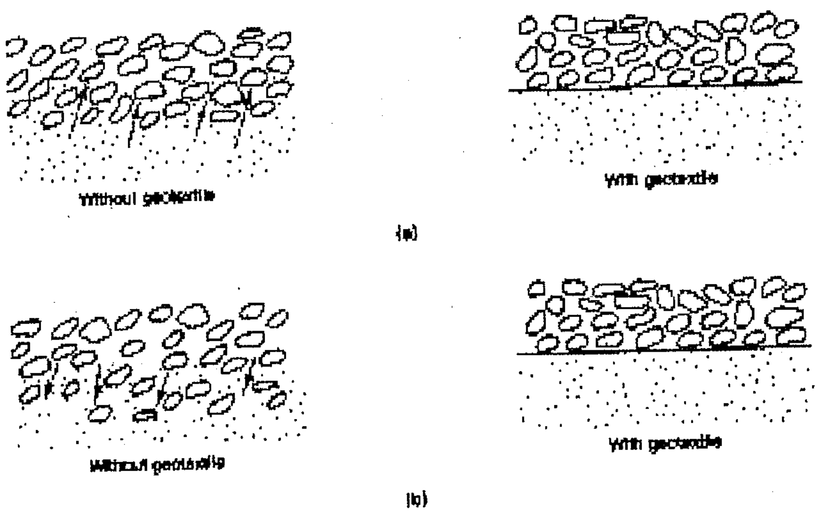
Separation Functions



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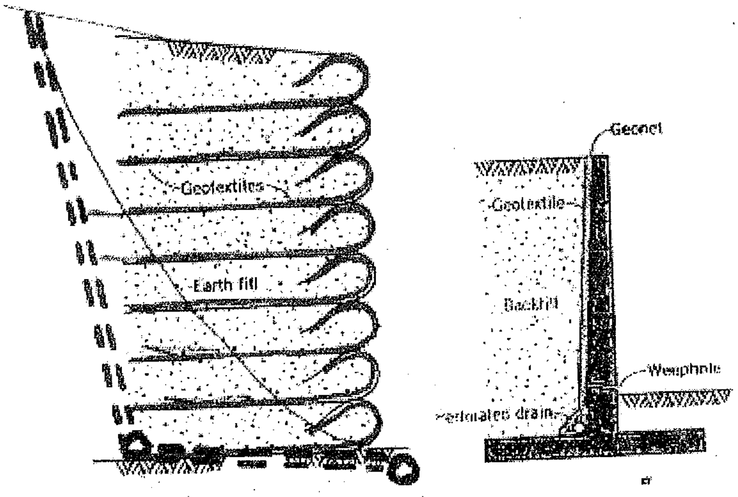
Reinforcement Functions



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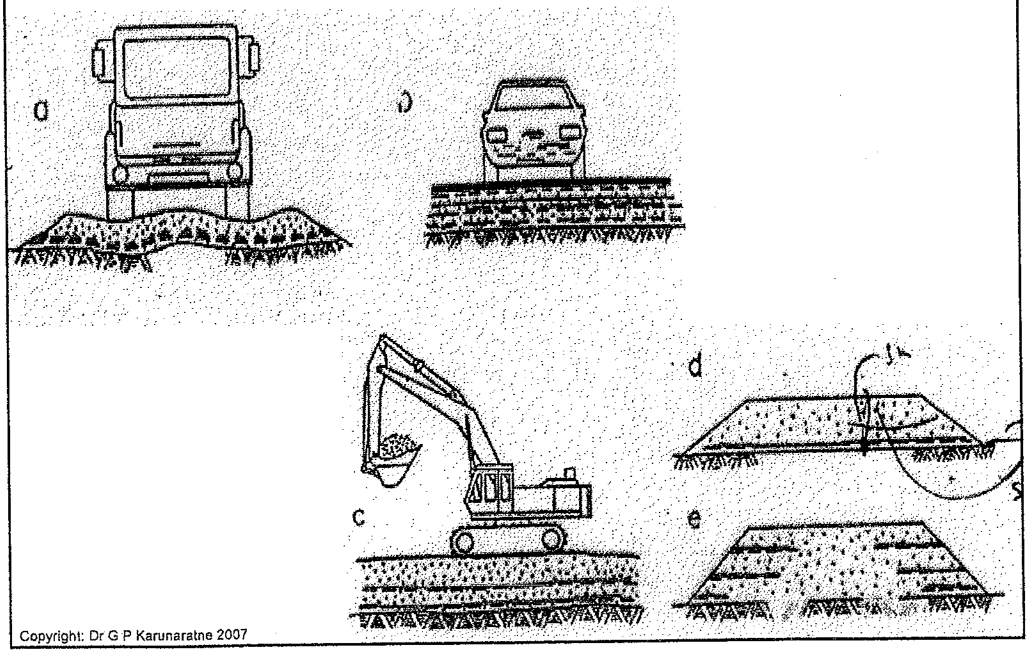
Reinforcement Functions



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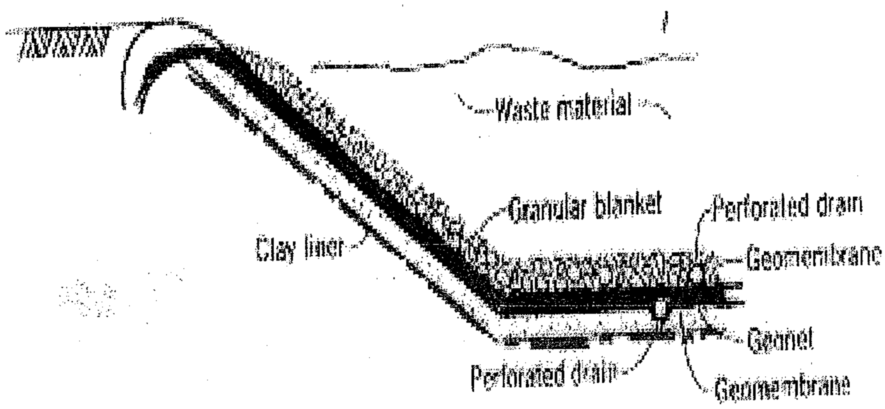
Fluid Barrier Functions



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Application 1 – Reinforced soil walls and slopes

History

- Agar-Quf Ziggurrat – 5 km north of Baghdad – Constructed with clay bricks (130 – 400mm thickness) reinforced with woven mats of reed laid horizontally on a layer of sand and gravel at 0.5m to 2m vertical spacing. Reeds were made into ropes 100mm dia to act as reinforcement. Originally 80m tall, now 45m, and 3000 years old.
- Great wall of China, 200 B.C. reinforced with weeds and soils

- 2000 yrs ago- Romans – used alternate layers of logs and earth fill
- 1200 yrs ago- Thames river – 1.5 km long, 2m high timber wharf structure
- 150 yrs ago - British army used in reinforced soil walls

- 1904 In dykes for river bank protection and in dams
- Munster, 1925, wooden reinforcement members and light facing

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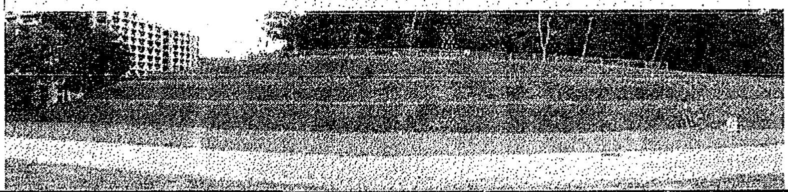
J. Wei, K.C. Chua, W. H. Ho,
S. K. Lim

HOUSING & DEVELOPMENT BOARD

Performance of a geosynthetic reinforced steep slope in residual soil



NUS
National University of Singapore

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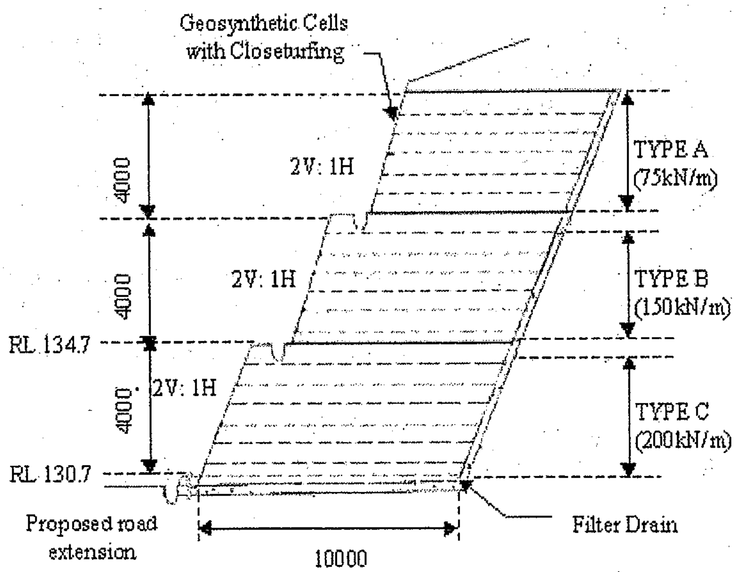
Application 1 – Reinforced soil walls and slopes



Application 1 – Reinforced soil walls and slopes

Will discuss more in detail in the afternoon...

Application 2 – Filtration/Drainage applications

Functions of a Filter

Retain particles of the base soil to be filtered
Avoid piping

- Allow free flow of water
  - upstream of the filter
    Avoid *external* clogging
    (With unstable soils)
  - through the filter
    Avoid *internal* clogging
- Survive construction and environmental stresses
- Function can be provided by either natural aggregates or by Geotxtiles

Application 2 – Filtration/Drainage applications

	AGGREGATES	*GEOSYNTHETICS*
SIMILARITIES		
- Risks of internal clogging by 1. finer particles of the soils to be filtered 2. aerobic bacterial activity (ochre clogging) 3. deicing salt precipitation 4. ice lens formation within the frost penetration zone		
DIFFERENCES		
- Thickness	High (> 150 mm)	Low (< 30 mm)
- Porosity	25 - 40 %	75 - 95 %
- Capillary rise h_c	Important (h_c < 500 mm)	Low to none (h_c < 50 mm)
- Tensile strength	None	Low to high
- Compressibility	Negligible	Medium to high
- Transmissivity under confining stress	Invariable	Variable
- Uniformity	Variable gradation as per borrow pit	Factory-controlled mass per unit area and thickness
- Durability	Completely inert	Altered by ultraviolet rays
- Installation	Must not be contamined by the surrounding soil. Compaction needed	Must be installed in intimate contact with the soil to be filtered Installation eased by seaming of the joints
- Risk of damage	None	Subject to puncture and tearing

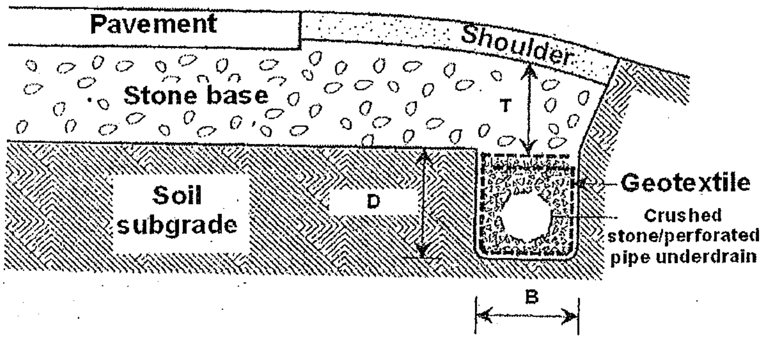
Application 2 – Filtration/Drainage applications

- **Clogging: the voids of a medium are progressively filled by solid matter to the point that the passage of water is compromised**
  - Decrease in hydraulic conductivity
- **Internal clogging**
  - By mineral particles
  - By precipitation and chemical deposition in the voids by water containing iron, de-icing salts
  - By biological growth encrustation in aerobic conditions

Application 2 – Filtration/Drainage applications

Filter Applications

- Wrapping of trench drains (Koerner, 1998)



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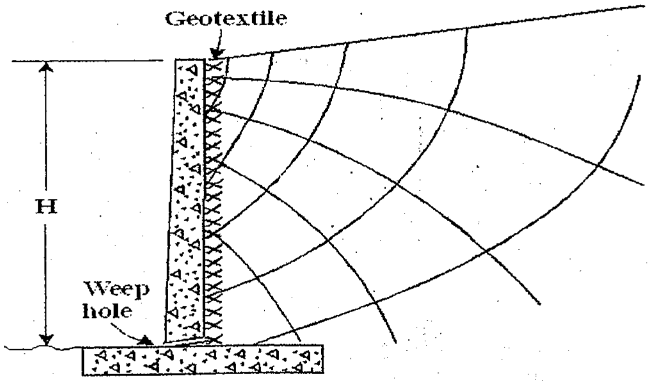
Application 2 – Filtration/Drainage applications

Filter Applications

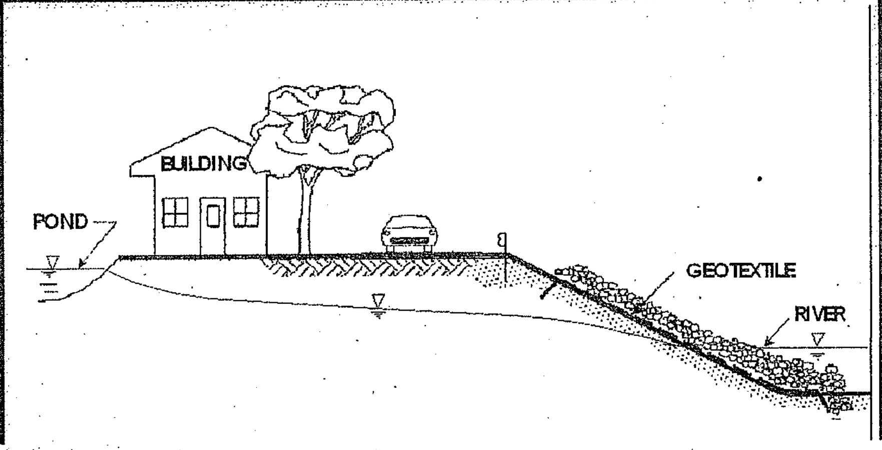
- Wall drains



Application 2 – Filtration/Drainage applications

Filter Applications

- Erosion protection (Pilarczyk, 2000)



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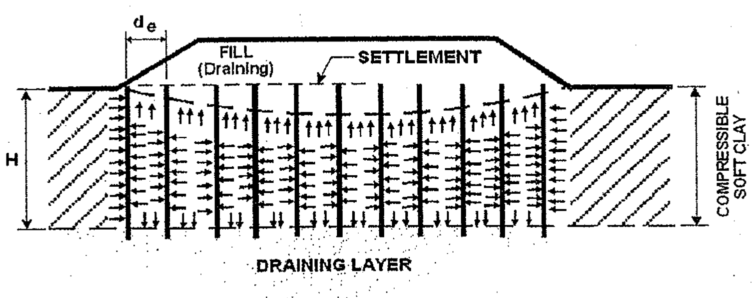
Application 2 – Filtration/Drainage applications

Filter Applications

- Vertical consolidation drains (Van Santvoort, 1994)



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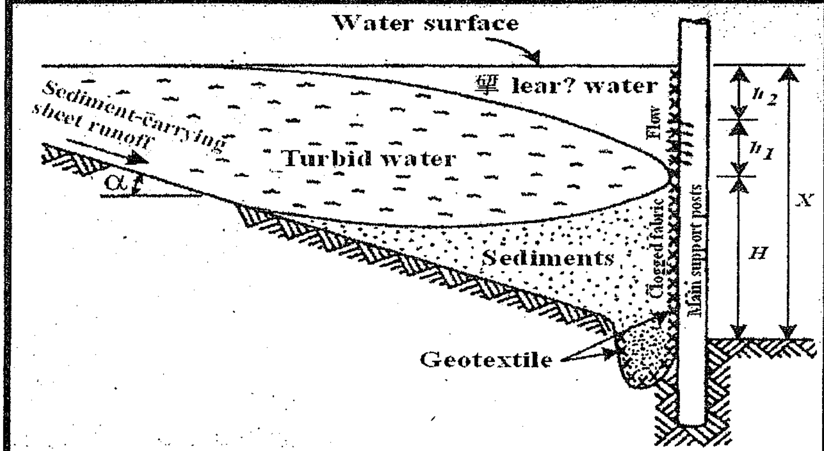
Application 2 – Filtration/Drainage applications

Filter Applications

- Silt fences (Holtz, et al., 1997)



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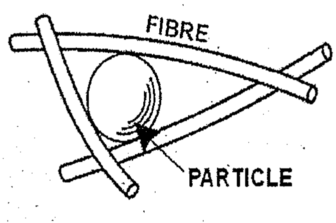
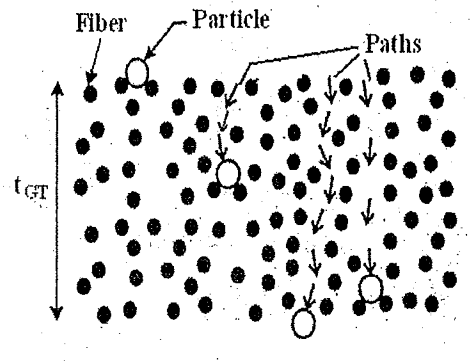
– Filter Characteristics

- Constriction size vs filtration opening size

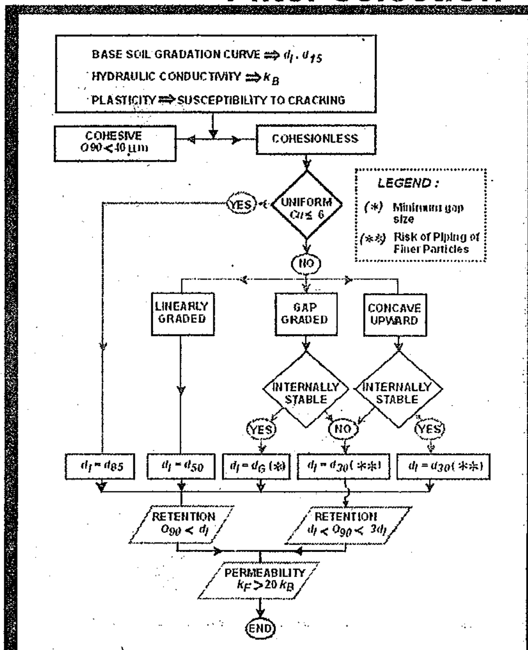
PLANAR CONSTRICTION



FILTRATION OPENING SIZE O_F =
SIZE OF THE LARGEST CONSTRICTION
OF THE LARGEST CONTINUOUS PATH



– Filter selection



Filter Selection

- By index tests on base soils (Lafleur, 1999)

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– Filter selection

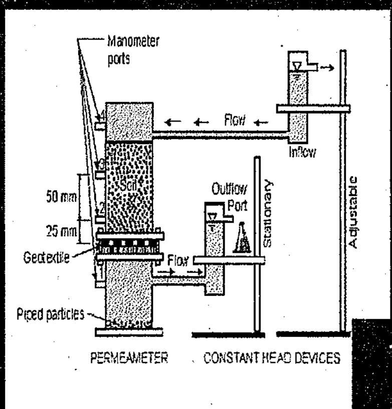
- By performance test (Fannin, et al., 1994)

Gradient Ratio

$$GR = \frac{(h_2 - h_1) / 25}{(h_3 - h_2) / 50}$$

Mass of Piped Particles

$$M_p = \frac{\text{Mass}}{\text{Sample Area}}$$



- Gradient Ratio

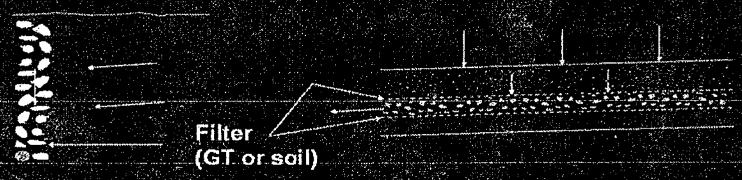
  GR < 1

- Mass of Piped Particles

  M_p < 2500 g/m²

Application 2 – Filtration/Drainage applications – 2b: Geocomposite Drains

- Traditional drainage layers are made of graded granular material



- The granular filter prevents penetration of fines into the drain (Filtration)
- The granular core transports the water in the plain of the filter (Drainage)

Application 2b - Geo-composite Drains

Applications For Geocomposite Drainage Materials

- **Pavement edge drains**
- **Interceptor trenches on slopes**
- **Drainage behind abutments and retaining structures**
- **Relief of water pressure on buried structures**
- **Substitute for conventional drains**
- **Leachate collection and gas venting**
- **Drainage mats in horizontal applications e.g. roofing**

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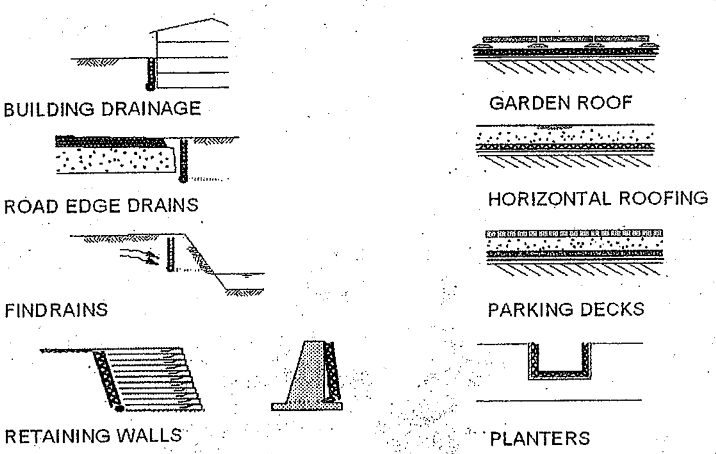
Application 2b - Geo-composite Drains

Cross-Section of Typical Application (1)



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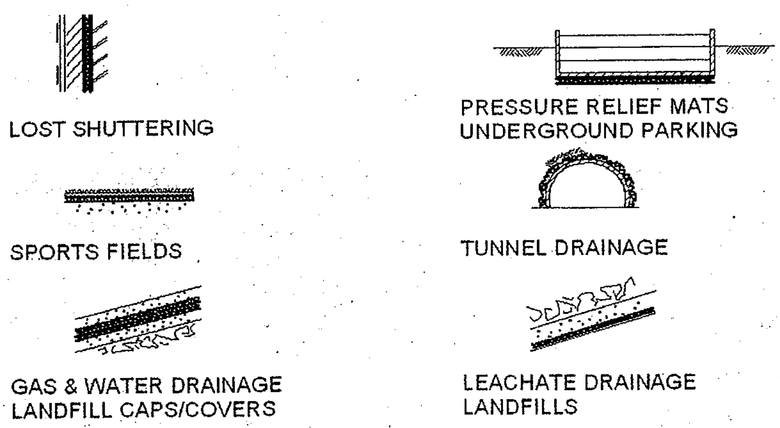
Application 2b - Geo-composite Drains

Cross-Section of Typical Application (2)



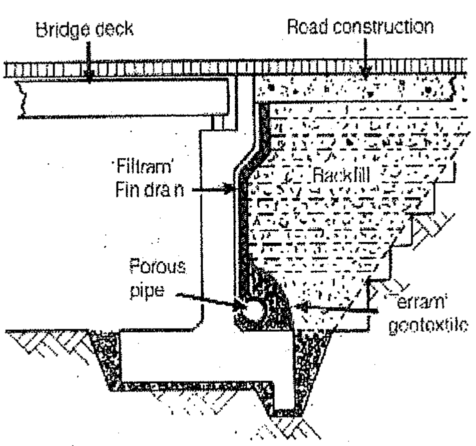


Figure 46a: Use of Fin drains against bridge abutments

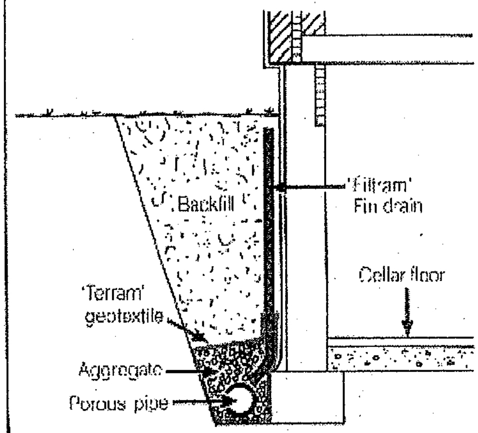
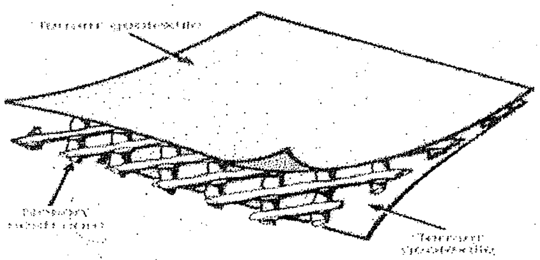


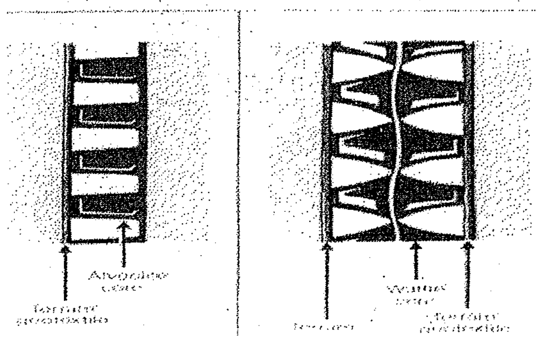
Figure 46b: Use of Fin drains against basement walls

Figure 46: Use of Fin drains against bridge abutments and basement walls

Application 2b - Geo-composite Drains



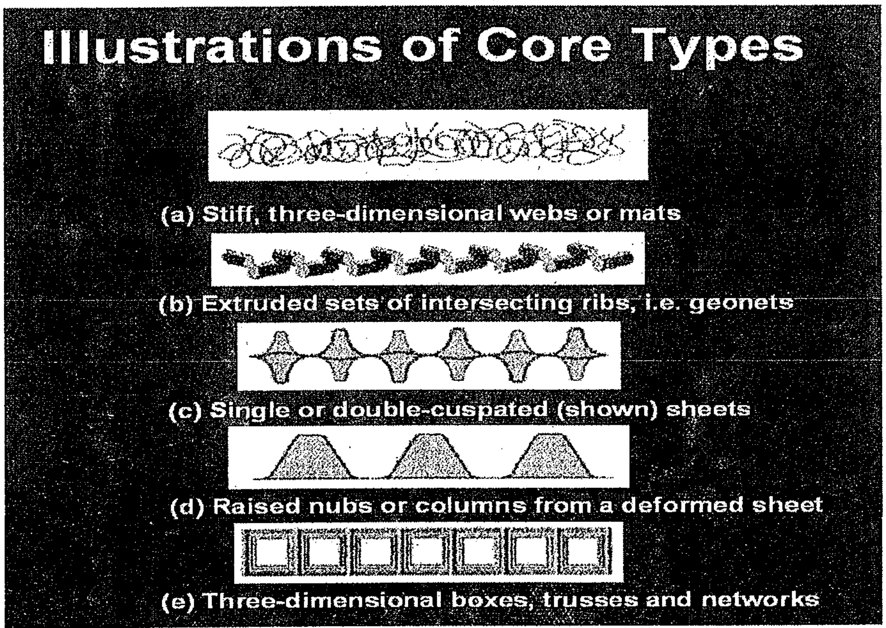
- One or two nonwoven geotextile layers
- High discharge capacity core 5 - 25 mm thick
- Discharge capacity in plane 0.0002 - 0.01m^3/m width/sec



Geocomposite Drains : Types of core

- Extruded or fluted plastic sheets
- 3 dimensional meshes or mats
- Plastic waffles
- Nets and channels to convey water

Application 2b - Geo-composite Drains



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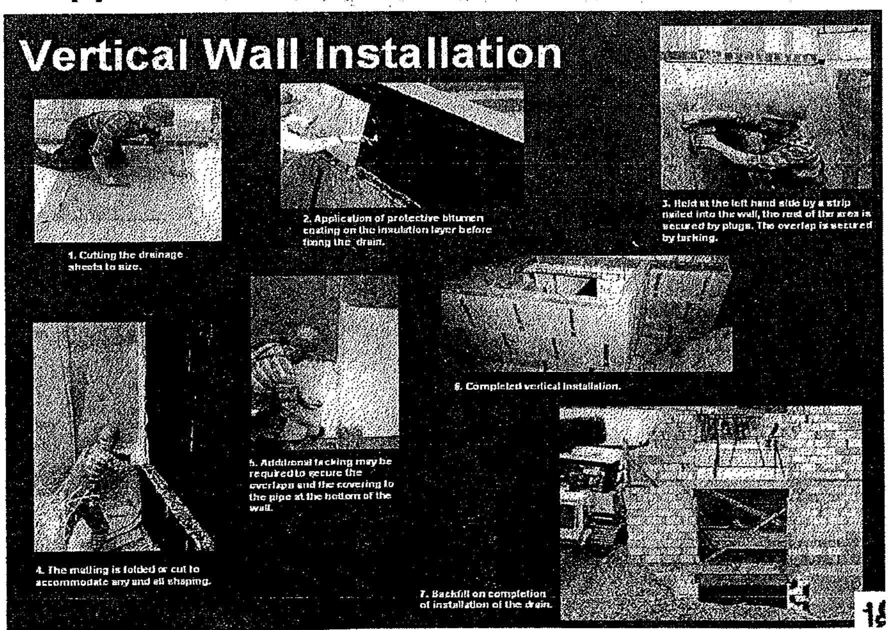
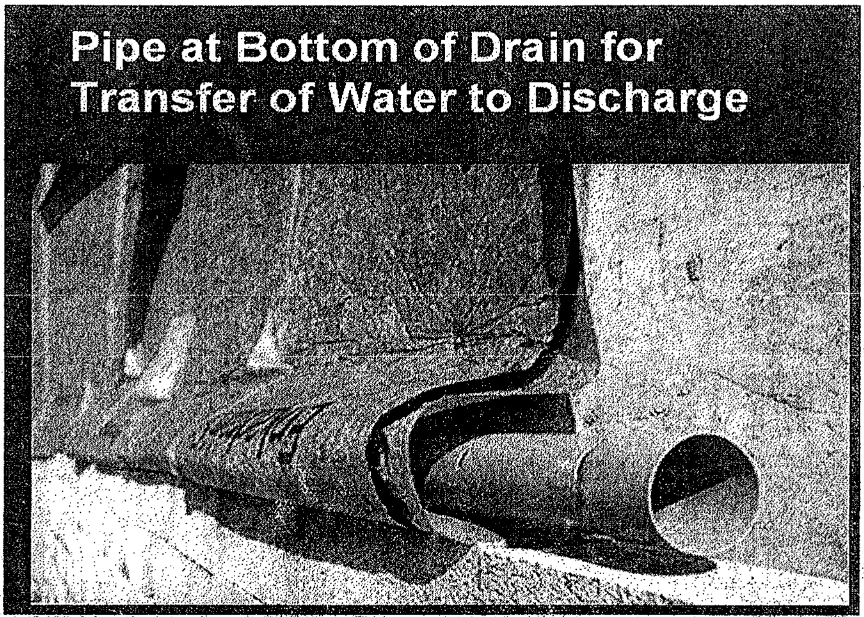
Application 2b - Geo-composite Drains



Application 2b - Geo-composite Drains



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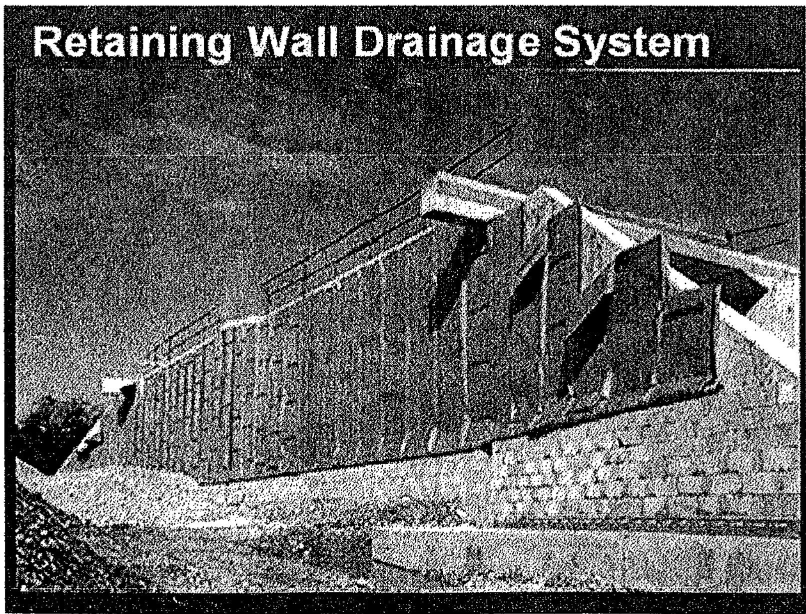
Application 2b - Geo-composite Drains



Application 3- Landfill

Waste Containment Liners with Geotextiles

Geotextiles are used in containment applications to fulfil various functions such as:

- separation
- filtration
- protection
- drainage
- reinforcement

Other functions such as sealing are carried out by geomembranes and (GCLs).

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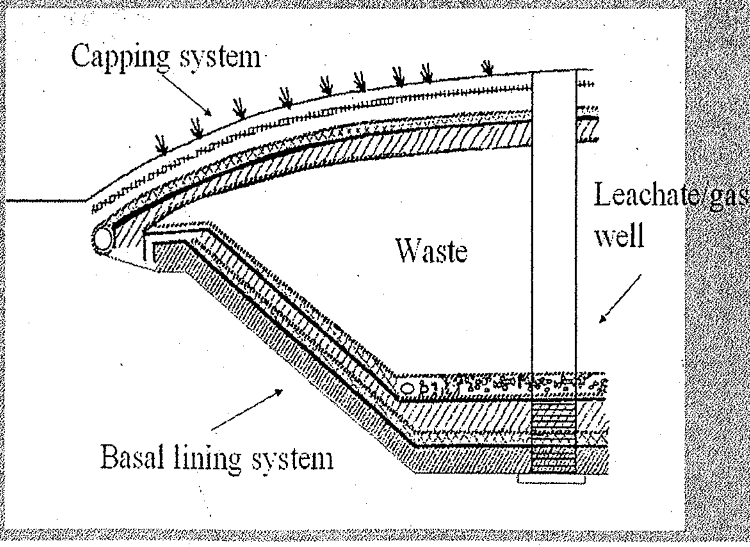
Application 3- Landfill

Schematic of a Landfill Showing the Main Components



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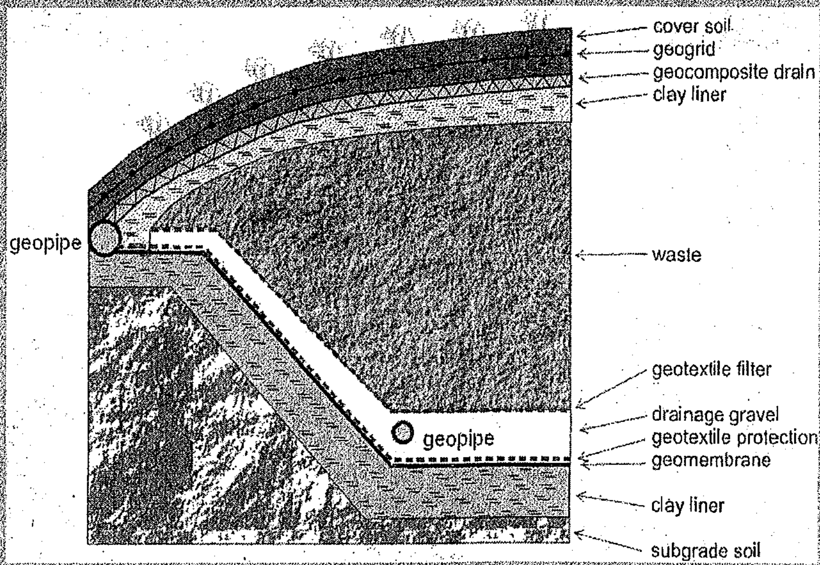
Application 3 - Landfill

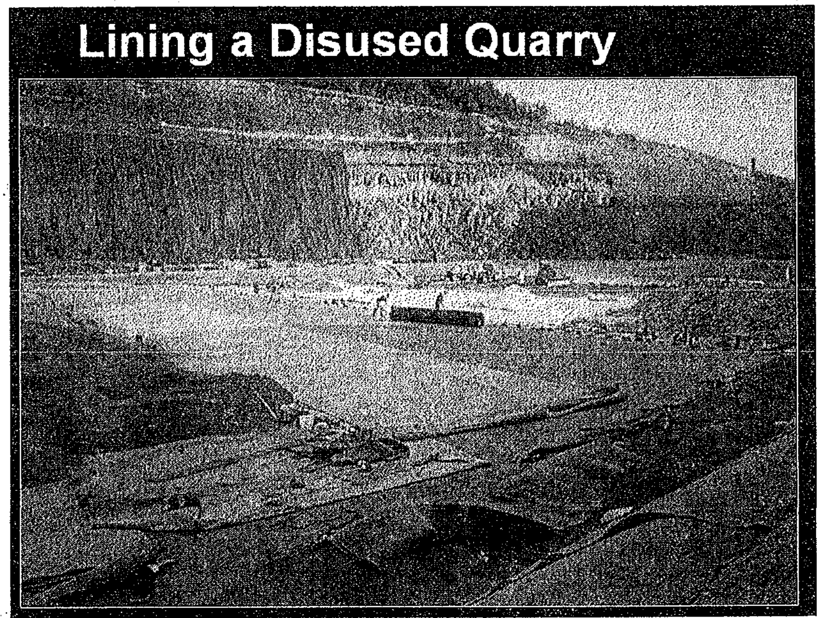
Use of geosynthetics in landfills



Application 3- Landfill

Landfills During Construction: Shallow Slopes

Application 3- Landfill



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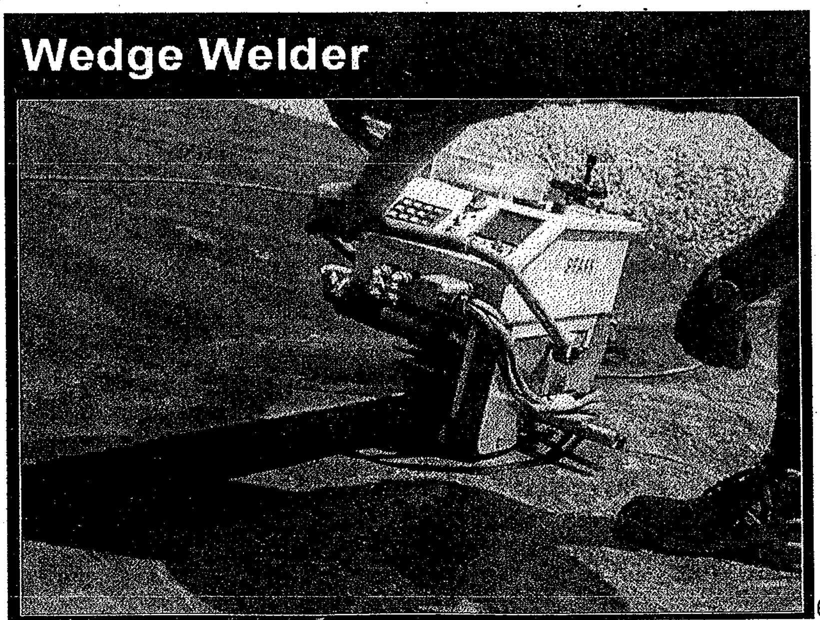
Application 3- Landfill



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Application 3- Landfill

Defect in Geomembrane

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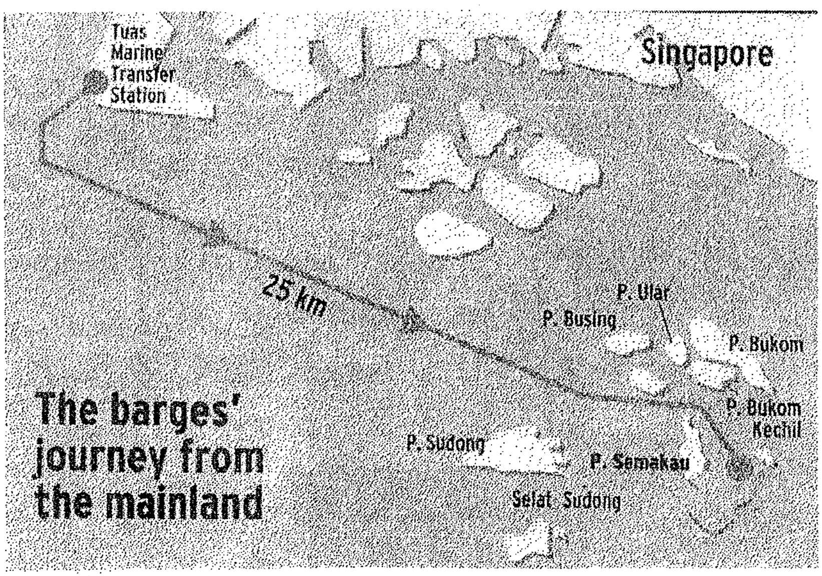
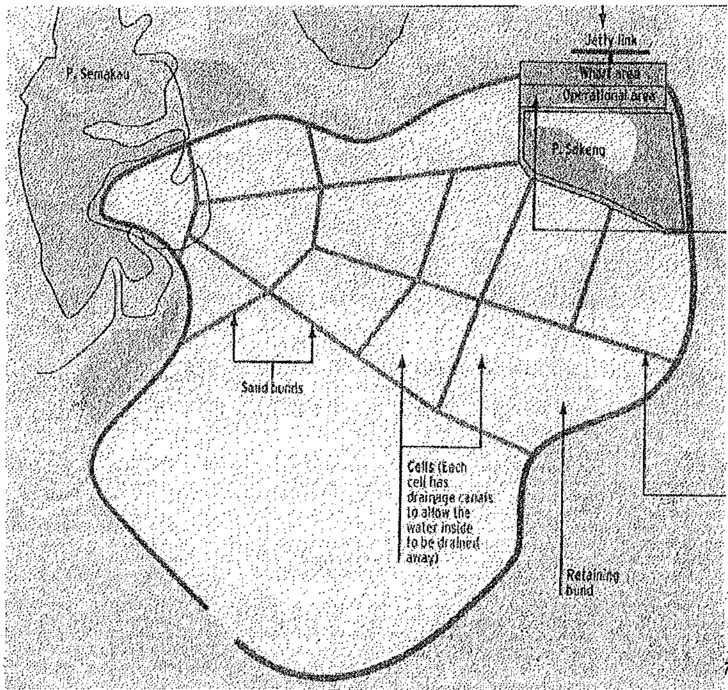
Offshore Landfill in Singapore





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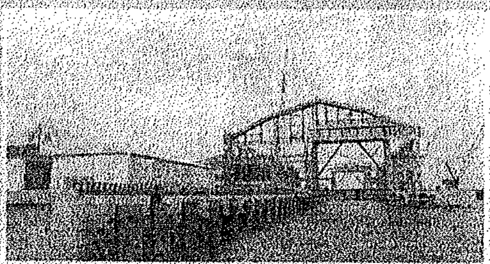
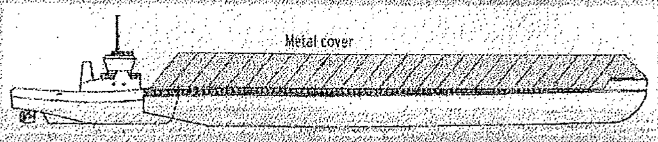
WHERE DOES THE RUBBISH GO?

1 The refuse is loaded onto two barges from the Tuas Marine Transfer Station during the day.



The unfinished jetty and wharf area at Pulau Sakeng.

2 The barges are pushed by tugboat to the jetty at Pulau Sakeng, a three-hour journey, and arrive there at night. Two empty barges waiting at the jetty are pushed back to Tuas.



Artist's rendition of a typical barge to be used.

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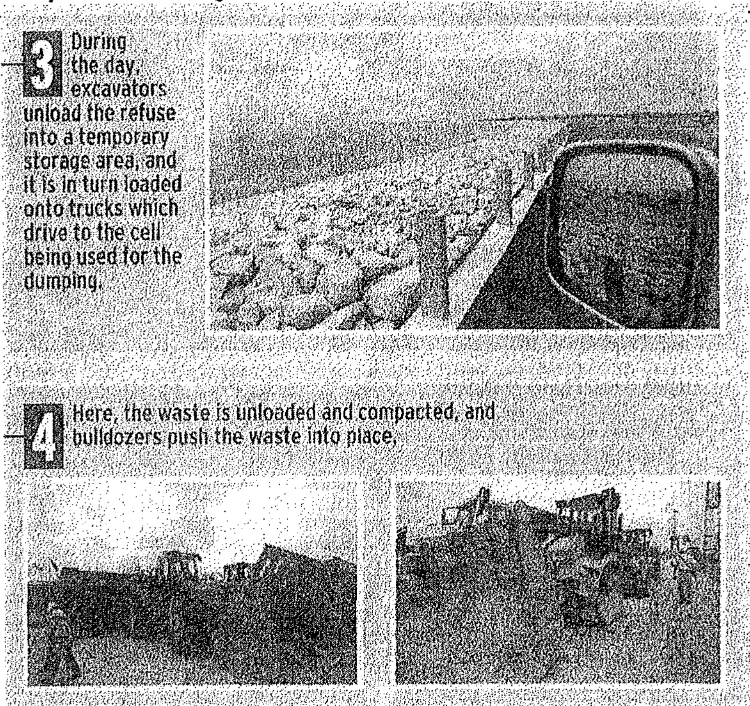

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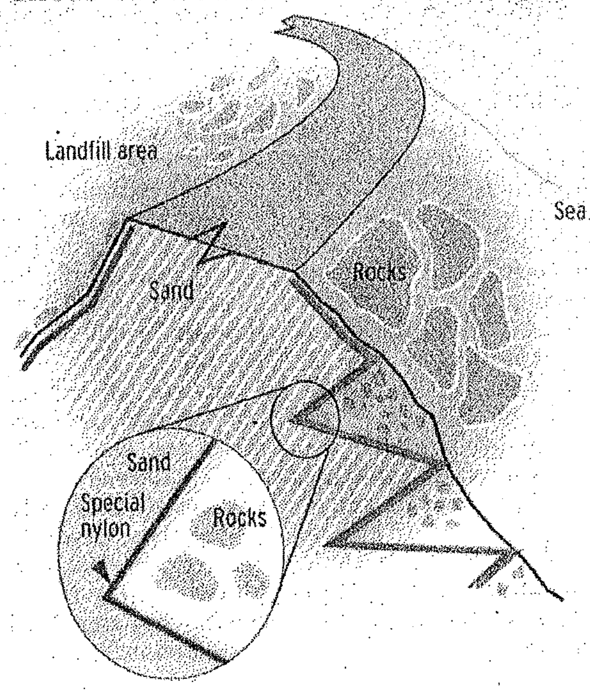
CROSS-SECTION OF THE RETAINING BUND



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Largest Offshore Landfill ..

- S$840 million landfill
- Operational in April 1999
- Holds all Singapore's domestic refuse
- Lasts beyond 2030

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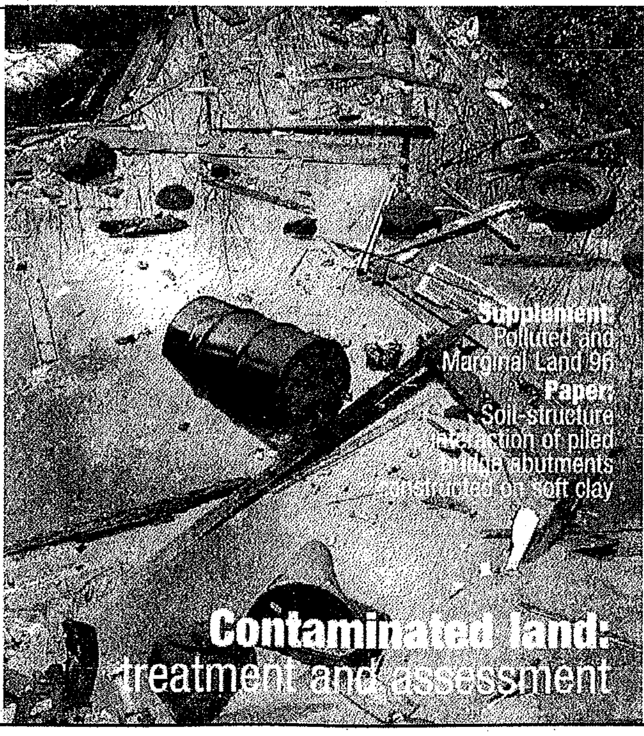

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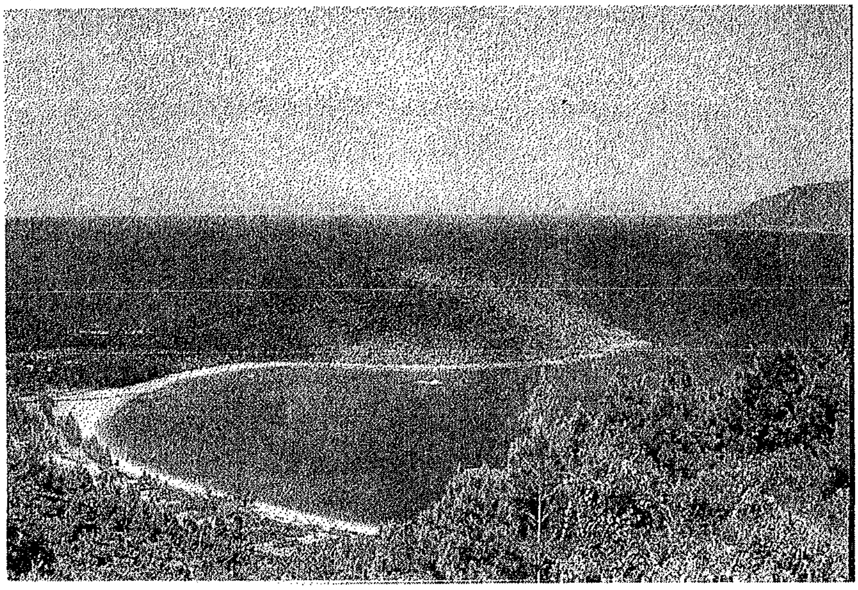
Semakau Landfill after 2030 ++



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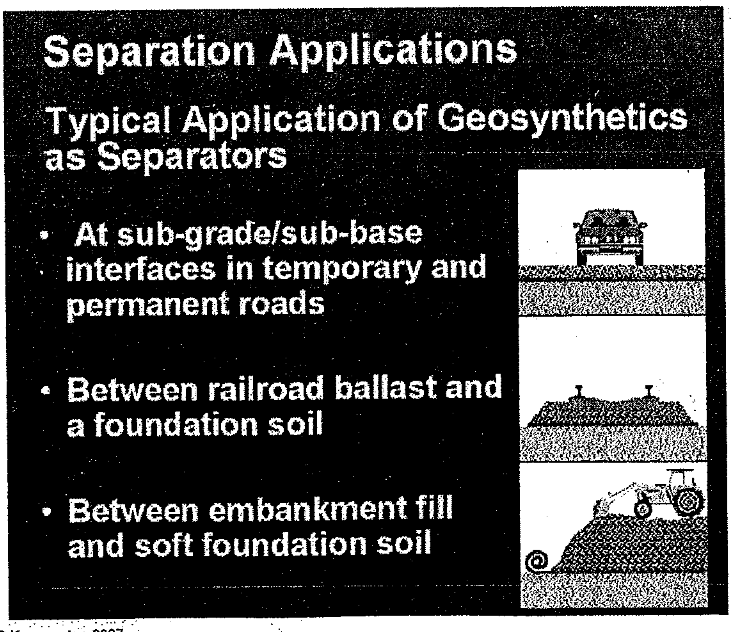
Application 4 – Applications and Roads & Railroads



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Application 4 – Applications and Roads & Railroads

Separation Geotextile Being Laid and Covered

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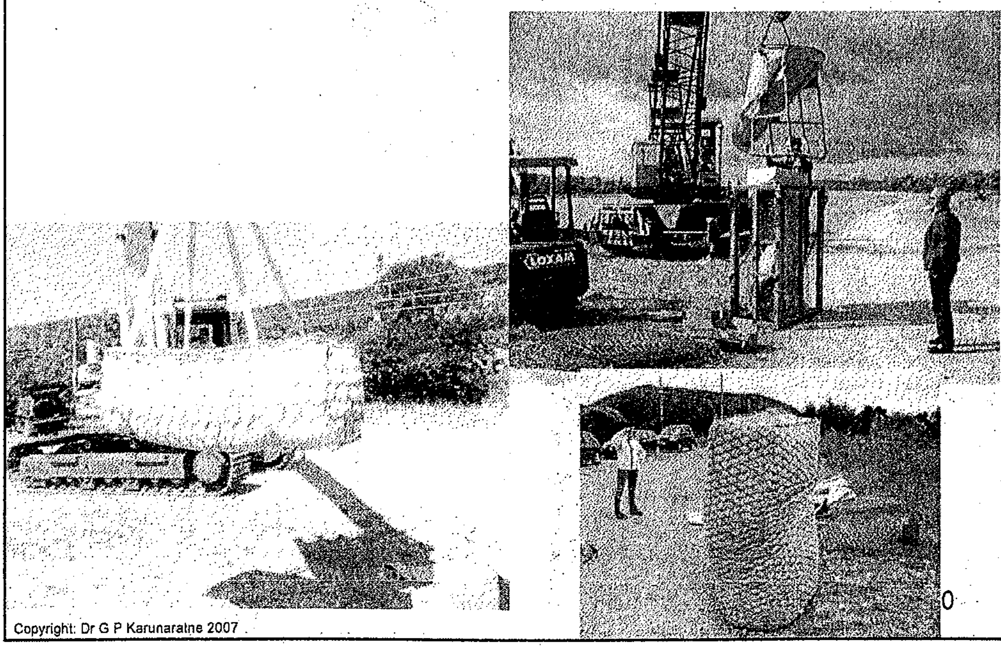
Application 5- **Geobag in Coastal application**



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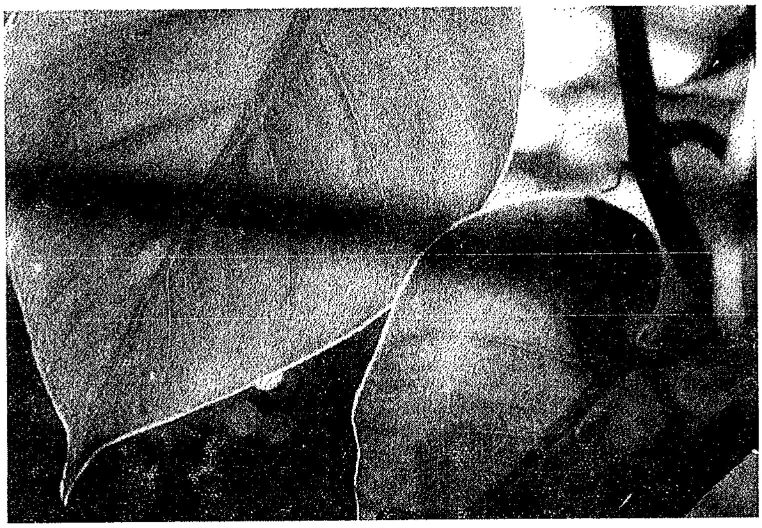
To be continued



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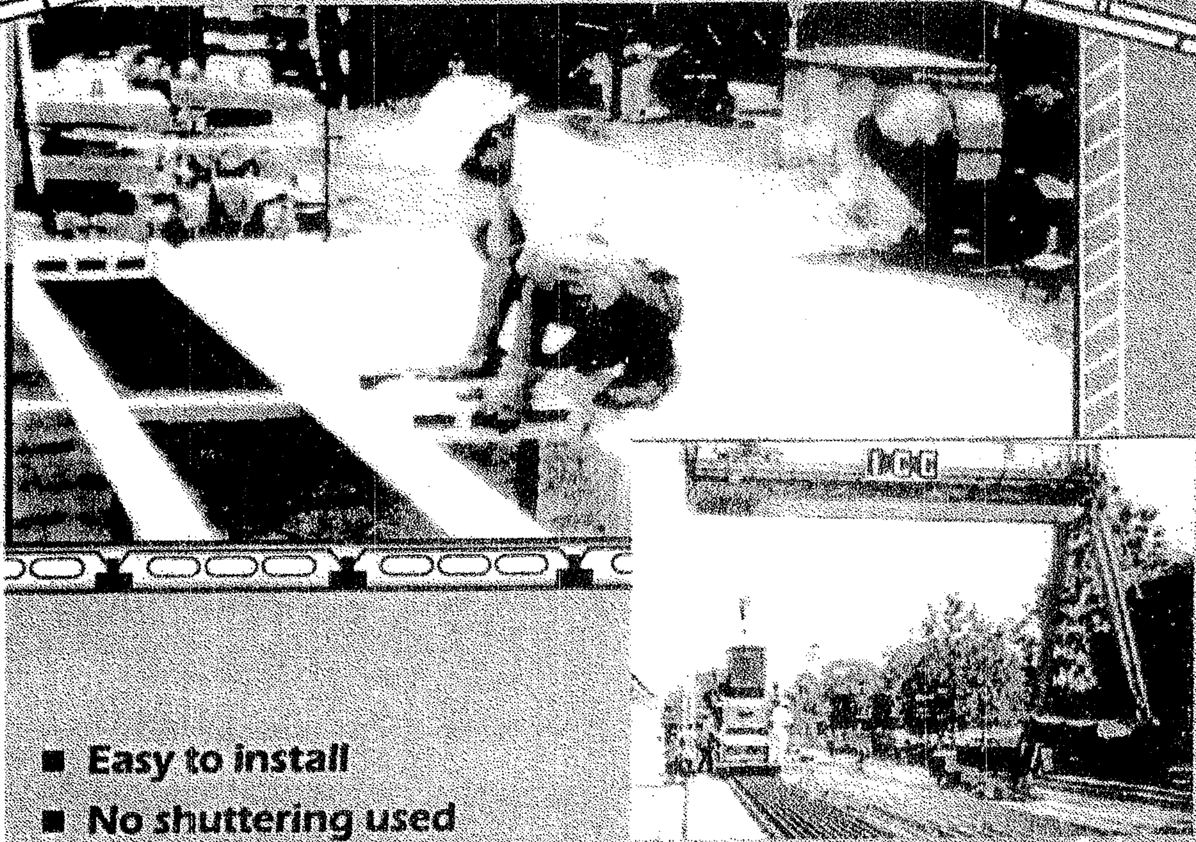

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Geotextiles for Road Pavement Rehabilitation

Michael Chong (BE)

Sale Manager – TENCATE Geosynthetics Asia Sdn Bhd

BIODATA OF Mr. Michael Chong

Michael Chong is currently the Sales Manager of TENCATE Geosynthetics Asia Sdn Bhd (Formerly known as Polyfelt Asia Sdn Bhd). He is responsible for marketing and technical support to various countries in Asia.

Academic Background

- **Bachelor of Civil Engineering, 1998, University of Science Malaysia**

PROFESSIONAL EXPERIENCE

TENCATE Geosythetics Asia Sdn Bhd 2002 – present as Sales Manager and responsible for marketing and technical support to Hong Kong, Taiwan, Philippines, Vietnam, Indonesia, Malaysia, Bangladesh and Sri Lanka. In addition, he is also the product manager for Geosynthetics Clay Liner.

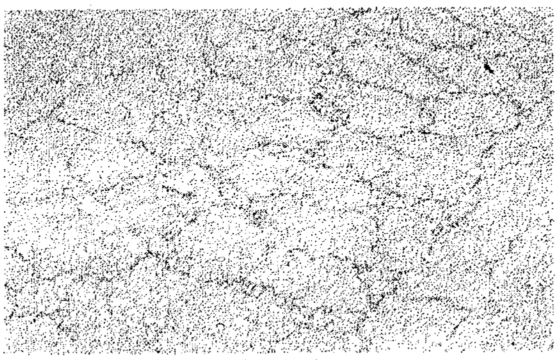
Michael conducts various technical trainings and presentations on Geosynthetics to Public Works Department, University and Professional Institutions in Asia.

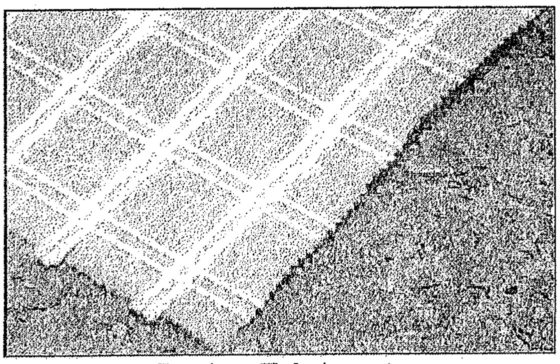
Professional membership

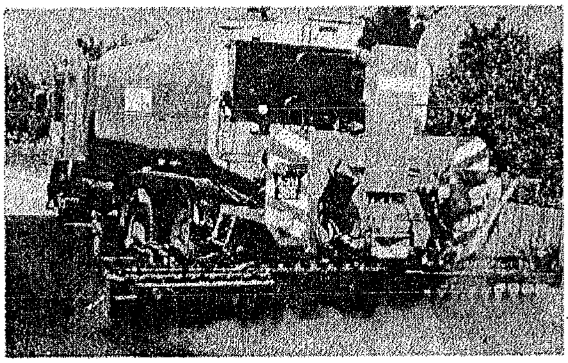
- Member of International Geosythetics Society
- Graduate Member of Institution of Engineers Malaysia
- Member of Board of Engineers Malaysia.

The application of paving fabrics in areas such as road and runway rehabilitation can provide significant economic benefits to end users. Paving fabric is a proven product which, prolong the life span of roads and runways. The maintenance cost will be reduced substantially.

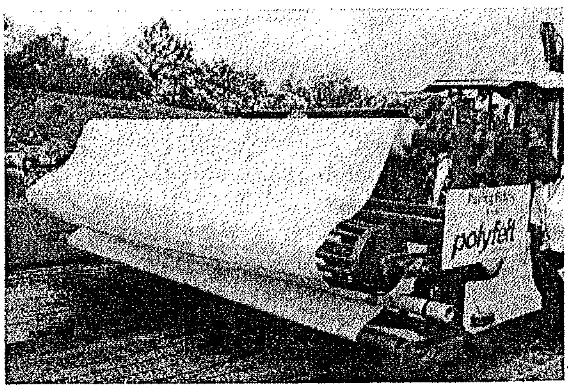
Basic theory as well as laboratory testing on paving fabrics will be presented. The success of paving fabrics applications lies on the installation know-how. The installation methodology as well as other important information such as bitumen type, spray rate and weather will be highlighted. Various project references in Asia will be presented in the seminar.


Road cracking

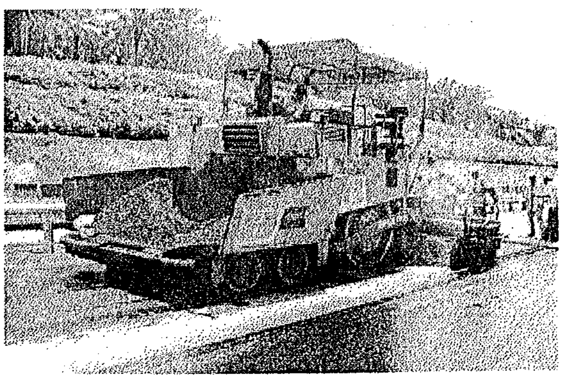

Paving Fabric

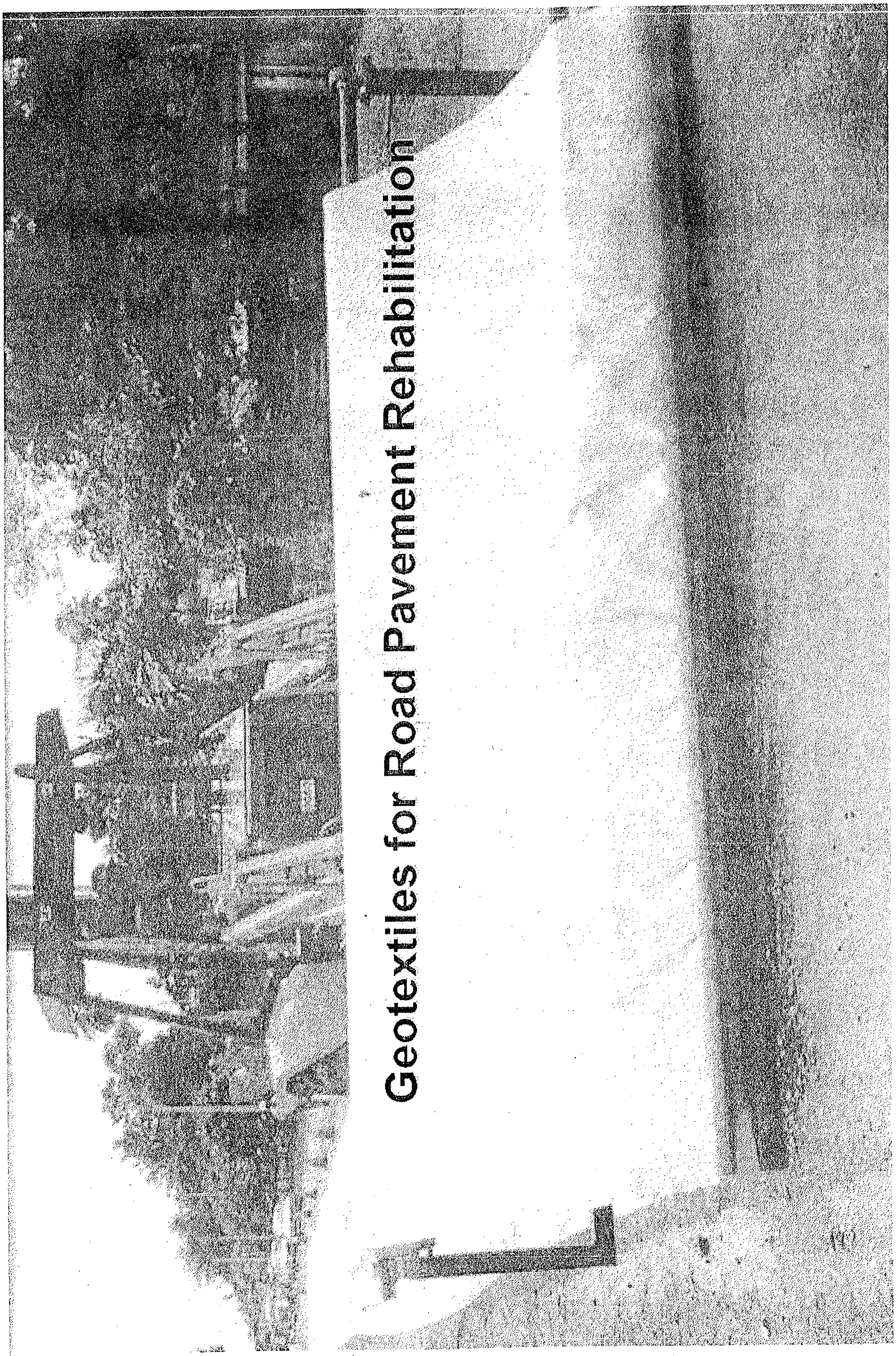

Bitumen Spray Tanking


Manual installation



Paving rig


Asphalt paver


Geotextiles for Road Pavement Rehabilitation

Introduction

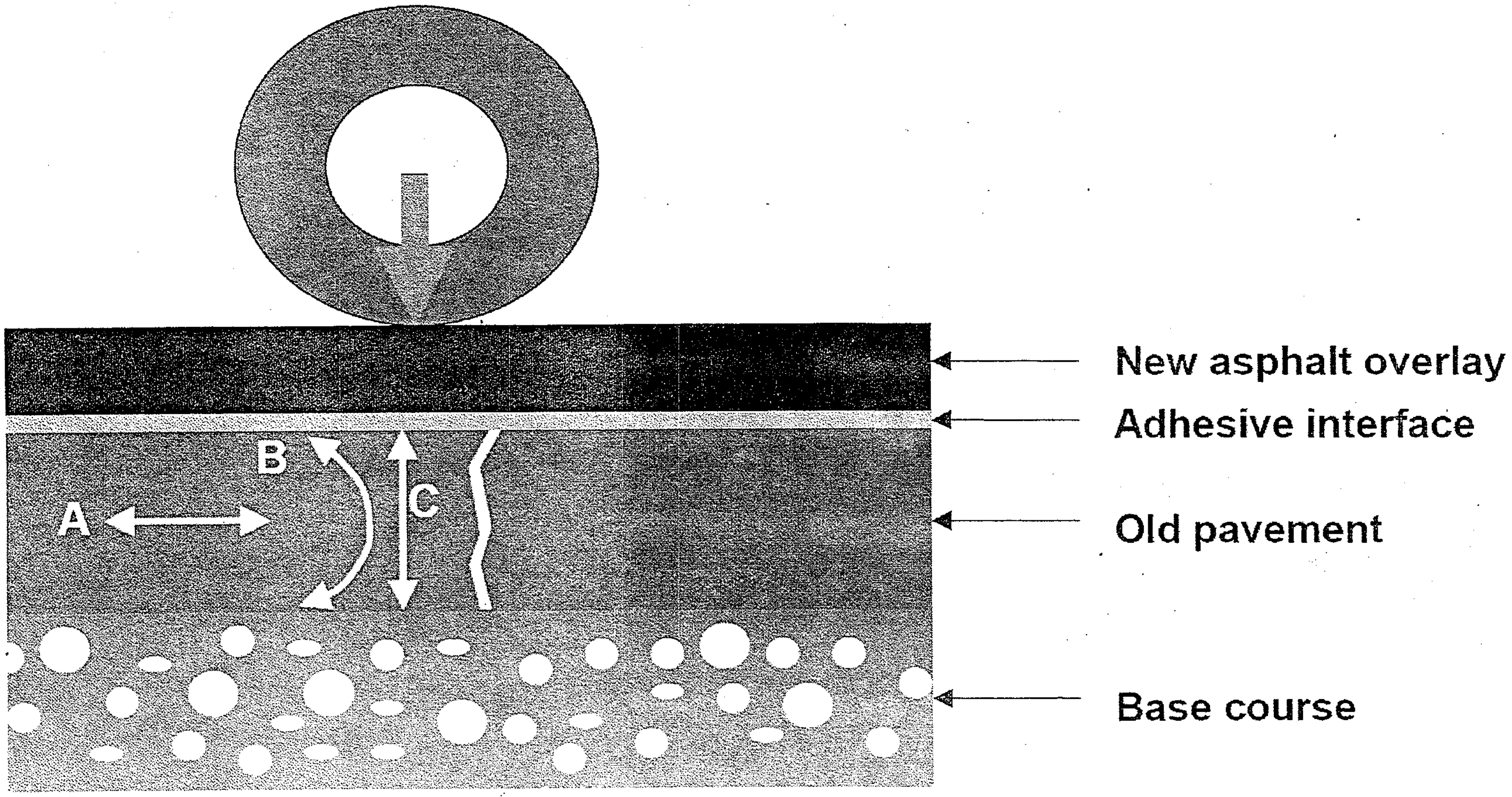
Non-woven fabrics manufactured from continuous polypropylene fibers are widely utilized in the maintenance and rehabilitation of asphalt road pavements.

Paving Fabrics alone do not prevent cracking. They perform their function as part of a fabric/bitumen system. The quality and quantity of bitumen directly influences the performance of the system.

Correctly installed, Paving fabrics offer the following benefits:

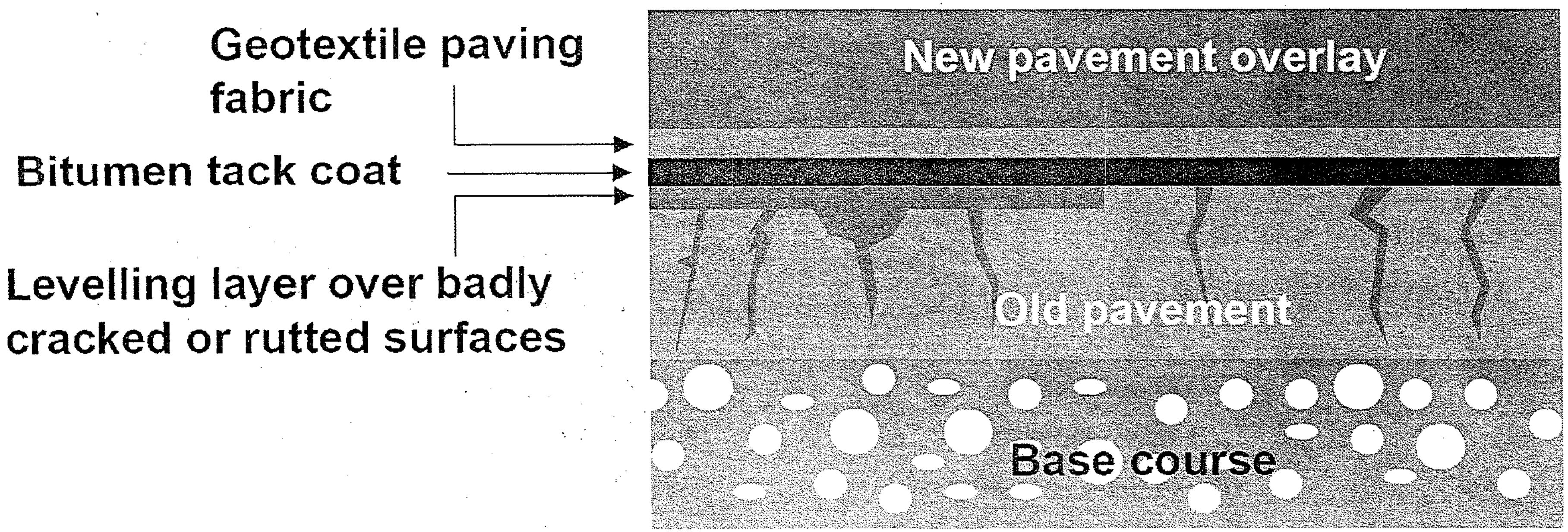
- Delay the reflection of stress cracks through the overlay, extending the life of the overlay.
- Provides for a more uniform adhesion of the overlay to the old asphalt surface.
- Provides a flexible waterproof layer between the old road surface and new overlay, preventing water migration into the road pavement.
- Eliminates problems associated with frost heave and freezing of water within the pavement structure.

Problems associated with new asphalt overlay without geotextile paving fabric



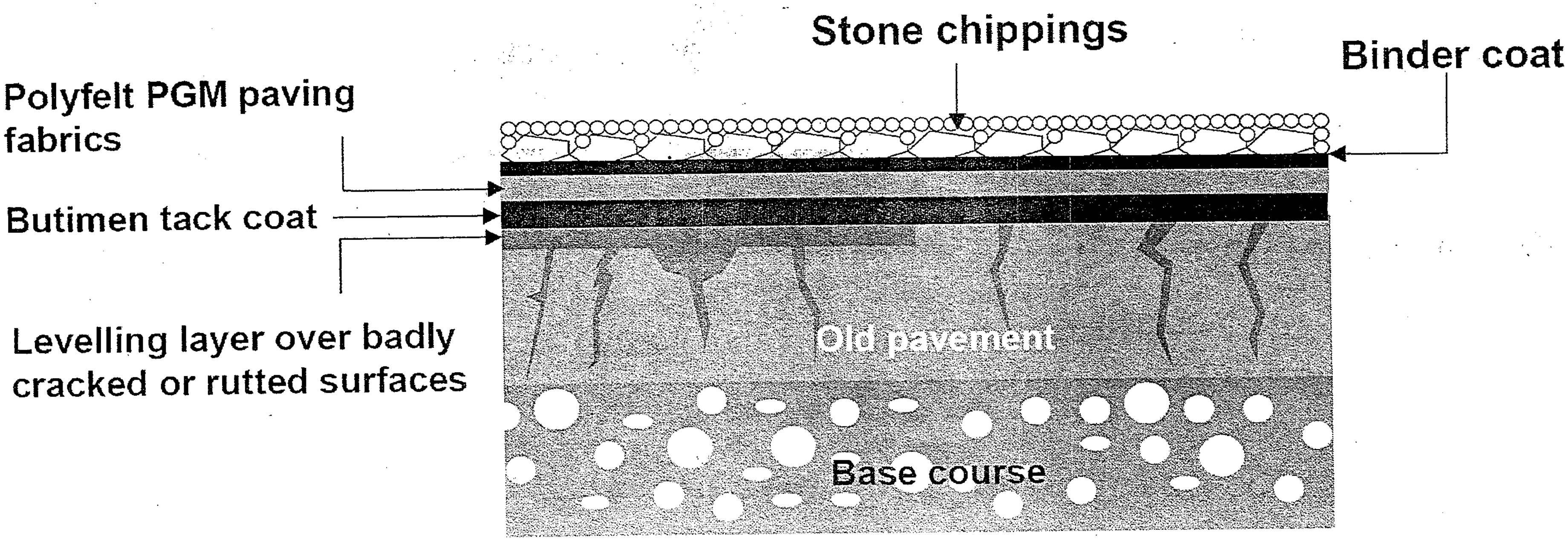
A. Thermal movements
B. Bending (deflection)
C. Differential vertical movements

Improved new pavement overlay with geotextile paving fabric during rehabilitation



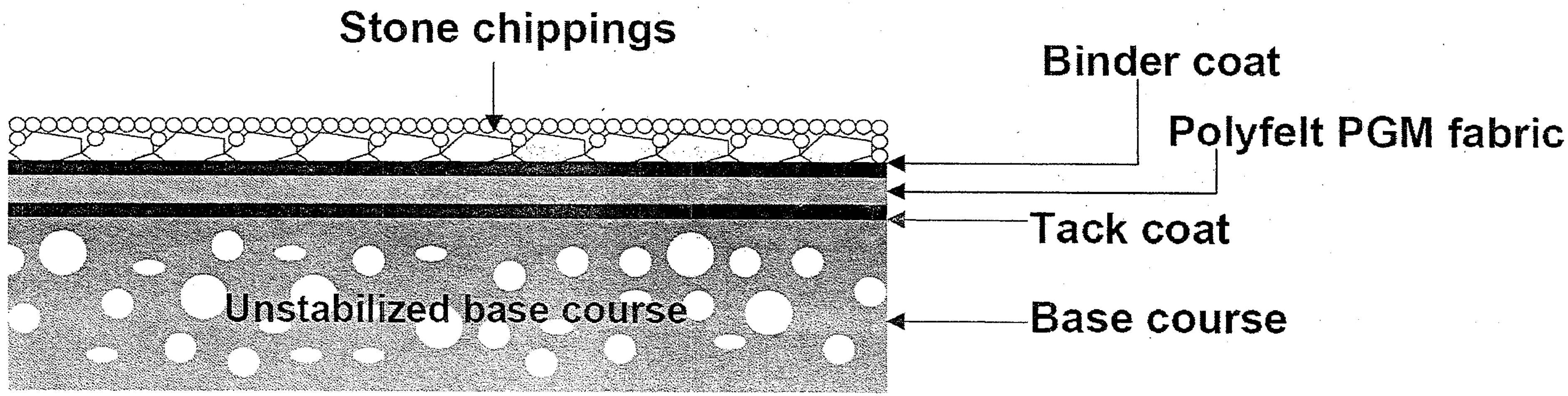
Polyfelt PGM Areas of Application

Between stone-chip surface dressing and old cracked road pavements, with or without levelling layer



Polyfelt PGM Areas of Application

Under stone-chip surface dressing over unstabilized base courses

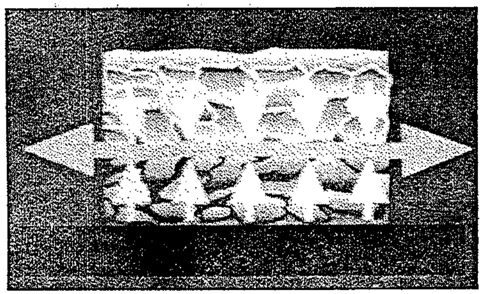


Primary Functions of Polyfelt PGM / PGM-G Paving Fabrics

Polyfelt PGM 14: Needle punched polypropylene continuous filament non-woven fabric with optimum asphalt retention properties

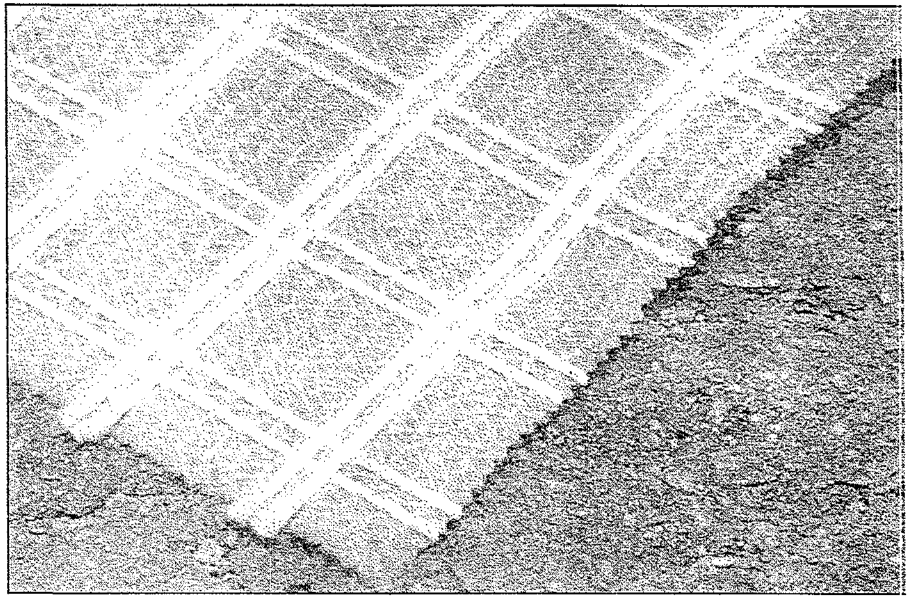
Polyfelt PGM-G: Fiberglass reinforced polypropylene fabric for reinforcement and sealing

Function(s):



Reinforcement

Sealing



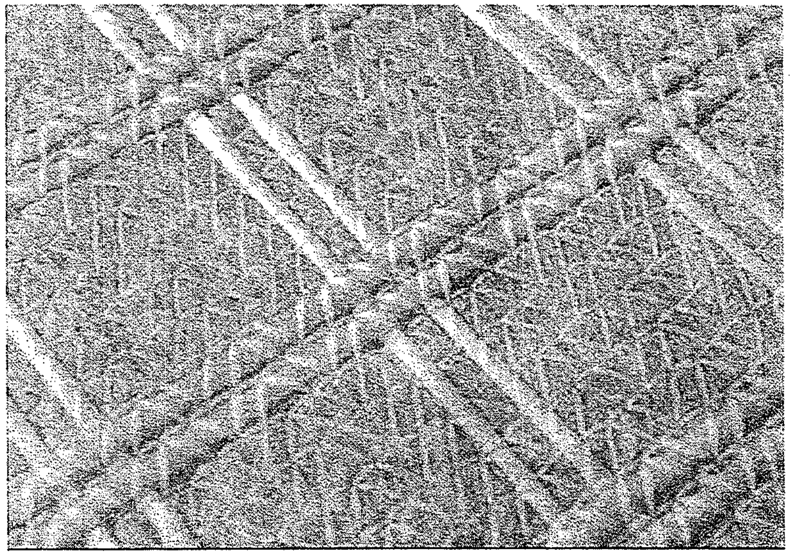
Properties of Polyfelt Paving Fabrics

Polyfelt PGM 14

Polymer	Polypropylene
Asphalt retention	1.1 l/m^2
Melting point	165 °C
Mass/area	140 g/m^2

Polyfelt PGM G50/50

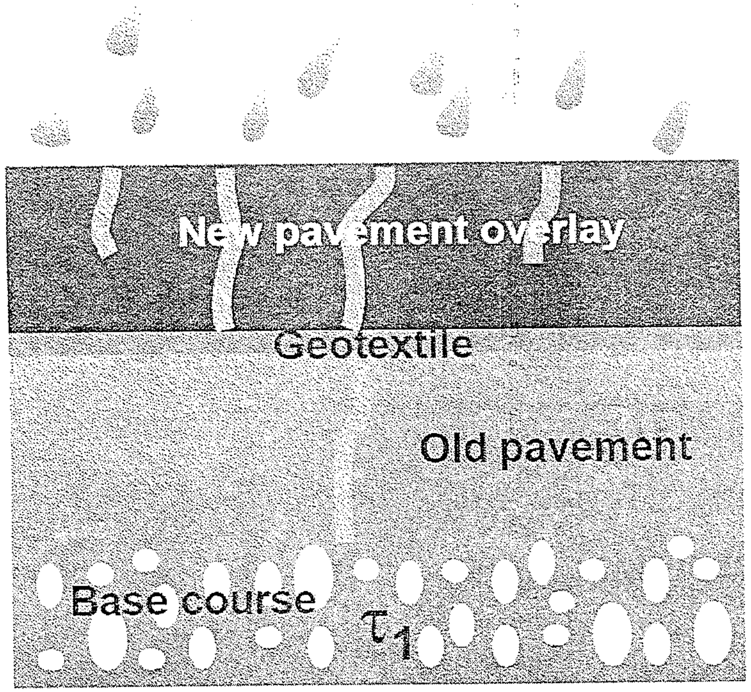
Polymer	Polypropylene glass fibre
Asphalt retention	1.1 l/m^2
Melting point	165 °C
Mass/area	140 g/m^2
Tensile strength (md/cd)	50 kN/m
Elongation (md/cd)	3 %



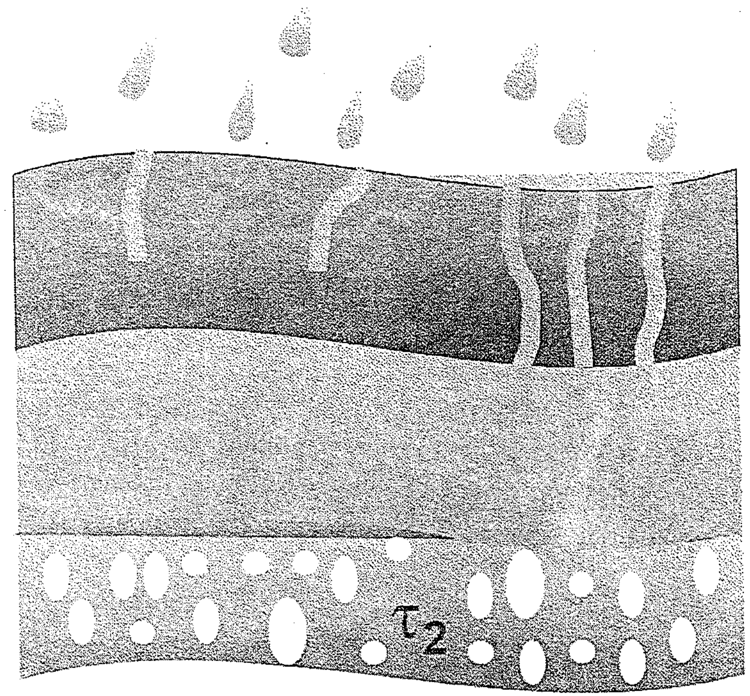
Functional Performance of Geotextile Paving Fabric

1. Sealing

With geotextile paving fabric



Without geotextile paving fabric

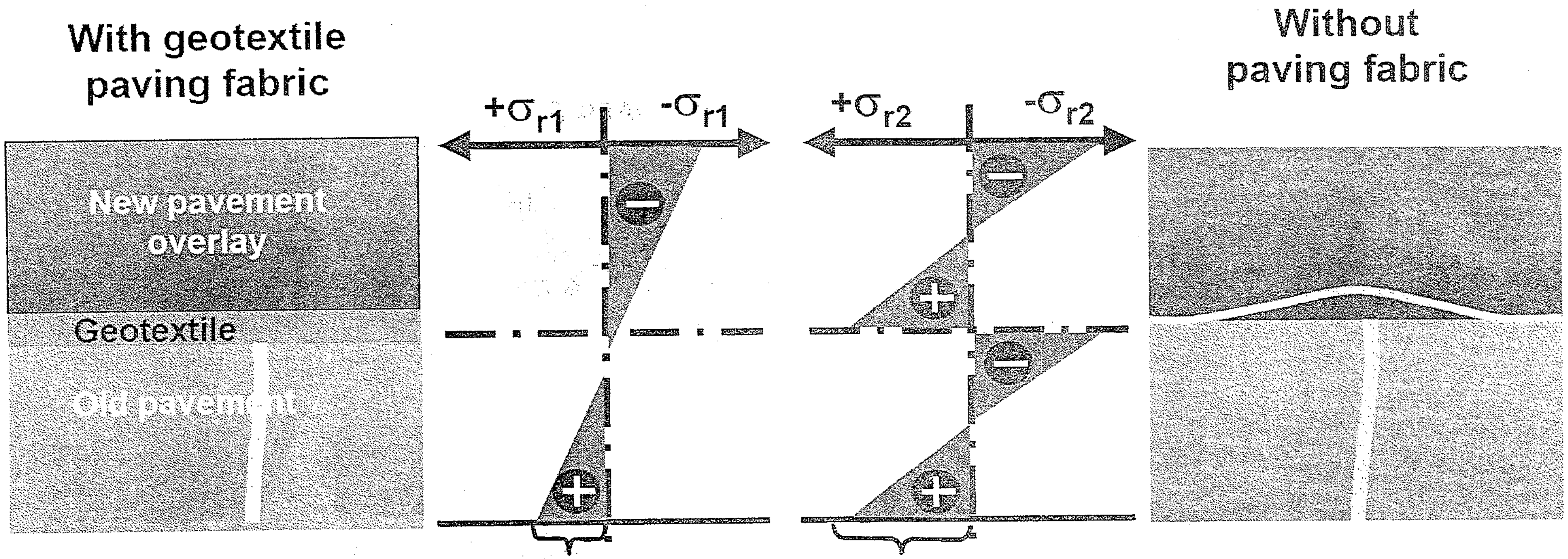


$\tau_1 > \tau_2$

Sealing underlying layers against penetration of surface water

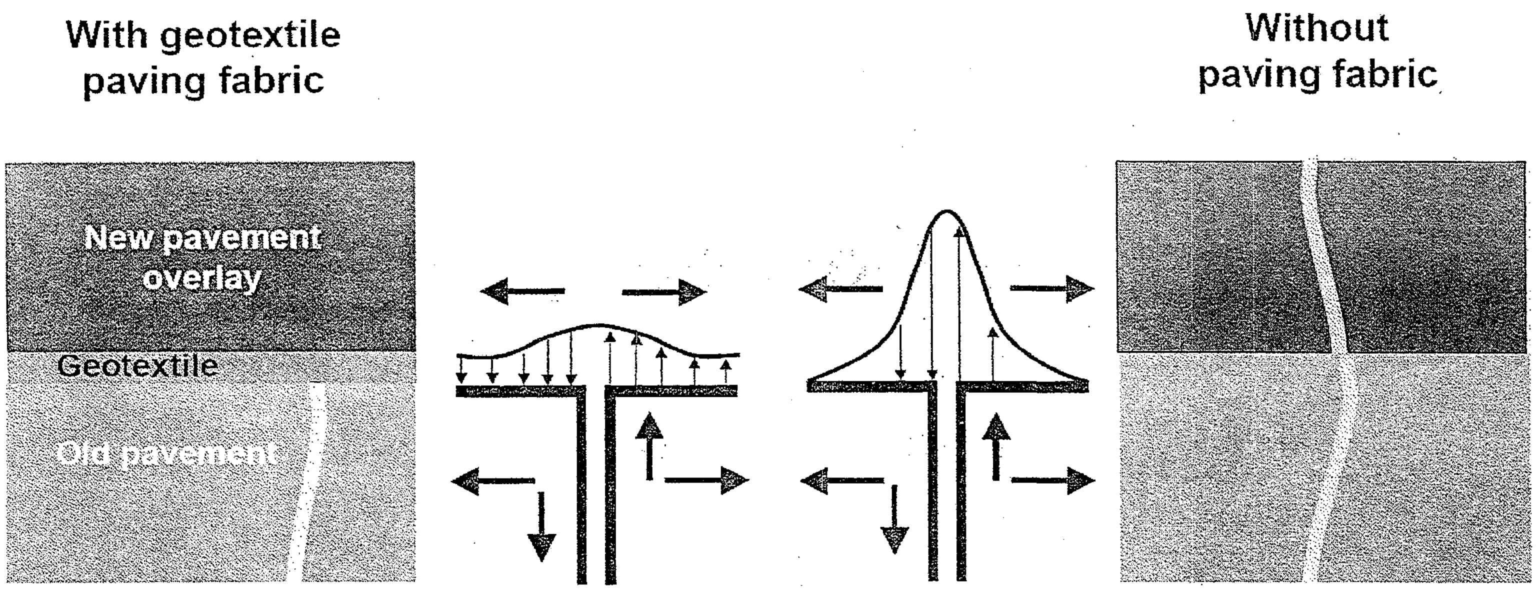
Functional Performance of Geotextile Paving Fabric

2. Bonding



Functional Performance of Geotextile Paving Fabric

3. Stress Reduction (reinforcement)

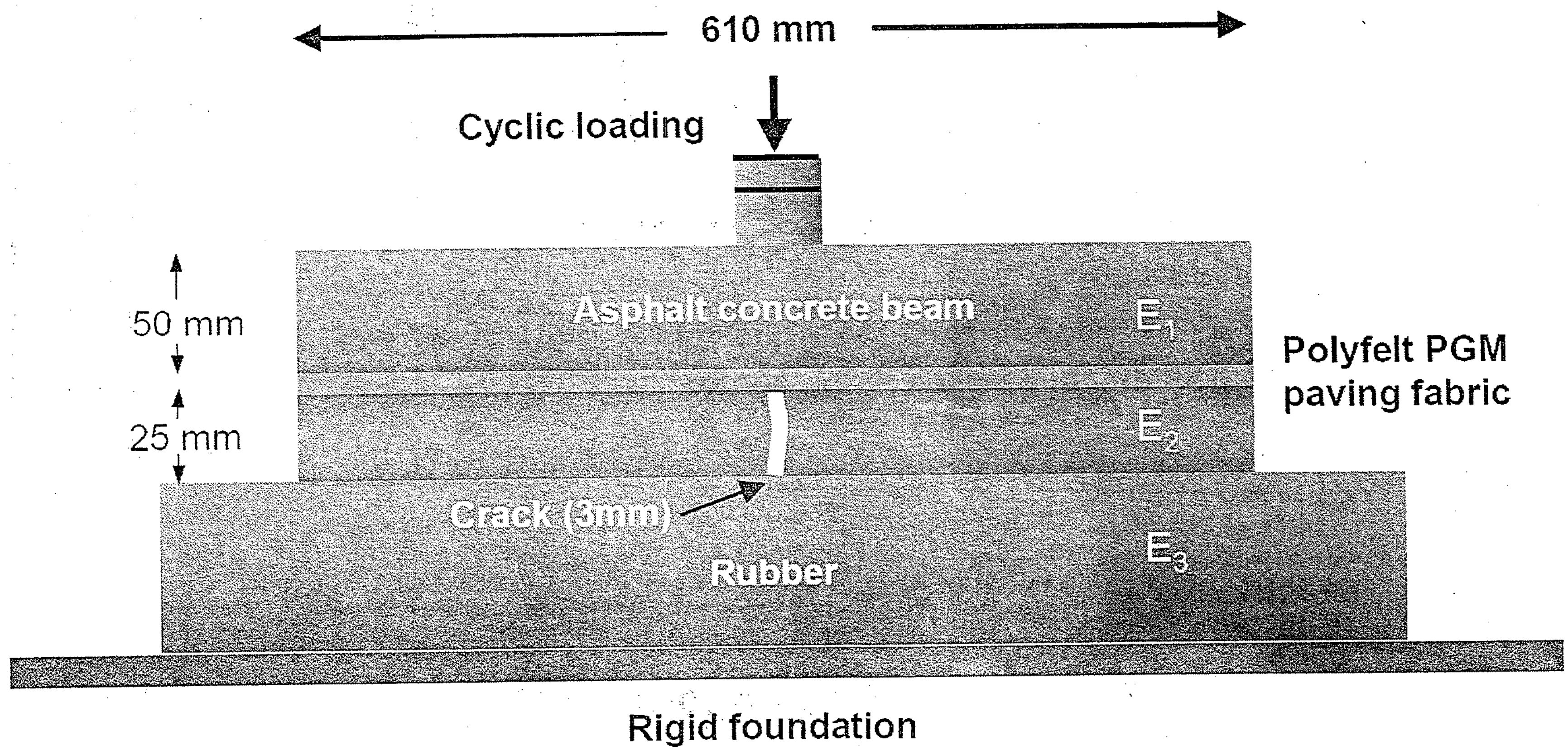


Research & Testing

Laboratory Testing

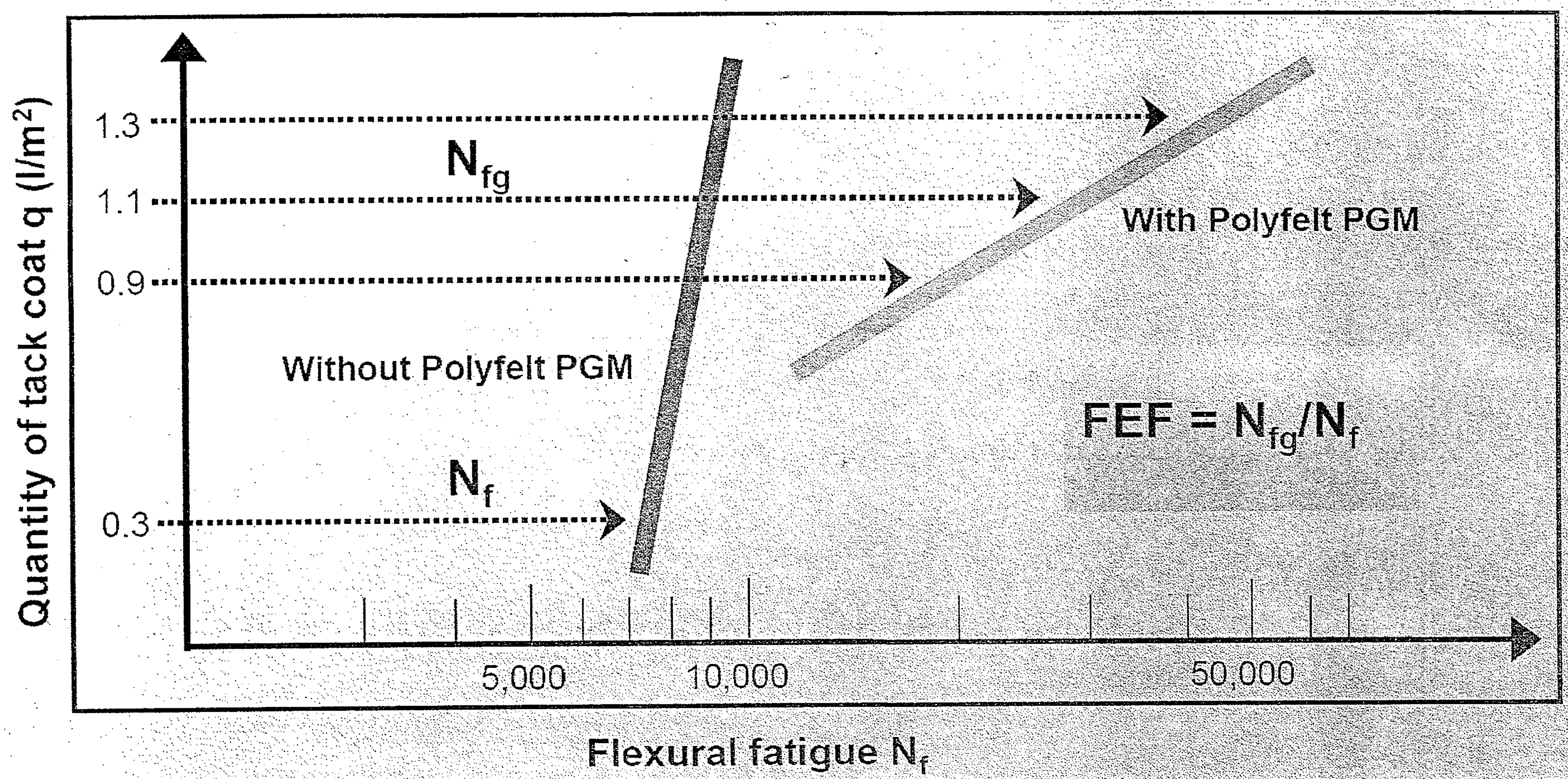
Research & Testing

Evaluation of the paving fabric effectiveness factor (FEF)



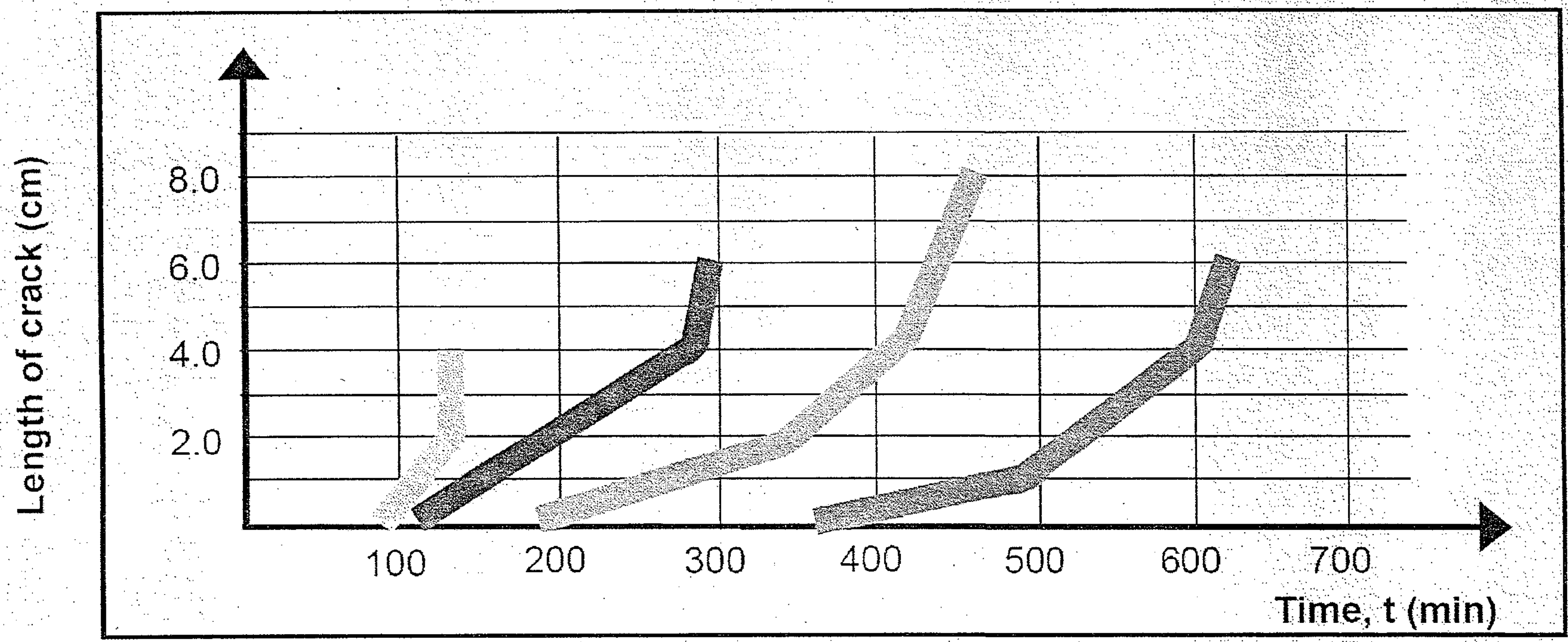
Research & Testing

Flexural fatigue N_f and FEF as a function of bitumen quantity



Research & Testing

Crack propogation in the asphalt overlay



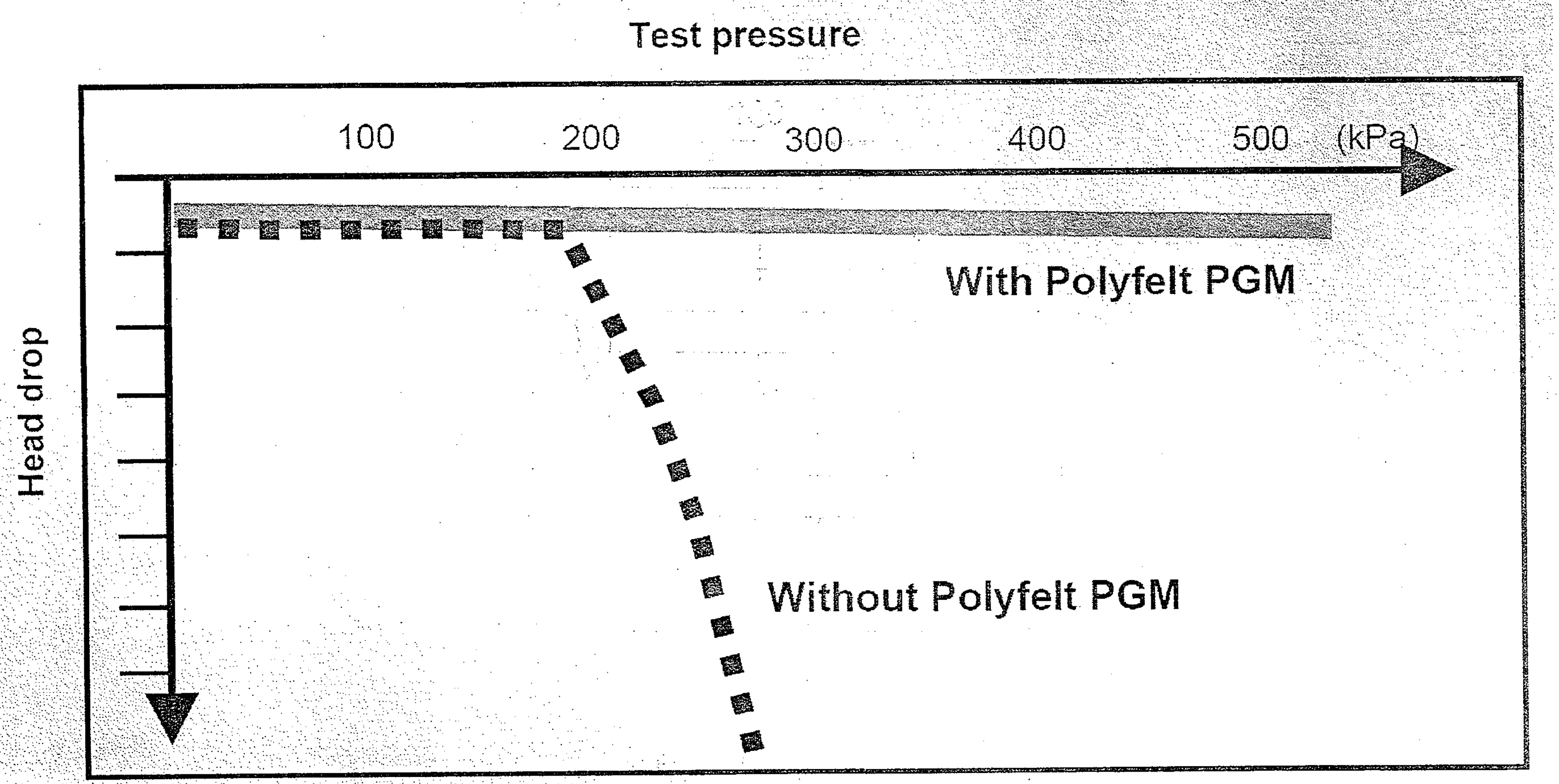
Flexural Tensile Test

4cm AC · 6cm AC 0/10 · 2cm bituminuous sand +6cm AC 0/10

Polyfelt PGM14 + 1.1 l/m² polymer modified bitumen + 6cm AC 0/10

Research & Testing

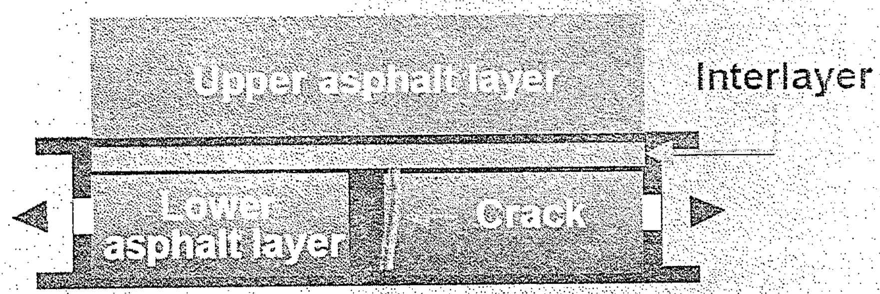
Sealing efficiency

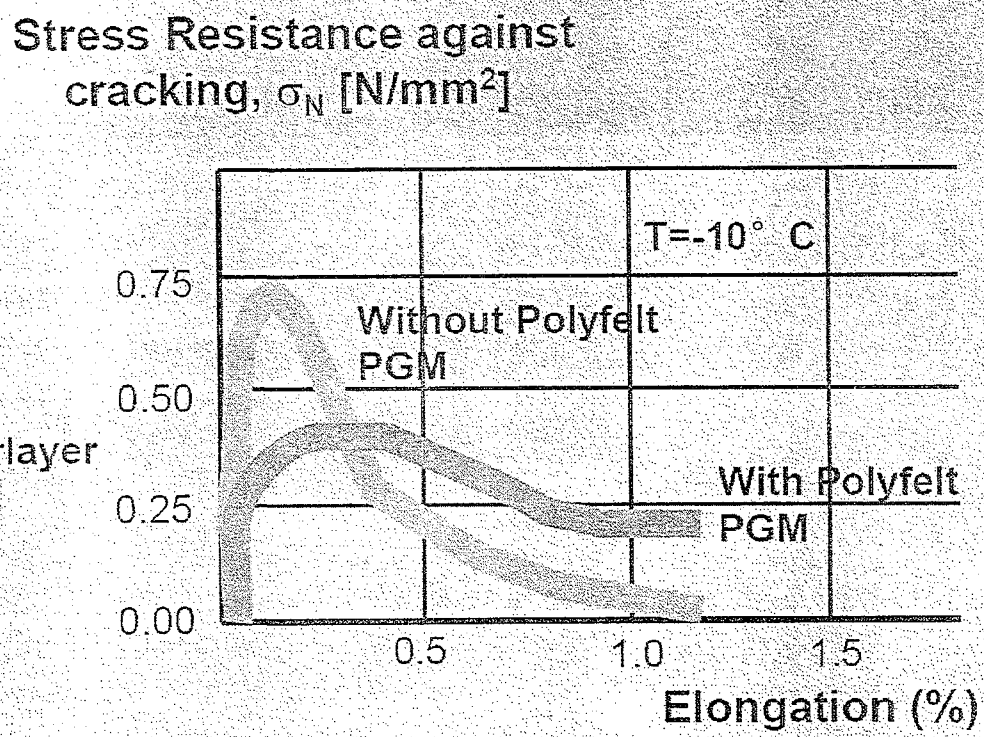


Research & Testing

Tensile test at low temperature

Test set-up





Research & Testing

Determination of temperature in a composite concrete-bitumen-interlayer-asphalt material, University of Liege, Belgium

Summary of test results

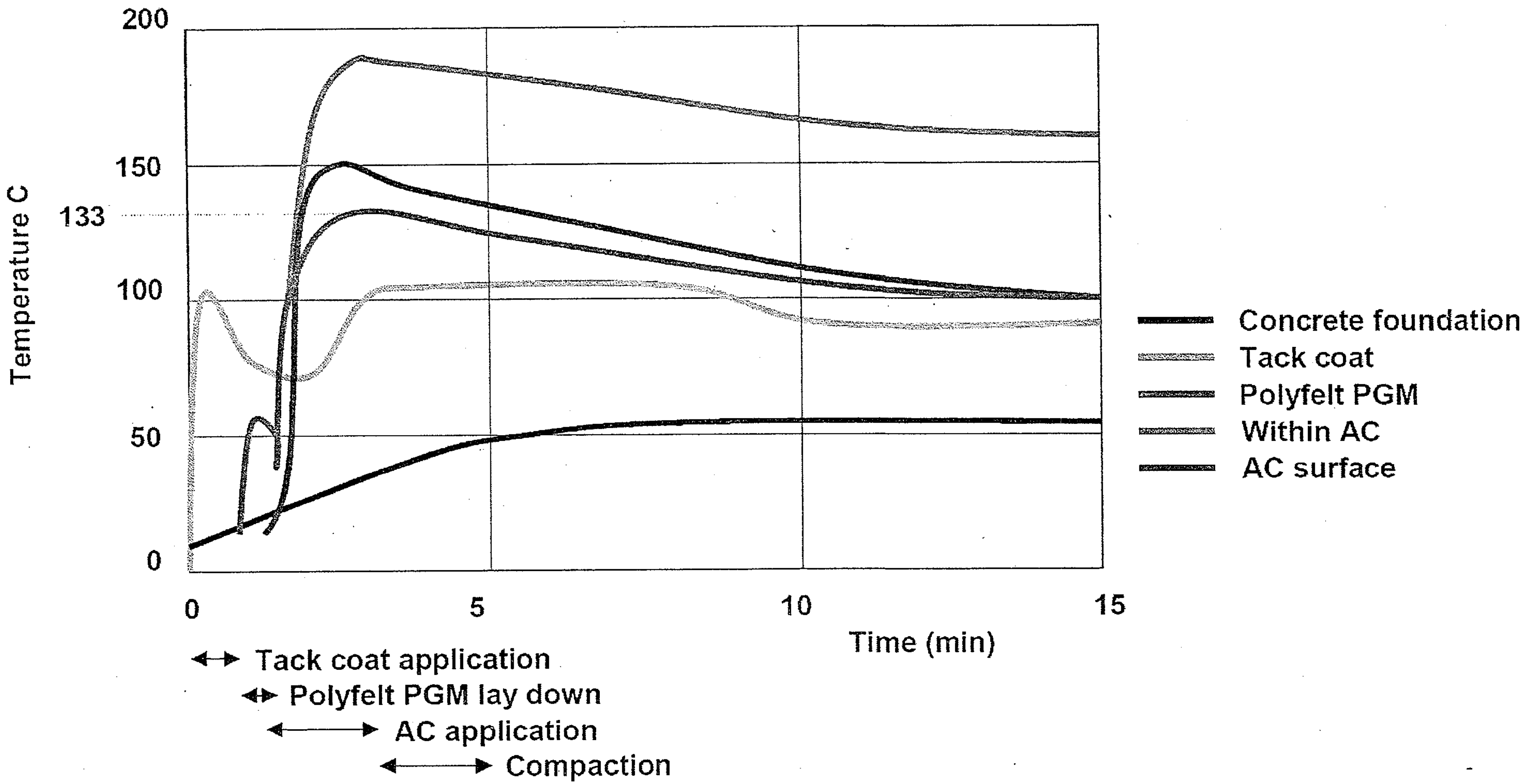
Location of thermocouples	Layer	Working temp	Measured temp when asphalt is place and compacted
Ambient temp			16C
Surface temp			150C
25mm into asphalt	Asphalt material 50mm	Mixing 3min. 180 +/- 10C	190C
	Geotextile PGM (fusion temp 163C)		133C
	Bitumen	110 +/- 5C	Conc surface 85C
30mm into concrete	Concrete 70mm	8 +/- 1 C and 35 +/- 2C	29C

● Thermocouple

Conclusion: No thermodegradation of geotextile

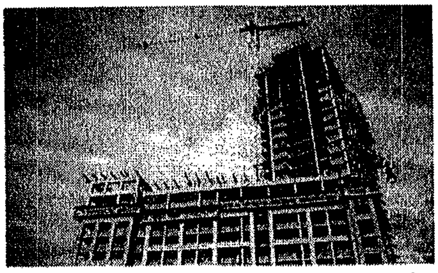
Research & Testing

Determination of temperature in a composite concrete - bitumen geotextile -asphalt material, University of Liege, Belgium



No.1 steel bar

CN STEEL



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Bio Data of Dr. Kamalnath Dissanayake

Graduated from University of Peradeniya in 1994 and served the University as a teaching assistant till 1996 and a research assistant from 1996-1997.

He proceeded to Hiroshima University Japan 1997 under a Monbusho Japanese Government Scholarship.

Obtained the Master of Philosophy from University of Peradeniya in 1999.

Title of the thesis: "Some Geotechnical aspects of Sri Lankan landslides"

Obtained the Doctor of Engineering from Hiroshima University Japan in 2002

Title of the Thesis : Rain induced landslides in Japan

Became a Chartered Engineer in 2005

Senior Lecturer in Civil Engineering at University of Peradeniya from 2002-2004

Visiting Academic Consultant at University of Ruhuha in years 2004 & 2005

Chief Geotechnical Engineer – China Harbour Engineering (Group)-STDP project Since 2005

Use of Heavy Tamping in Highway Construction-Experience in Southern Transport Development Project

A. K. Dissanayake *B.SC.Eng.(Hons), M.Phil., C.Eng., D.Eng., MIESL, MSLGS*
Chief Geotechnical Engineer, China Harbour Engineering Co. (Group),
STDP Package 1: Kottawa to Dodangoda

NANR Wanigaratna *B.SC.Eng.(Hons)*
Geotechnical Engineer, China Harbour Engineering Co. (Group),
STDP Package 1: Kottawa to Dodangoda

Abstract
Very frequently the infrastructure development activities, especially highways, railways water supply schemes, etc. and related structures are now constructed on land with poor ground support conditions. Further, the countries like Japan, Singapore, Malaysia etc., where the developable areas are scarce, reclaim land from the sea that the underground conditions are very much unfavourable for construction. Sri Lanka now faces this similar situation in par the world and most of the structures and other related constructions are taking place on improved ground. About 55% (18.9km out of 34.3km) of the main road length of package 1 of Southern Expressway runs from Kottawa to Dodangoda has gone over soft ground areas and thus warranted treatments to improve. In addition, ramps and access roads totalling 11.5km are also found on such deficient subsoils. Within this area there is a very soft soil layer along 3.6km section on the road, where the embankment height reaches an average of 5m. This paper thus presents some of the details of the subsoil conditions at these locations, their engineering properties and the design criterion adopted in Heavy Tamping method of ground improvement. Some details of method of construction, details of the trial embankment and its behaviour and the improvement to the design are also discussed herewith. It is also intended to present some measurements of pore water pressure after the design improvement.

1.0 Introduction

Very frequently the infrastructure development activities, especially highways, railways water supply schemes, etc. and related structures are now constructed on land with poor ground support conditions. Further, the countries like Japan, Singapore, Malaysia etc., where the developable areas are scarce, reclaim land from the sea that the underground conditions are very much unfavourable for construction. Sri Lanka now faces this similar situation in par the world and most of the structures and other related constructions are taking place on improved ground. About 55% (18.9km out of 34.3km) of the main road length of package 1 of Southern Expressway runs from Kottawa to Dodangoda has gone over soft ground areas and thus warranted treatments to improve. In addition, ramps and access roads totalling 11.5km are also found on such deficient subsoils.

In earlier times, poor ground areas have usually been avoided or structures with deep foundations such as a bridge supported roadway have been constructed over the top of the loose deposits. Many types of site improvement techniques are now available that allow embankments and interchanges to be constructed directly on densified ground. One form of dynamic compaction using heavy tamping is adopted in Southern Transport Development Project between Kottawa to Dodangoda to improve substantially poor ground condition at few locations. This note thus presents the technical background of this method of treatment which has been deployed first time in Sri Lanka for road construction.

2.0 Subsoil Condition

2.1 General geology of the area

The trace from Kottawa to Dodangoda is found on the southern part of the Highland complex. The rock structures within are dominated by layering with an internal fabric showing evidence of ductile deformation, including transportation of layering coupled with extreme flattening layering and stretching, characterised by a thick sequence of gneisses. The major rock types occurring within the area of concern are charnockite, Charnockitic gneisses, and garnitiferous quartzo feldspathic gneisses. These rocks are overlaid by residual soils and alluvium. The insitu weathered soil overlying the bedrock in hilly terrain shows complete range of weathering forming completely weathered lateritic clays and silts. River alluvial sediments that are usually found underlying the valley beds as well as in floodplains consist of brown micaceous clay layers occasionally intercalated with sand and gravel. Marsh and lake alluvial deposits encountered in the similar topographic conditions above are mainly black to brown peat and organic clays which sometimes carry patches of white sand.

2.2 Ground profile and soil parameters

Out of about 18.9km length of poor ground within the project area a section of about 3.6km in length is found to be very soft as shown in Fig. 1 below. Free fall of SPT has been observed at many instances along the centreline of the expressway. Ground exploration was carried out at each 50m interval using either Machine Boring with SPT, Cone Penetrometer with pore pressure measurements and Hand auger boring.

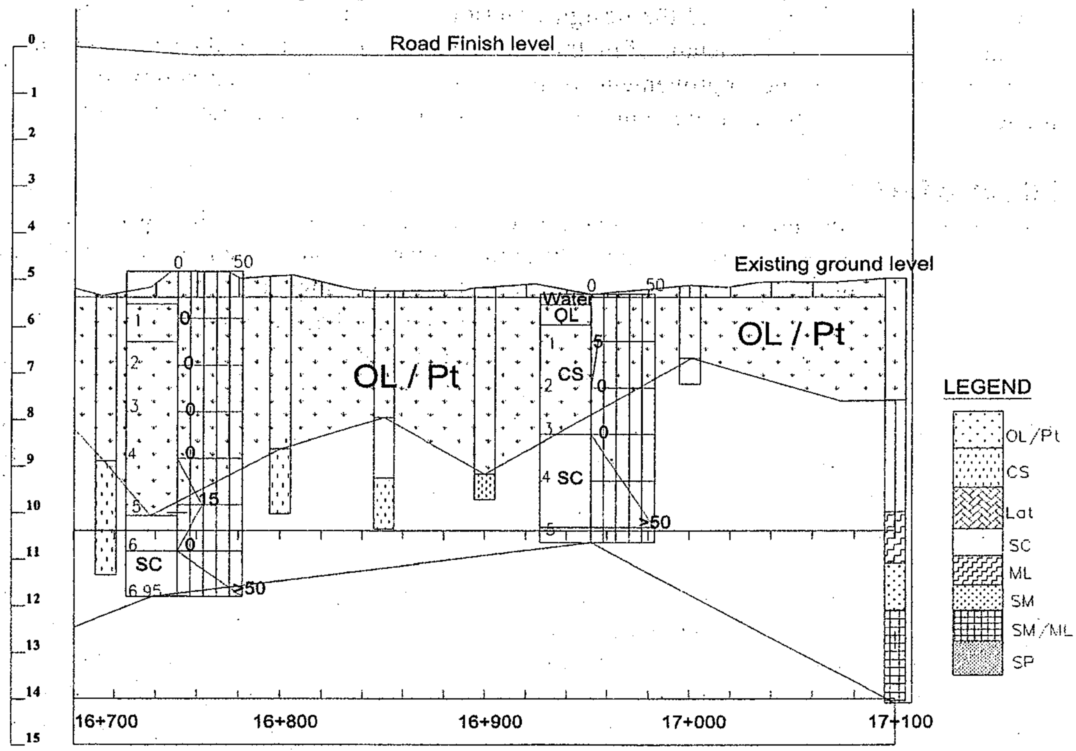


Fig. 1 Ground profile from Ch 16+700 to Ch 17+100

2

The poor ground encountered within this area comprises of valleys and depressions in the bedrock and decomposed rock formation that has been filled with very soft to soft alluvial clays, silts, organic soils and deposits of amorphous fibrous peat. The thickness of the amorphous peat layer is about 2m to 6m. The underlying organic clay layer also ranges from 1 to 4m and the total depth of poor ground comprising peat, organic soils and soft clay varies from 6m to 10m.

Some of the basic soil properties for different type of soil encountered at those poor ground area were also found from this additional investigation and are tabulated below in Table 1.

Table 1 STDP- Package 01 (Ch.25~27)

Material	Organic content (%)	Moisture content (%)	Specific gravity g/cm^3
Peat	87.693	210.30	1.73
	21.604	407.08	2.22
	51.582	248.66	1.68
	46.506	236.81	1.98
Clay / Organic Soil	49.407	84.85	2.36
	-	84.55	2.31
SC	-	23.48	2.34
	1.385	21.55	2.54
	-	20.94	2.4
SM	-	23.48	2.34
SW-SC	-	26.15	2.64

3.0 Heavy Tamping

Because of this very soft ground support condition it was earlier decided to adopt remove and replacement by rock method to support the embankment, the maximum height of which rises to about 7m above the existing ground level. However, due to adverse environmental and social considerations arising from remove and replacement method as experience elsewhere in the same project Engineer (consultant) proposed new method ground treatment method called heavy tamping. A trial of heavy tamping was proposed and constructed at a predetermined location along this problematic stretch of the road and following sections describe the method adopted in implementing the construction.

Dynamic compaction via heavy tamping technique consists of using a heavy tamper (pounder) that is repeatedly raised and dropped by a crawler crane from varying heights to impact the ground. The energy applied in phases on a grid pattern over the entire area using either single or multiple passes. Following each pass, the craters that are created due to the drop of tamper are either levelled with a dozer or filled with granular fill material before the next pass of energy is applied.

All of the energy is applied from existing grade and the degree of improvement is a function of the energy applied: i.e., the mass of the tamper, the drop height, the grid spacing, and the number of drops at each grid point location. Figures 2 to 4 illustrate the dynamic compaction process and the equipment that has been used on a regular basis

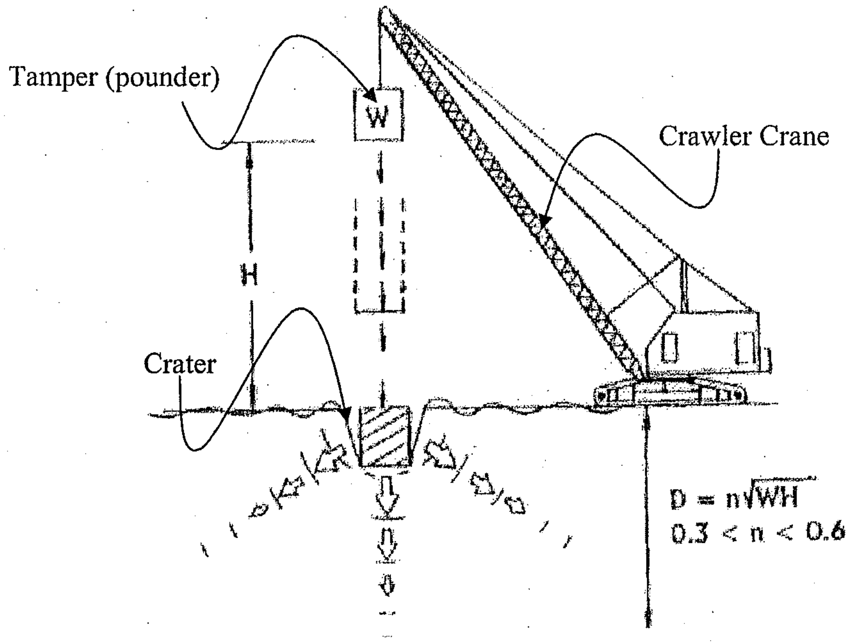
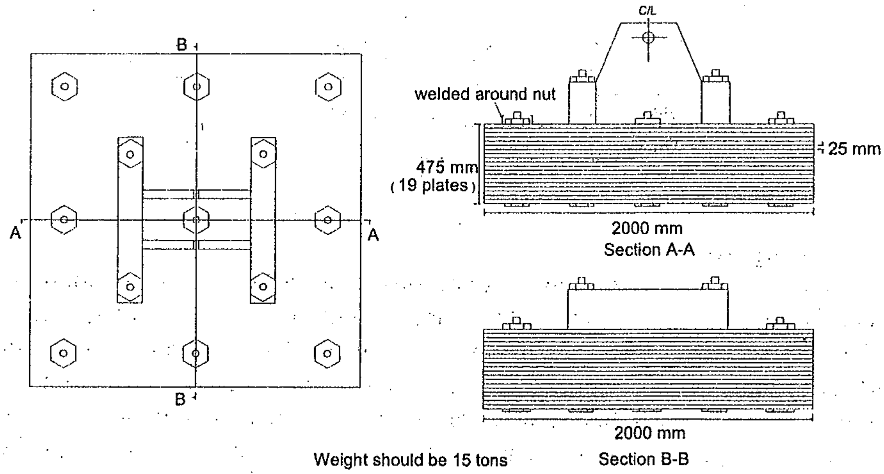


Fig.2 Basic equipment use in Heavy Tamping



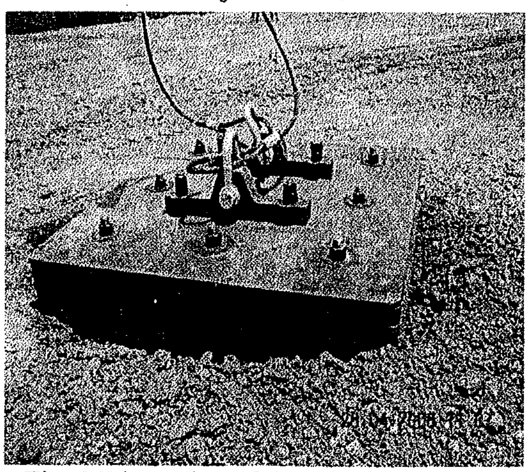


Fig. 3 Dimension of the pounder and its view

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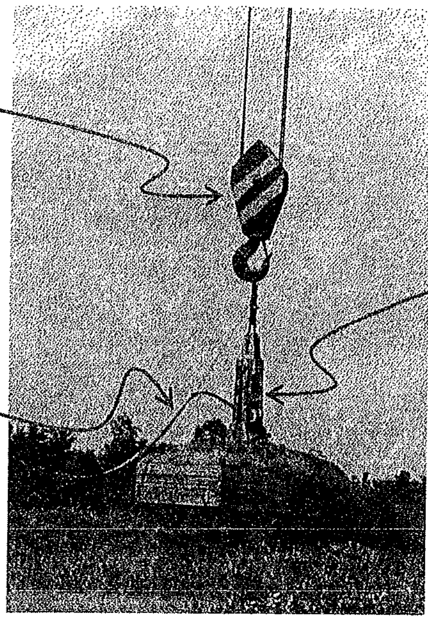


Fig. 4 Hydraulically operated releasing mechanism (ALARM) attached to the crane

The basic necessary equipment and accessories for heavy tamping operation are crane, pounder and a releasing mechanism. In order to get 15T weight, steel plate assembled unit as shown below (Fig. 3) was fabricated. The releasing mechanism of the pounder is attached to the crane as shown in Fig. 4 above.

3.1 Design Plan

In order to carry out the improvement to the poor ground conditions a proper heavy tamping plan must be developed by considering following items.

1. Selection of the tamper mass and drop height to correspond to the required depth of improvement.
2. Determination of the applied energy to be used over the project site to result in the desired improvement.
3. Determination of the grid spacing and number of phases.
4. Establishing the number of passes.
5. The need for a surface stabilizing layer.

This detailed design process could be summarised and presented in Table 2 below. The ***n*** value for the Equation 1 is varying from 0.35 to 0.4 for Clayey soils and thus, the lowest value of 0.35 was considered in this particular design. Therefore, the estimated depth of ground improvement with a 15T pounder weight and an average height of drop of the pounder about 8m is 3.8m below the ground level.

It is also important to note that for this operation the capacity of the crane should be always greater than four times the weight of the pounder and hence, 80T cranes are used to carry out the improvement work.

Table 2 Design guidelines (Modified from FWHA publication no. SA-95-037-Dynamic compaction)

Parameters to be Determined	Evaluation Process
Step 1: Selection of tamper and drop height for required depth of improvement Equation 1: $D = n(WH)^{0.5}$	A. Determine thickness of loose deposit from subsurface exploration or the portion of the deposit that needs densification to satisfy design requirements. B. Use Equation 1 and select n value from Table 7 for soil type. C. Use Figure 21 as a guide in selecting tamper mass and drop height for dynamic compaction equipment currently in use.
Step 2: Determine applied energy to achieve required depth of improvement	A. Use Table 8 to select the unit energy for the proper deposit classification. B. Multiply the unit energy by the deposit thickness to obtain the average energy to apply at ground surface.
Step 3: Grid spacing and drops Equation 2: $AE = \frac{W\sum_{i=1}^{P} Ni \times Hi}{\text{Representative grid area}}$ Where: N = number of drops P = number of passes W = mass of tamper H = drop height	A. Select a grid spacing ranging from 1.5 to 2.5 times the diameter of the tamper. B. Enter W and N from step 1 and applied energy from step 3 into Equation 2. C. Use Equation 2 to calculate the product of N and P. Generally 7 to 15 drops are made at each grid point. If the calculations indicate significantly more than 15 or less than 7 drops, adjust the grid spacing.
Step 4: Multiple Passes Prediction of crater depths or ground heave in advance of dynamic compaction is difficult. The contract should provide for multiple passes where very loose deposits like landfills are present or where silty deposits are nearly saturated.	A. Crater depths should be limited to the height of the tamper plus 0.3 m. B. Energy application should stop if ground heave occurs. C. If items A or B occur before the required number of drops are applied, multiple passes should be used to: * permit ground levelling if item A occurs * allow pore pressure dissipation if item B occurs
Step 5: Surface stabilizing layer	A. Not needed for Zone 1 soils. May be required for Zone 2 soils if nearly saturated. Usually required for landfills. B. When surface stabilizing layer is used, the thickness generally ranges from 0.3 to 0.9 m.

The trial are of 60m×40m is selected to be carried out at the section from ch17+370 to ch17+430 based on the ground condition observed during the additional site investigation. The applied energy requirement of this area was calculated based on the estimated settlement of the soft soil at the trial location upon the loading from the embankment and the traffic loads. The design curves for enforced settlement and the relation of applied energy and enforced settlement are shown below in Figs 5 and 6.

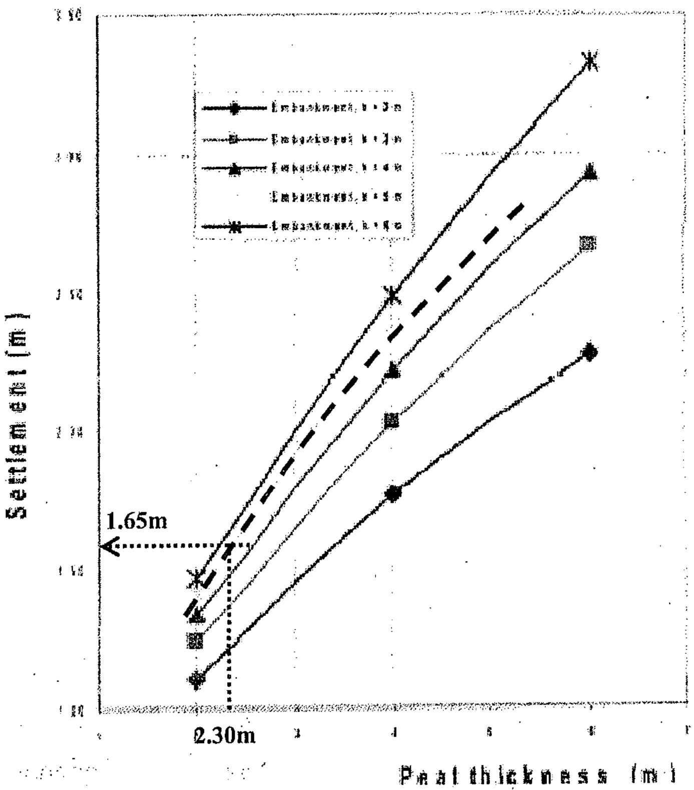


Fig. 5 Approximate settlement under embankment height in Peat or highly Organic Clay

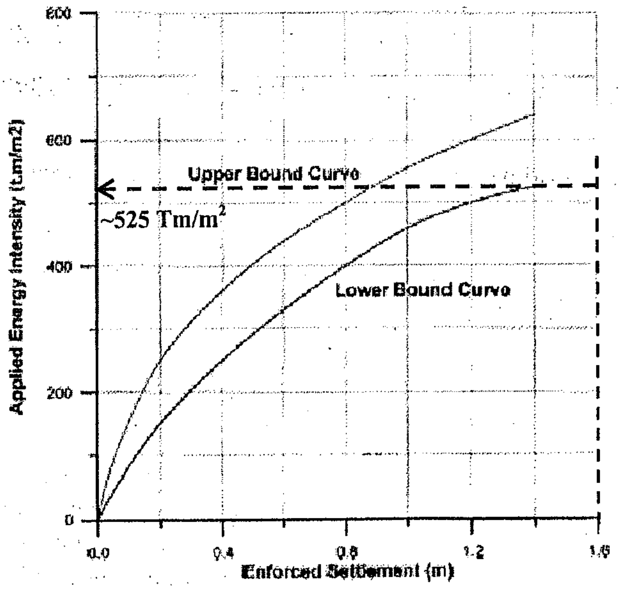
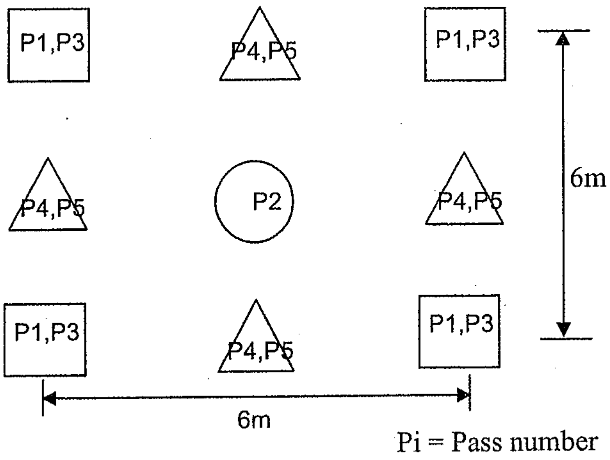


Fig. 6 Relation of applied energy and the enforced settlement
(Figure 3 of Prescribed procedure and Specification)

Therefore, the requirement of the energy application to the trial area was about 500 Tm/m^2. For the above energy level, the grid spacing and passes were decided as shown below.



Stage	Pass	Energy status (Weight ×Height × # blows)	# Prints	Expected energy Tm
I	1	15T×5m×10	74	55500
	2	15T×5m×10	58	43500
	3	15T×5m×10	74	55500
	4	15T×8m×20	68	163200
II	1	15T×8m×25	74	222000
	2	15T×8m×25	58	174000
	3	15T×8m×25	74	222000
	4	15T×8m×25	68	204000
Total Energy applied to the area of 60m×40m				**1139700**

Fig. 7 grid pattern and the schedule of multiple passes

Form the above figure, the total estimated applied energy intensity at the trial area is estimated to be about 475 Tm/m^2.

Based on the above design plan, the actual work at the site was carried out as per the activity diagram shown in Fig. 8 below.

Although the guidelines are useful in planning and implementation heavy tamping, it cannot replace good judgement. Frequently during densification, adjustments must be made to the planned programme based on how the ground is responding to impact. Hence, the design engineer must always use good judgement to supplement or alter the guidelines.

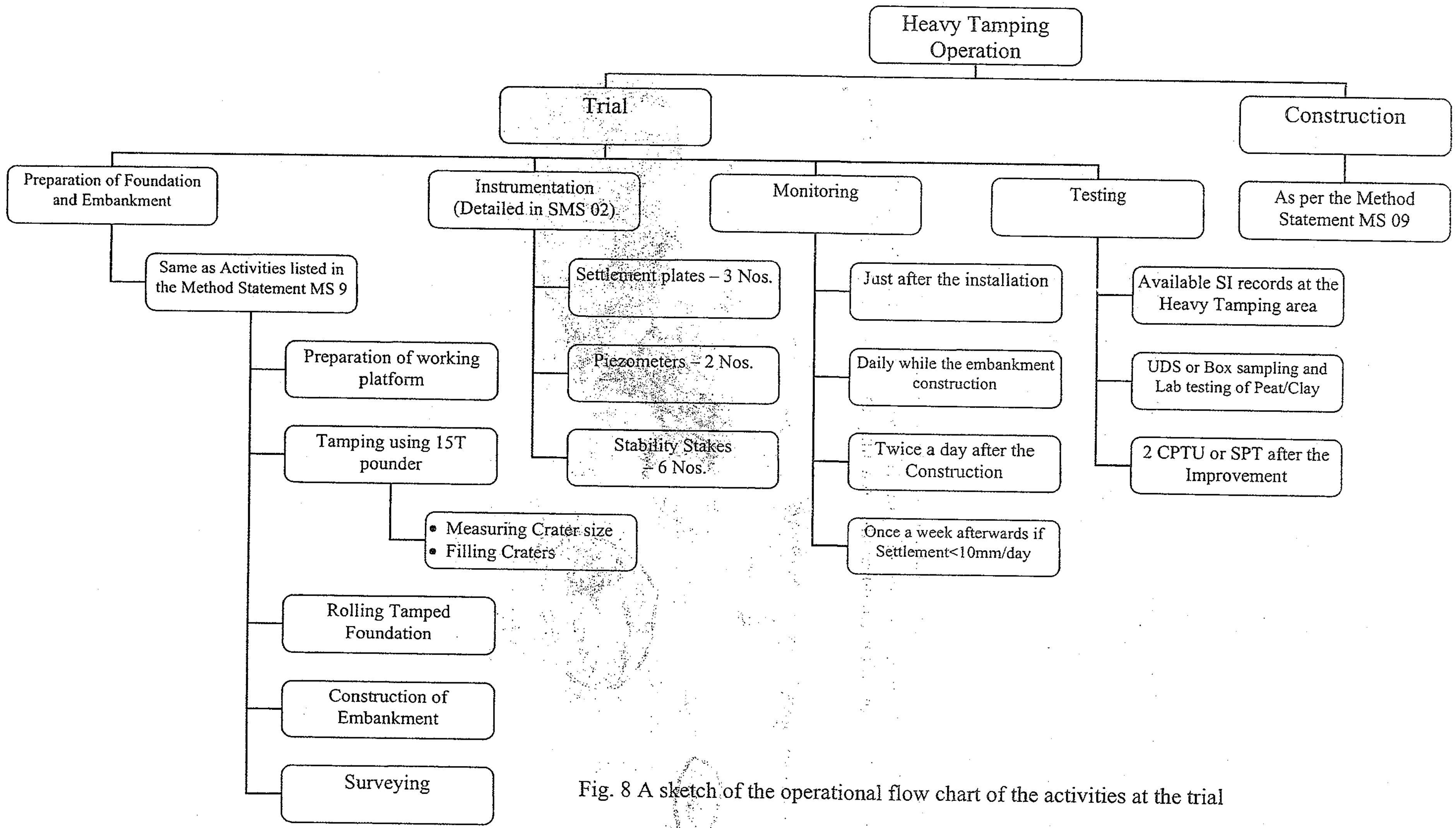


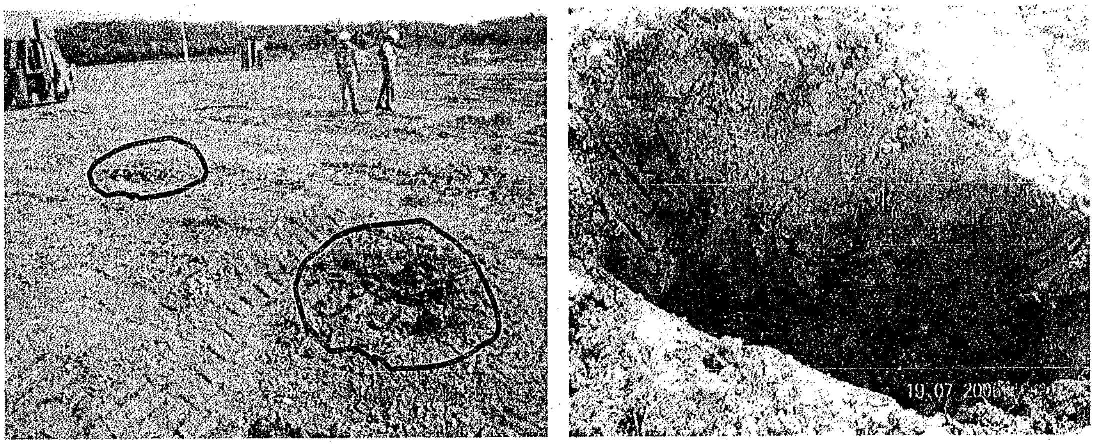
Fig. 8 A sketch of the operational flow chart of the activities at the trial

3.2 Site operation

The first activity of this operation is to construct the working platform over the soft ground area along alignment of the trace. This has been carried out using soil from general cutting area and it was understood that the platform must be well above the existing water level or existing ground level, which ever is higher, in the area for efficient performances. Therefore, a platform of about 1m above the soft ground was constructed at these locations. After the completion of the platform preparation, a levelling survey was conducted to determine the height of the platform before the tamping.

According to the scheduled programme showed in Fig. 7 above, the tamping operation started with the Stage I energy application and completed the operation within the month of June 2006. While carrying out tamping, the sizes of the craters formed were measured to obtain their volumes. Similarly, the tamping for the Stage II energy application was carried out starting from Phase 1 as per the above Fig. 7. Similar to Stage I operation, all the site measurements of craters formed during tamping was obtained and reported. The other operations such as transporting materials to the site and filling craters were carried out as per the general filling operations.

While completing the Phase 4 of this stage it was observed that the softer material underneath is oozing out of the platform from several spots (Fig. 9) and thus, the tamping operation was ceased with the direction of the Engineer after reaching the actual energy application of about 162 Tm/m^2.



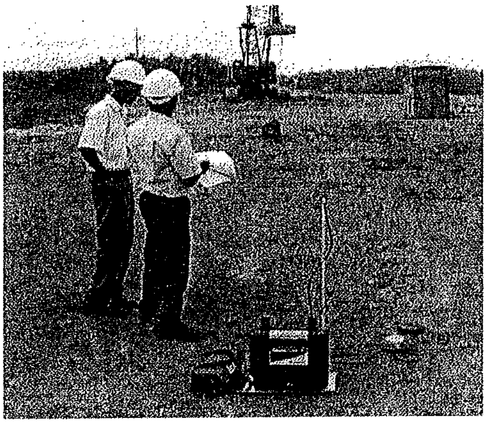
a. Some spots where soft soils had oozed out from the platform

b. Excavation made at a oozing out location to observe the condition

Fig. 9 Oozing out of soft material due to intense tamping at the trial location

3.3 Noise and vibration monitoring

Noise and vibration monitoring has been carried out at different locations along the trace and also perpendicular to the traverse as shown in Fig. 10 below. These tests were carried out understand the impact of ground vibration and noise due to tamping operation along the trace and would be useful when deciding the next location for the operation with consideration to mitigate nuisance to the public.



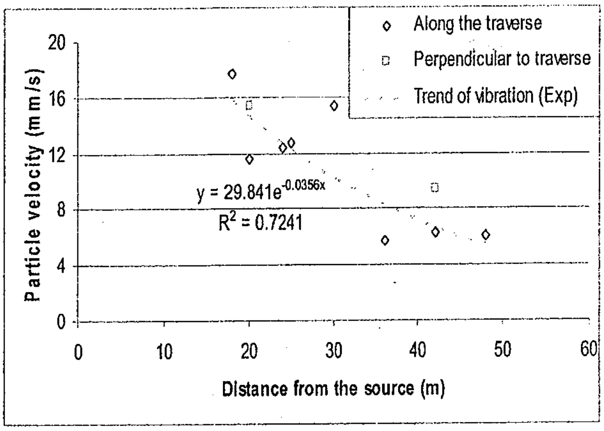


Fig. 10 Variation of vibration due to tamping at the trial area

3.4 Foundation preparation and surveying

After the completion of tamping the undulations on the ground surface due to heaving of soil was scraped and few roller passes were made to make the foundation surface much flatter for supporting the embankment loading. Thereafter, such prepared platform was surveyed once again to obtain the final level of the platform.

3.5 Geotechnical investigation

The ground profile before the trial was shown in Fig. 1 above following the additional investigation carried out for embankment locations.

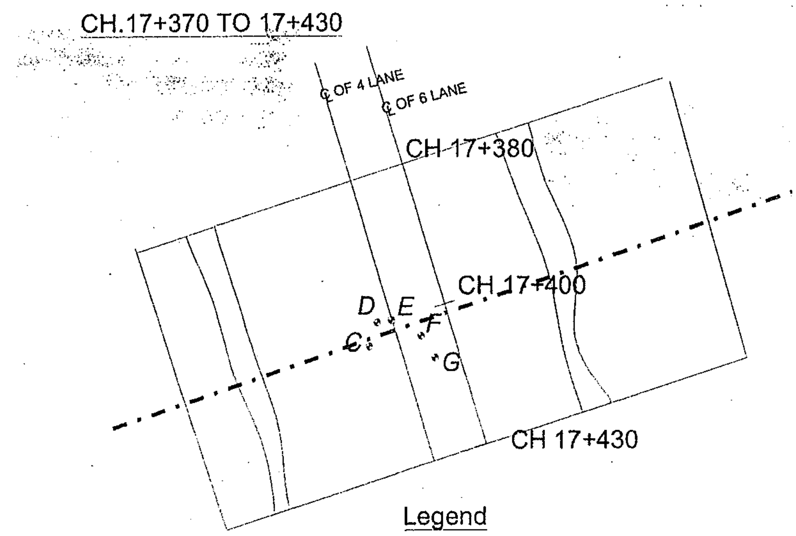


Fig. 11 Locations of exploratory points at the trial area after improvement

Ground profile after the improvement

The variation of subsurface soil profile was investigated after the improvement using Standard Penetration Tests carried out at locations shown in Fig. 11 above. Exploratory points were selected such that at least one should be located on a print location and another on a non-tamped location; i.e. a space in between two tamped (print) locations where tamping was not carried out. Thus obtained soil profile is then estimated and plotted in Fig. 12 below.

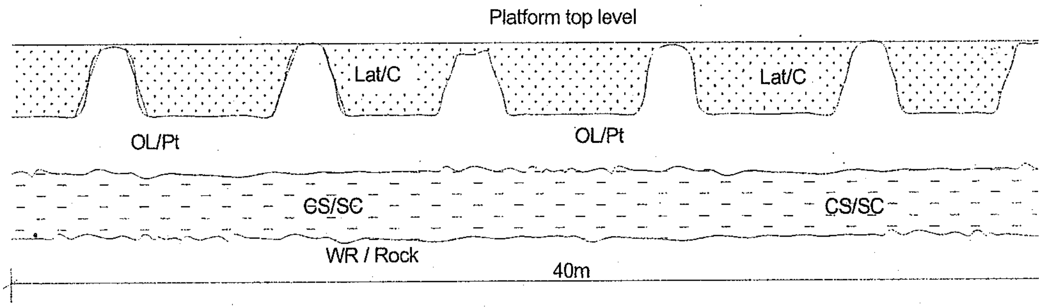
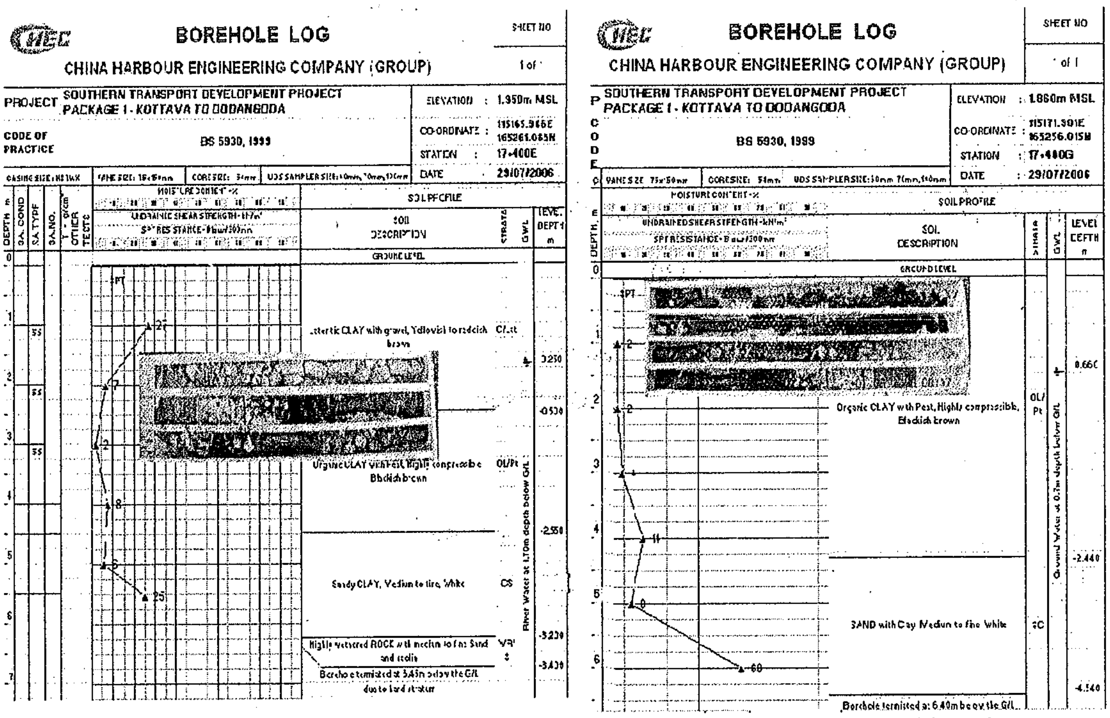


Fig. 12 Estimated soil profile along A-A of Fig. 14 after the subsoil investigation using SPT

The logs obtained from boreholes carried at above locations are given in the Fig. 13 below.



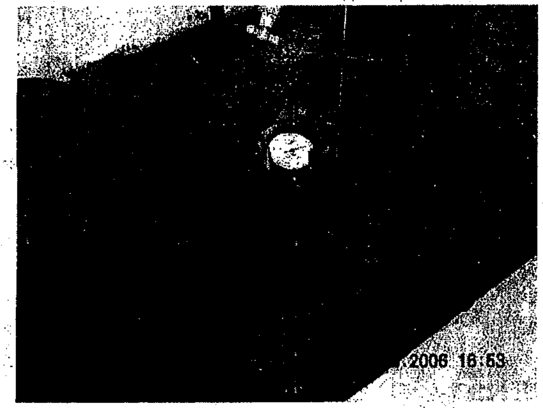
a. Borehole at a print location

b. Borehole at non tamped location

Fig. 13 Borehole logs after heavy tamping

Laboratory testing

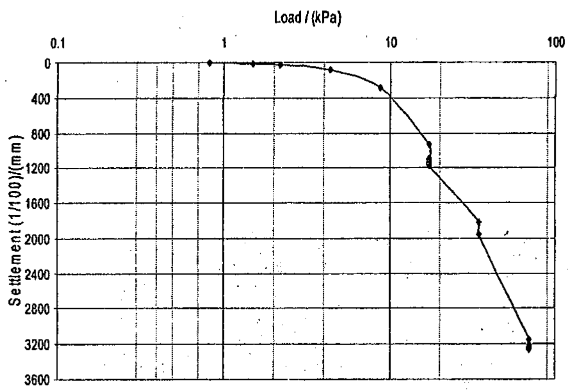
In order to test the settlement characteristics of the peaty soil found at the trial location, an undisturbed sample of soil was extracted from the trial location to a CBR mould and a large scale consolidation test was carried out at the lab in the office premises. Some of the photos taken from this exercise are shown in Fig. 14 below.

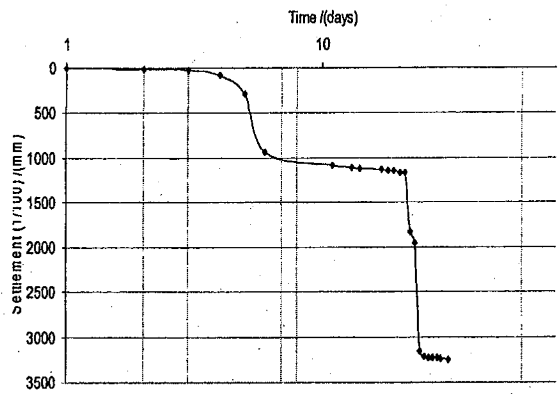


a Extracted sample ready for testing *b* Loading in the laboratory

Fig. 14 Sampling of soil at trial area and preparation for laboratory testing

Thus, the 1-D consolidation test results obtained are presented in Figs. 15 below.





a. Time Vs Settlement plot during loading *b.* Load settlement plot of the behaviour

Fig. 15 Characteristics of the peat under large scale laboratory consolidation test

3.6 Instrumentation and monitoring

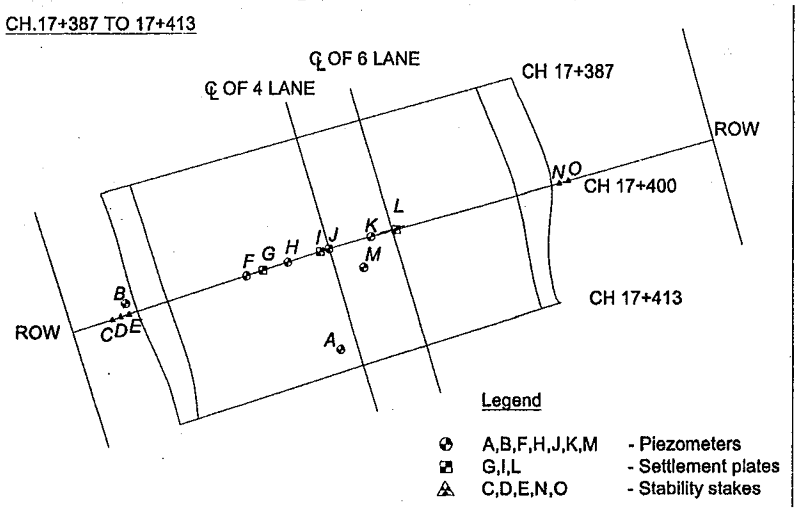
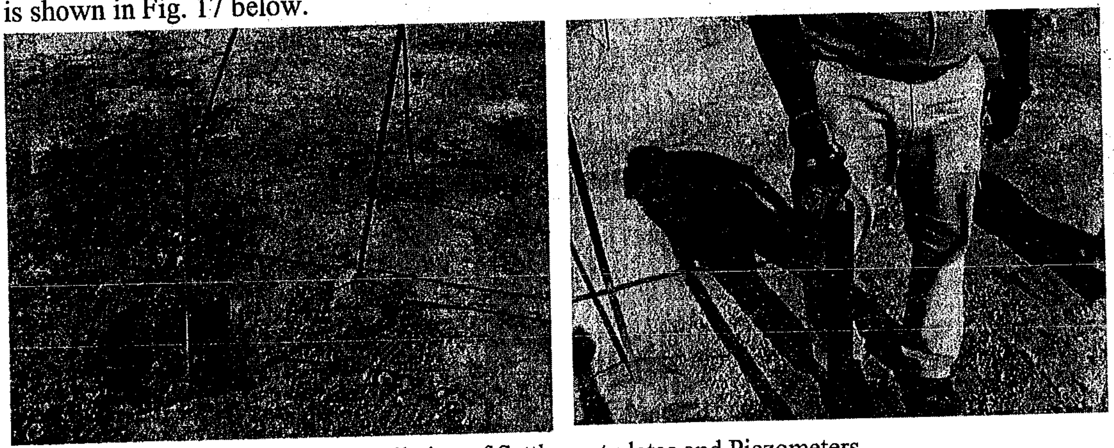
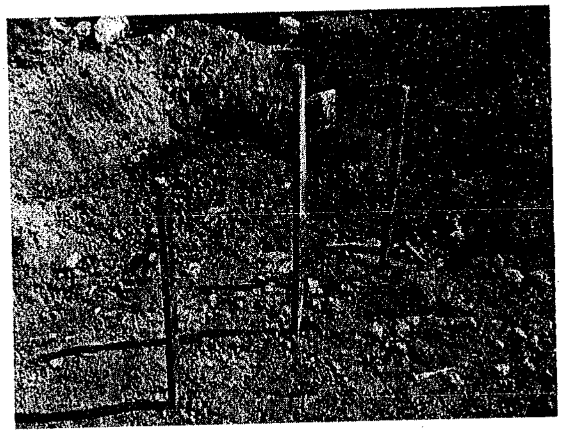


Fig. 16 Layout plan showing the locations of instruments at the trial area

After the preparation of platform by tamping, followings were installed before the loading of embankment takes place as presented in the layout plan shown in Fig. 16 above. As per this figure, seven stand pipe piezometers, three settlement plates and five stability stakes were installed to monitor the performance of the embankment after the treatment. The number of piezometers to be installed was increased from 3 to 7 when the pore pressure under the foundation just after the improvement was observed high. The instrumentation at the trial site is shown in Fig. 17 below.



a Installation of Settlement plates and Piezometers



b Installation of stability stakes at the edge of embankment

c Instrumentation at site

Fig.17 Instrumentation at the site

After installation, the embankment soil surcharge was placed in loose condition and the construction was completed within two weeks raising the embankment level about 3m from the platform level.

Piezometric Observations

During and after the surcharge the observations from the piezometers at the site are presented below in Fig. 18. They show a considerably high pore pressures under the embankment. This figure herewith presents the difference of pore water pressures within and outside the foundation area for the trial before and after loading the embankment. It is obvious from this observation that the pore pressure once imposed either due to tamping or surcharging remains virtually constant over the time and thus, the dissipation excess pore water pressure is found to be extremely slow. At the last observation the pressure difference is about 5.5m to 6.0m in excess and some of the standpipe piezometers were found overflowing occasionally.

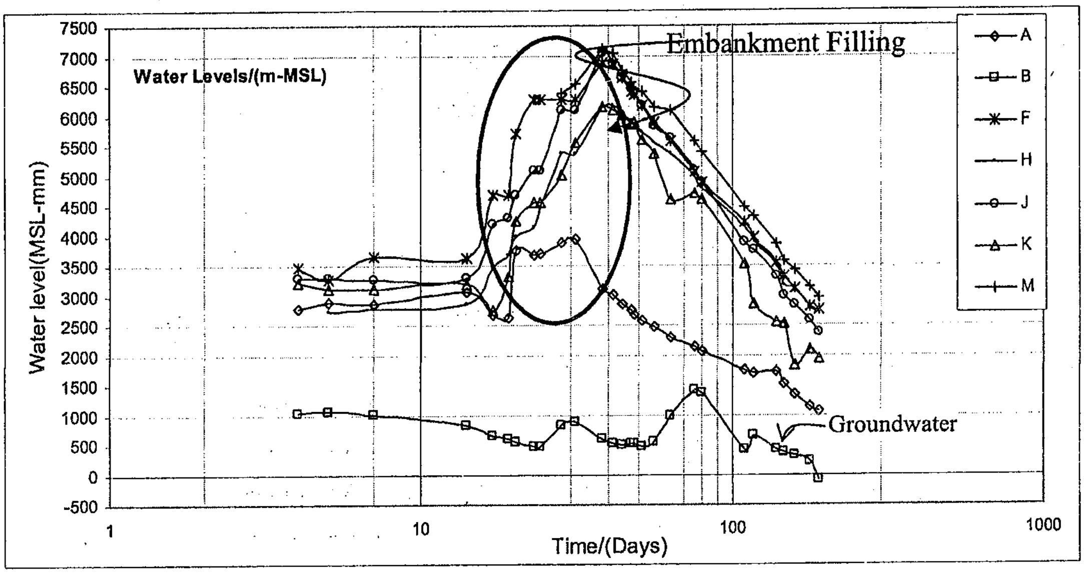


Fig. 18 Pore water pressure under the platform

Settlement observations

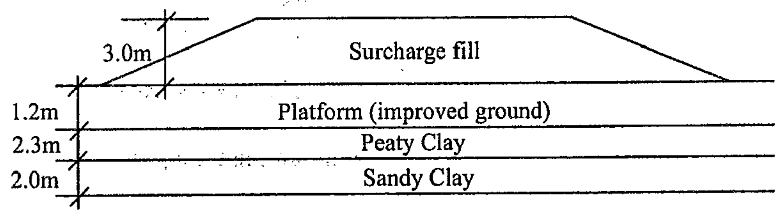


Fig. 19 Estimated ground profile at Ch 17+400 (see interim report)

The estimated soil profile beneath the embankment after the improvement using heavy tamping, which is used for the calculation of settlement, is presented in Fig. 19 above. Settlements observed at the heavy tamping trial area on the settlement plates are plotted, along the predicted settlement curve for 2.3m of peaty clay layer, as shown in Fig. 20 below. The locations of the settlement plates are shown in Fig. 16 above and the parameters used for settlement prediction is presented in Table 3 below. It is shown from this figure that the estimated total settlement of the trial embankment is 725mm. It could be also observed that the measured settlement of the plates slowly approaches the predicted settlement curve and would be continued for long period unless the excess pore water pressure is dropped.

Table 3 Properties of platform, embankment and underlying soil at trial area

Parameter	Material		Values
Unit weight γ_t in kN/m^3	Embankment (Loose)		16
	Platform		20
	Peat		10.5
	Sandy Clay		15.7
Shear Strength	Peat		C = 5kPa, $\phi = 0$
	Sandy Clay		C = 14.7kPa, $\phi = 0$
Consolidation	Peat	C_c	3.286
		C_r	0.1429
		e_o	6.85
		P_c /(kPa)	15

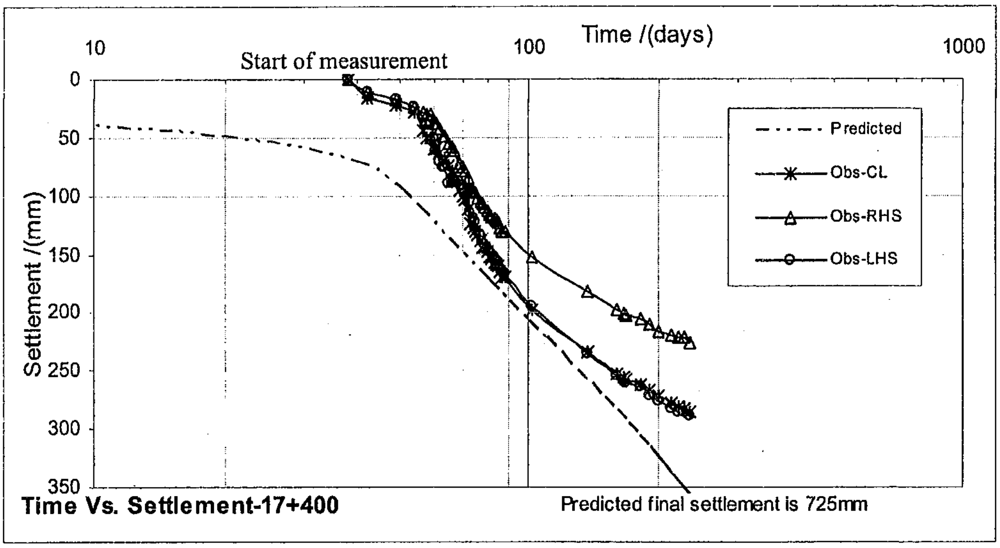
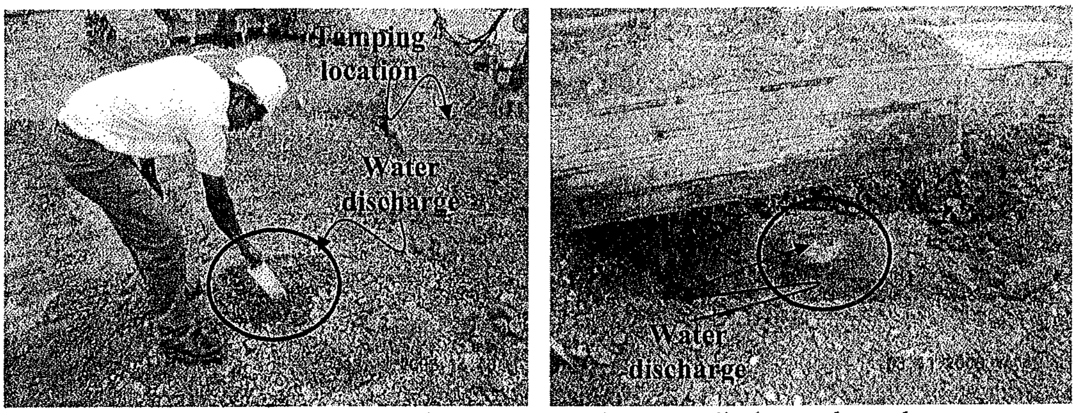


Fig. 20 Measured settlement of the embankment against the predicted

3.7 Application of Wick (Band) Drains

Since the observation of very high pore water pressure at the site after embankment fill, it was decided to install band (wick) drains in 1.5m grid spacing to accelerate discharging excess pore pressure before the next pass. Then the band drains were installed outside the trial area at ch 17+600 to 17+660 as per the photo below (Fig. 21) and tamping was carried out as discussed previously. Measurement of pore water from a standpipe piezometer located at ch17+630 showed a substantial improvement of excess pore pressure dissipation at this tamping location (Fig. 22). Therefore, it was learnt from this exercise that for soft clayey or silty sub soils it is necessary to install PVD prior to tamping in order to get rid of excess pore pressure generation before the next pass, which otherwise would lead to non densification due to early energy saturation condition.



a. Water discharging from a band drain near the tamping location
b. Water discharge through a crater

Fig. 21 Functioning band drains at the site during tamping operation

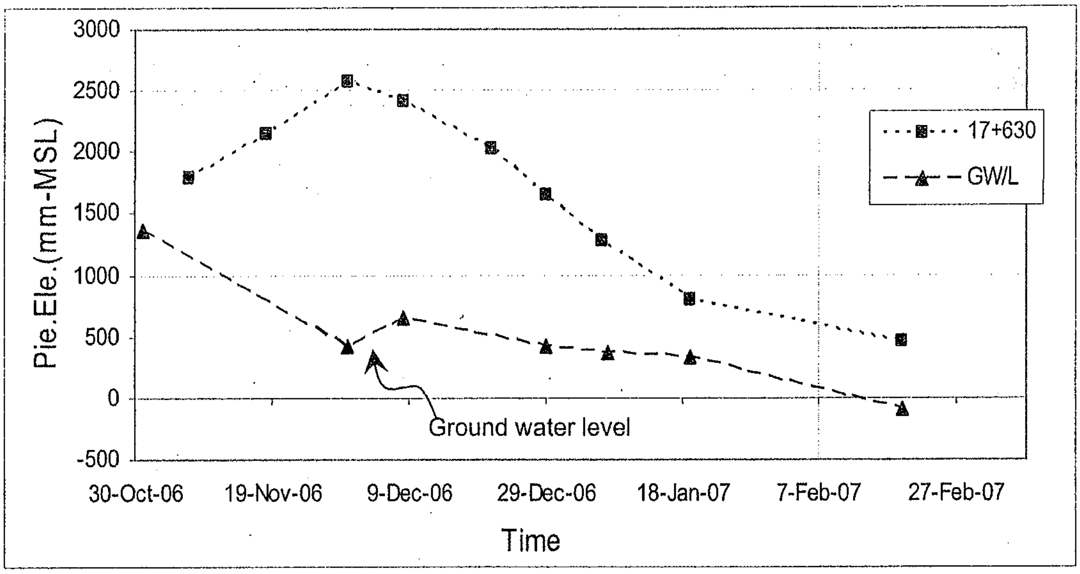


Fig. 22 Rapid pore pressure dissipation during and after tamping operation with 300Tm/m^2 energy density

4.0 Stability analysis of the embankment

Stability of the improved ground with the platform was analysed for possible different possible conditions and the results are presented in Table 4 below. The Slope/W 2004 software was used to carry out the analysis and an example of the stability analysis results is shown in Fig. 23 below. The shear strength properties are assumed following the results of SPT at the trial area. This analysis indicates that when the undrained shear strength is as high as 100kPa, the stability of the embankment is marginal with ground water pressure as high as 6m.

However, during the monitoring period up to now since July 2006, no instability of the trial embankment was observed and almost no movement is recorded on stability stakes installed either side of the trial embankment.

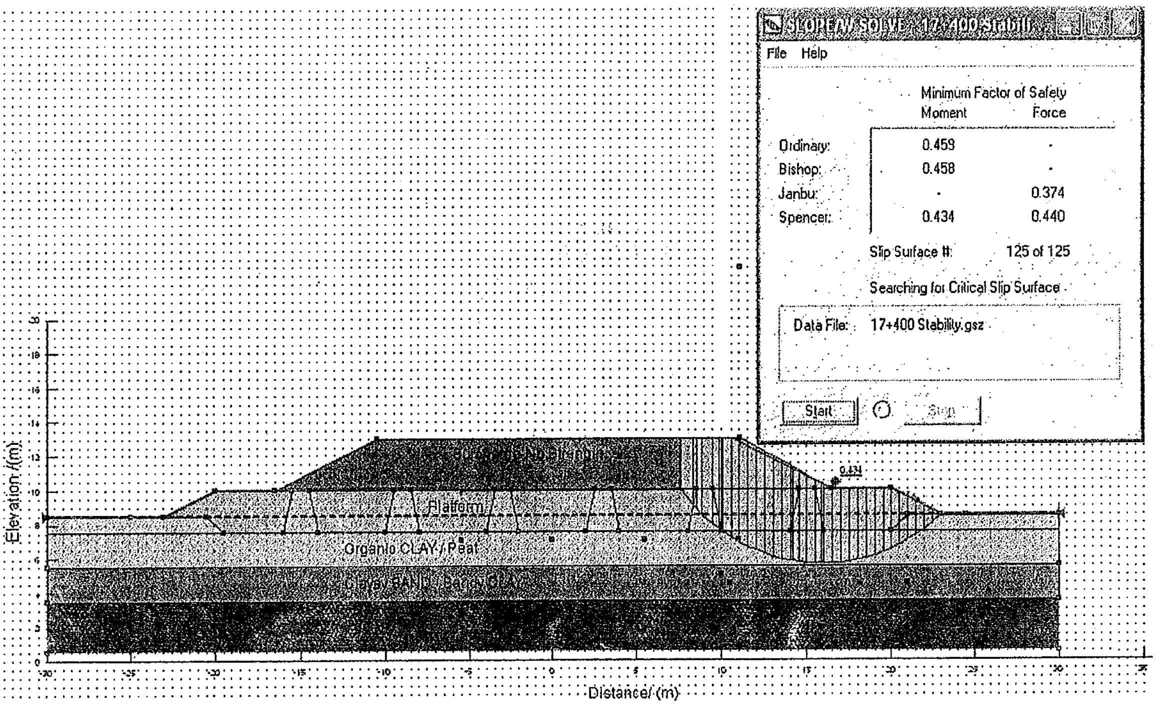


Fig. 23 Failure of the platform in long term stability (C'=0, ϕ'=30°)

Table 4 Summary of the results of stability analysis

Case No.	Material property: Platform		Material property: Embankment		Pore water Pressure	FOS: Spencer	FOS: Janbu
1	C	$100kN/m^2$	(No shear Strength) $\gamma = 16kN/m^3$		Piezometric (observed)	1.04	0.98
	ϕ	0					
2	C	$50kN/m^2$			Piezometric (Observed)	0.67	0.62
	ϕ	0					
3	C	$30kN/m^2$			Piezometric (Observed)	0.53	0.45
	ϕ	35^0					
4	C	0			Normal Ground water	0.44	0.37
	ϕ	30^0					
5	C	0	γ	$20kN/m^3$	Normal Ground water	0.99	0.93
			C	$30kN/m^2$			
	ϕ	30^0	ϕ	35^0			

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5.0 Concluding remarks

When considering a safe design of an embankment the general requirement is that both settlement and stability criteria have to be satisfied. Therefore as a summary, behaviour of the embankment with respect to both settlement and stability is discussed herewith.

5.1 Settlement

From the compression properties of peat and underlying compressible material, the total settlement of the embankment was estimated to be 725mm which would span through about 10 years. However, it could be seen from the settlement monitoring curve shown in Fig. 20 that the measured settlements are now deviating from the predicted curve. Therefore, Asaoka's settlement prediction curve is drawn with the measured data to find the trend of the actual settlement at trial location (Fig. 24)

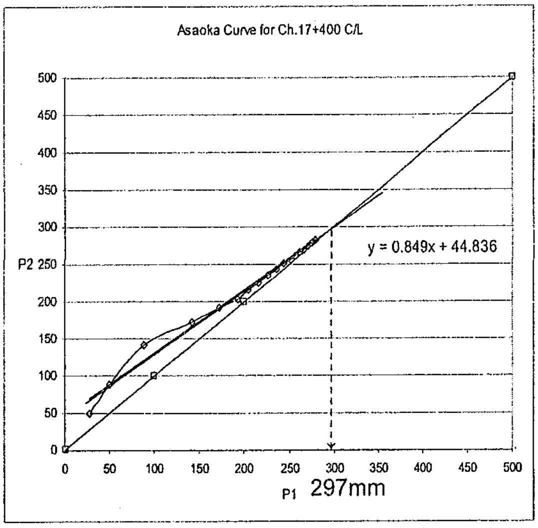

Fig. 24 Asaoka's plot of measured settlement

From this figure it was found that the final settlement of the embankment with surcharge loading is going to be only 297mm. Therefore, it could be concluded that during the Heavy Tamping operation the compressive strength of the underlying materials have substantially improved so that the predicted settlement based on the measured values are far less than the estimated settlement from parameters before tamping. Hence, much controversial secondary component could be easily arrested by simple preloading. The application of PVD would also contribute positively for rapid completion of required settlement.

5.2 Stability

It was understood from the stability analysis of the trial embankment that due to the rise of pore water pressure underneath during construction would lead to marginal instability in short term. Hence, if the rise of pore pressure could be controlled then the embankment would become stronger and stable even just after the construction. Therefore, it is necessary to provide a rapid water pressure discharge mechanism, such as PVD installation, to be adopted together with the heavy tamping to enhance the stability of the structure.

6.0 Acknowledgement

Firstly, we must be thankful to the Road Development Authority for allowing us to present these data from an ongoing project. We sincerely grateful to Prof. G.P. Karunaratne, who in fact, has introduced this environmentally friendly method of ground improvement to the Sri Lankan Civil Engineering industry and particularly this time for the road sector. We really appreciate his enormous effort and dedication towards teaching and support he rendered on practical application. We extend our sincere thanks to our Project Manager who provided his fullest support and permission for publication and the staff China Harbour Engineering Group of Company (Group) who involved in the construction and monitoring activities at the site. Finally, our sincere thanks are offered to the organisers of this event for letting us to share our experience with the engineering community.

Reference

1. R.G. Lukas, Federal Highway Administration USA, Dynamic Compaction, Geotechnical Engineering Circular No. 1, Publication no. FHWA-SA-95-037, p108, 1995.

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Ir. Lee Eng Choy is the Senior Technical General Manager of Emas Kiara Marketing Sdn. Bhd. He graduated with a Bachelor of Science degree in Civil Engineering from the University of Miami, Florida in 1985 and obtained a Master of Philosophy in Geotechnics from the University of Sheffield, United Kingdom in 1993. He is currently the Deputy Chairman of the Geotechnical Engineering Technical Division of the Institution of Engineers, Malaysia (IEM) as well as a Council Member of IEM.

He started his career as a Structural Design Engineer with the Public Works Department, Malaysia from 1985. He then went on to acquire experience in geotechnical engineering as a Geotechnical Engineer with the Public Works Research Institute, where he has been involved in the planning of soil investigation works, geotechnical designs, and applied research in geotechnics. He then continued on to join a civil engineering consultancy group before being engaged in Emas Kiara Marketing Sdn. Bhd. He has been closely involved in the design and construction of numerous maritime projects, including the Port of Tanjung Pelepas, the expansion of Kuantan Port, and the re-construction of berths at North Port, Port Klang. He thus has extensive experience in Geotechnical Engineering design, investigation and research. Ir. Lee Eng Choy has published more than 30 papers in local and international conferences and seminars.

A Review of Soft Ground Improvement by Electro-Osmosis

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ABSTRACT:

Numerous techniques have been used in improving the mechanical properties of soft clay. Such methods have included stone columns, pre-loading with vertical drains, vacuum preloading, lime/cement columns and electro-osmosis, etc. Electro-osmotic treatment of soft clays and the use of Prefabricated Vertical Drains (PVD) with surcharge load have been well documented and proven to accelerate the improvement of soft clay. The coupling of these two technologies has given rise to a fairly new concept of soft ground improvement using EVD or Electro-Osmotic Vertical Drain. This new technology has been driven by the successful manufacture of drainage electrodes in the identical form of PVD, hence enabling very competitive installation method, which has for long been a barrier to electro-osmotic ground improvement. The primary difference between EVD and PVD is that EVD achieves a very much faster rate of treatment.

Two field applications of the electro-osmotic ground treatment technology using EVD have been carried out in Malaysia. This paper describes the field implementation and the effectiveness of the treatment. Marked increase in the undrained shear strength of the soft clay has been observed. Further analysis showed that the improved foundation soil could support a higher embankment load than the traditional PVD ground treatment under similar conditions.

1. INTRODUCTION

Constructions of built facilities have now progressed onto marginal land in many instances, needing improvement of the properties of the soft soils to improve its bearing capacity and load-deformation characteristics. There are currently several methods available to improve the properties of soft clay. Conventionally, geotechnical engineers have applied vertical drains with preloading to improve the properties of soft clay. Other methods include stone columns, cement and lime columns, vacuum preloading, etc. Innovative applications of geosynthetics in ground treatment of soft clay have been progressed by incorporating the electro-osmotic phenomena with the traditional functions of geosynthetic materials. This entails the development of a new range of geosynthetic materials that are electrically conductive. These electrically conductive geosynthetics can be applied in accelerating the consolidation of soft clay. This innovative product takes advantage of effectiveness and simplicity of Prefabricated Vertical Drains (PVD) to provide vertical drainage and incorporating electrical conductivity along core to facilitate electric current flow into the soil, enabling electro-osmotic (EO) ground treatment process. These special materials are referred to as the Electro-Osmotic Vertical Drains (EVD) in this paper.

EO is a process where flow of water between the soil particles is induced under an applied electrical potential. EO based soil improvement is suitable for fine-grained soils (clay) which possess a net surface negative charge. It would have equal charges of cations and anions to maintain electrical neutrality of the clay soil (Mitchell, 1993). Under an applied electrical potential, cations are attracted towards the anode while anions are attracted towards the cathode. As ions move, they drag along water of hydration and the surrounding free water by viscous force. This will induce a flow of pore water towards the cathode. Hence, this will initiate the dewatering and strengthening of clay.

Field applications of EVD recently at two road projects have shown promising results. The comparative influence of EO ground treatment approach and the conventional PVD consolidation approach; especially with regards to increase in undrained shear strength, is the main objective of this study.

2. BENEFITS TO THE END USER

These electro-osmotic vertical drains (EVD) will enhance the improvement of the shear strength and hence reduce the compressibility of the soft clay. A distinct benefit to the end user is the significantly shortened period of treatment to achieve the required shear strength. Typical treatment periods of 3 to 4 months using PVD can now be shortened to about 1 month with the use of EVD.

Furthermore, with EVD system, the need for significant surcharge material is negated.

3. GENERAL PRINCIPLES OF APPLICATION

EVD applies the principle of electro-osmosis in combination with prefabricated vertical drains to improve the engineering properties of soft clay and, in the process, accelerate the consolidation and strengthening of soft clay.

Amongst the many processes occurring during EO treatment is the reduction of pore water in the soil. This reduction in soil moisture indirectly improves the shear strength and compressibility characteristic of the soil. This type of soil improvement is potentially more effective in reducing the water content that conventional PVD which relies on hydraulic gradient created by external loading.

In electro-osmotic ground treatment, the hydraulic conductivity or flow of water under an electric gradient potential is a function of the coefficient of electro-osmotic permeability, k_e (m^2/sV), the electric potential gradient, i_e (V/m), and the cross-sectional area of flow, A (m^2). The relationship is similar to Darcy's Law and may be written as:

$$Q_e = k_e \, i_e \, A \qquad [1]$$

where Q_e = discharge capacity (m^3/s)
k_e = electro-osmotic permeability
i_e = potential gradient
A = cross sectional area

4. Electro-Osmosis Consolidation Phenomena

As discussed above, the electro-osmotic flow produced by an applied electric field is defined by Equation 1. During electro-osmosis, water flows from the anode and discharges at the cathode. However if there is no water to replenish the soil at the anode negative pore pressure, or pore suction, will develop. The magnitude of the negative pore pressures in soils due to the application of a uniform potential field is given by the following equation.

$$u_e = \frac{k_e \gamma_w}{k_h} V \quad [2]$$

where u_e = negative pore water pressure
k_e = electro-osmotic permeability
k^h = hydraulic permeability
γ_w = density of water
V = Voltage applied

The generation of negative pore water pressures causes an increase in the effective stress, σ', in the clay wit the change in total stress, σ;

$$\sigma' = \sigma = u_e \quad [3]$$

As there is an increase in effective stress, the soil particles pack together more tightly resulting in consolidation. For the 1-D case, the increase in effective stress is equivalent to an equivalent surface loading which would generate the same increase in effective stress.

The magnitude of the consolidation depends on the magnitude of the pore water pressure that can be developed at the end of the consolidation process. The rate of consolidation is controlled by the same relationship that applies to consolidation under directly applied (surcharge) loading. The time t for a given degree of consolidation is given by the following relationship.

$$t = \frac{c_v T_v}{L^2} \quad [4]$$

where $$c_v = \frac{k_h}{m_v \gamma_w} \quad [5]$$

and T_v = time factor
c_v = coefficient of consolidation (vertical drainage)
m_v = coefficient of volume compressibility
L = electrode spacing

5. Power Consumption

The efficiency of an electro-osmotic treatment can be evaluated by the power consumption. The power consumption to treat one cubic metre of soil for an hour, P (W/m^3) is a fuction of the electrical conductivity, λ (S/m), the applied electric field intensity, E (V/m), and the current intermittent factor, α.

$$P = \alpha \lambda E^2 \qquad [6]$$

6. FIELD DESIGN

Field design of EVD has to be tailored to the conditions governing each project. In some projects, there are more time constraints whilst in others, the constraints may be on surcharging and less on time. Power supply availability needs to be considered too.

In designing a treatment, the treatment plot shape and size needs to be considered for a treatment program, plot by plot, to complete within a time frame. Computations on the sub-plot would provide an idea of the charging hours and required power supply. These may then be varied to suit the client's schedule or suit available power units.

Typically, the following parameters are given:

Plot Size:	m^2
Thickness of Soft Clay:	m
Electrical Resistivity of Clay:	ohm.cm
Water Content:	%
Soil Electro-Osmotic Permeability:	$cm^2/s.V$

The variable parameters include:

Generator Capacity:	kVA [DC]
EVD Spacing:	_m x _m
Depth of EVD:	m
Equivalent reduction in voids	%

6.1 Voltage Setting

It is recommended from laboratory studies that the voltage gradient be in the region of 0.3V/m to 1.0V/m. Extremely low voltage gradient may unduly prolong the treatment and excessive currents may have adverse results. Also within this range, it is recommended to start with a lower voltage at the generator to allow for increases with progress of treatment or with polarity reversals.

6.2 Soil – Electrode Contact

Voltage losses at soil electrode contact are inevitable. A reduction factor of 1.6 may be used to account of this.

6.3 Current Demand

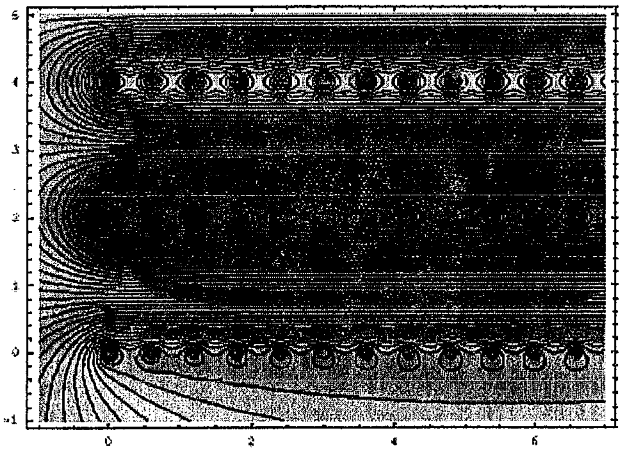
One of the key factors in the design of Electro-Osmotic Consolidation is the Total Resistance [R]. This comprises of the resistance of the soil body in tandem with the spacing grid of the drains (electrodes) and the resistance of the electrodes in the form that they are.

The current demand must be computed to know the required generator capacity and the charging time.

The total resistance [R] in the media comprising of a grid of ***m*** numbers of anodes and ***n*** numbers of cathodes is computed by the sum of the resistances between each pair of electrodes, in the manner:

$$\frac{1}{R} = \frac{1}{R_{[m_1, n_1]}} + \frac{1}{R_{[m_2, n_2]}} + \cdots \frac{1}{R_{[m, n]}}$$

where $R_{[m_i, n_j]}$ is the resistance between the i^{th} anode and the j^{th} cathode.



6.4 Treatment Time

Treatment time [charging hours] may be obtained from the governing equation:

$t = Q \div [\, k_e \times \rho \times I \times f(t_v, c_v)]$ where,

Q	=	cm^3
t	=	time (in secs)
k_e	=	cm/s/V/cm
ρ	=	ohm.cm
I	=	Amps [available into plot]
$f(t_v, c_v)$)		~ 1

Q may be approximated from PVD consolidation method.

7. FIELD APPLICATION LOCATION NO. 1

The project site is located in Dengkil near the Kuala Lumpur International Airport (KLIA), Malaysia. The project is part of a road project and the application of EVD is essentially to improve the foundation soils of an approach embankment to a bridge structure, with embankment heights of about 2.5m.

The site is underlain by soft to very soft alluvial silty clay of up to 10m thickness. Beneath the soft silty clay lies medium dense silty sand. Some decayed wood was encountered at various depths. Ground water table was encountered at approximately 1m below existing ground level (EGL). The initial properties of the soil are as shown in Table 1 below:

Table 1: Initial Soil Properties

Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)	**Moisture Content (%)**
154	**79**	**75**	**132**

The pre-treatment undisturbed shear strength as measured by geonor vane shear apparatus (push-in) ranged from 8.8 kPa at 1m below EGL to 18.5 kPa at 9m below existing ground level. Exceptionally high shear strength of 23 kPa was recorded at a depth of 7m; this was thought to be due to presence of fibrous and decayed wood material. Figure 1 shows the variations of undisturbed shear strength with depth.

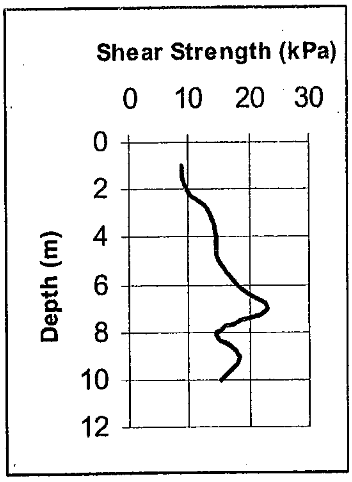


Figure 1: In-situ shear strength (Pre-installation)

7.1 Field Setup

A plot size measuring 20m x 20m was installed with EVD with a rectangular spacing of 1.5m. A total of 198 points were installed in 14 rows, as illustrated in the sketch shown in Figure 2.

A 0.5m thick sand drainage blanket was provided to enable the discharge of ground water to the ditches on the edges. The electrodes at the top of each EVD were connected to an electric power generator supplying DC current. The electric current was applied for 14 days on a 12 hour cycle with intermittent reversals in the polarity of the current.

Upon completion of the charging, the embankment fill was placed to a height of 2.5m. Figure 2 shows a layout of the test plot.

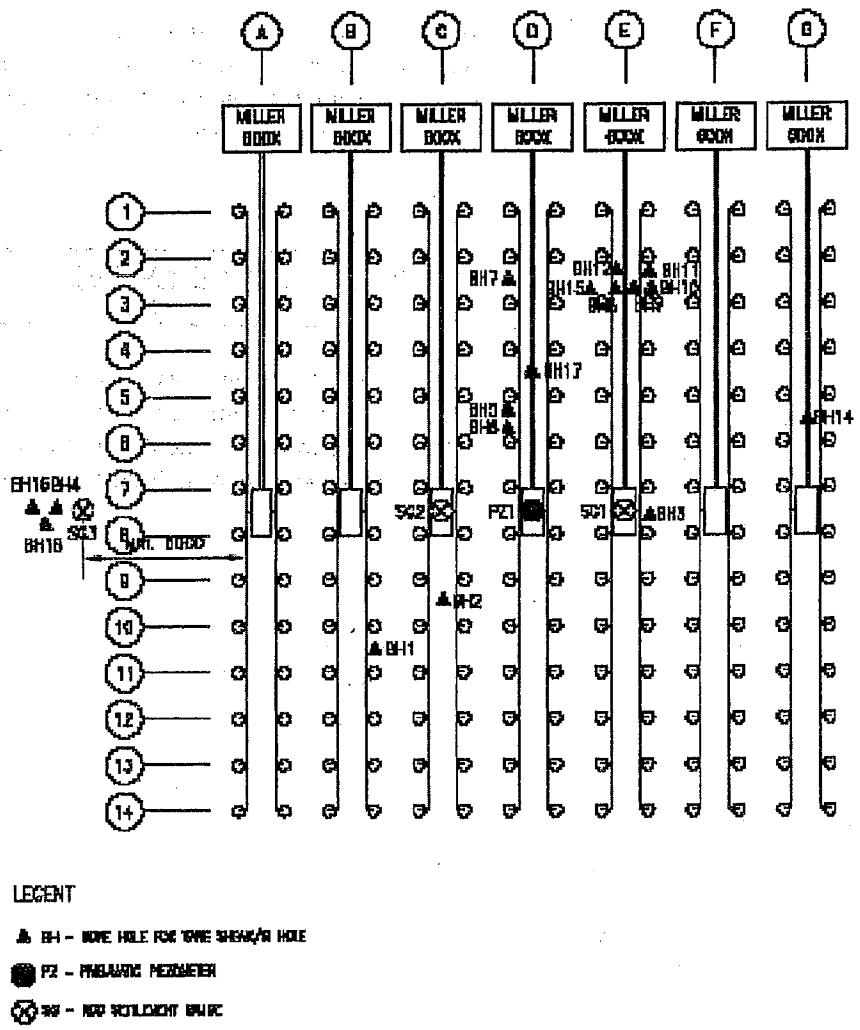


Figure 2: Layout of Charging Pattern, Instrumentation and Testing

The shear strength improvement was closely monitored on a regular basis following the commencement of charging. This was carried out using the penetration field vane (geonor vane) at 1m depth intervals.

7.2 Test Results and Discussion

The most distinct observation of the behavior of the EVD during charging was the constant discharge of water through the cathodes. This was accompanied with gas bubbles being discharged as well. This started to occur about twenty minutes after commencement of charging. It indicated that the electrodes was successfully transferring the electrical charge to the soft ground and the EO treatment as indicated in Section 2.3 above was observed. Figure 3 shows the setup at the site, and Figure 4 shows the discharge of water through the EVD.

There was some slight depression in the area immediately surrounding the anodes. This indicated that consolidation of the soft soil was occurring surrounding the anode, as discussed in Section 2 above.

Figure 3: Setup at site


Figure 4: Discharge of ground water at EVD

Figure 5 below shows the increases in undrained shear strength as measured in the vane shear tests. As indicated in the figure, initially there are increases in shear strength, particularly within the top 5m. This increase became more distinct at 72 days, which show a significant increase in the undrained shear strength. This increase may be attributed to the electro-osmosis treatment of the soil as well as the PVD effect of the EVD during the placement of the embankment load. The percentage of shear strength increase exceeded 100% of the initial values.

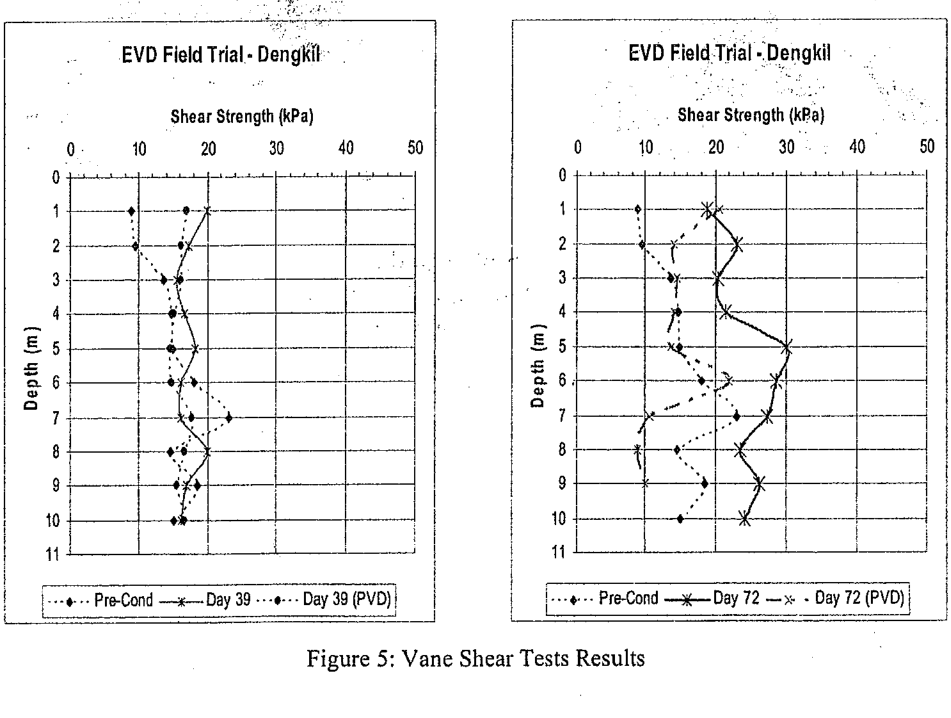


Figure 5: Vane Shear Tests Results

8. FIELD APPLICATION NO. 2

An upgrade of an access road to an oil terminal in Kuching, Sarawak, Malaysia required widening of the road from an existing road width of 8m to a new road width of 16m, constituting a 4-lane single carriageway. This required the widening of 4m on either wide of the existing carriageway. The proposed road structure consists of 1m high embankment fill with approximately 0.5m thick pavement structure. The treatment area measures approximately 560m x 4m on each side of the existing road.

The ground condition at the site consists of very soft to soft organic silty clay up to 15m depth. The ground water table exists at near surface levels. Because of the proximity of the site to the coastline, tidal water variations were observed. To prevent ingress of incoming tidal waters into the site, a bund was built along the external boundaries of the treatment.

The initial site investigation program included field tests such as the vane shear tests. Based on these tests, the undisturbed shear strength of the untreated soil ranges from 5 kPa at the surface to 16 kPa at 9m depth. Site preparation works included placement of a 1m thick granular fill above the soft soils. This served both as a drainage blanket as well as a working platform.

Due to budget constraints, it was decided to improve the soft soil up to a depth of about 6m with EVD. The spacing of the EVD is 1.4m x 1.0m. A total of 3 rows of EVD were installed on both sides of the road widening. Figures 6 and 7 below show the layout of the EVD installed at the site.

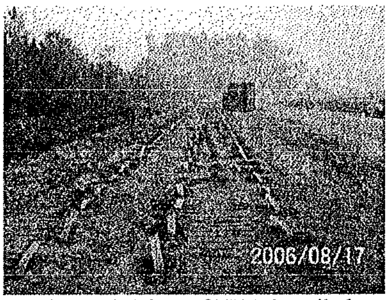


Figure 6: View of EVD installed

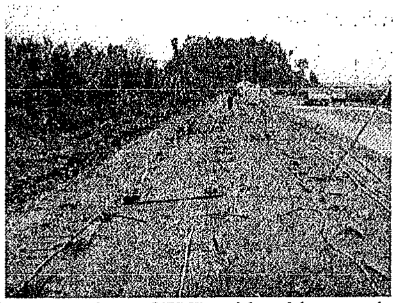


Figure 7: View of EVD with cables attached

The entire section was sub-divided into 8 sub-plots and the electrification period for the entire area took 2 weeks, based on a 20 hour working day. Polarity reversals were carried out daily. During the electrification period, discharge of ground water was observed at the cathode, as shown in Figures 8 and 9 below.

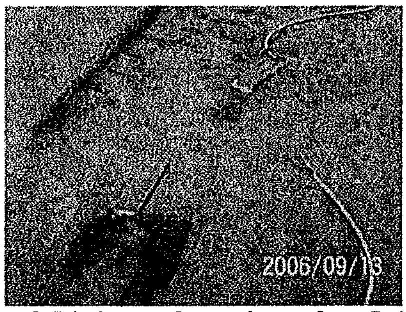


Figure 8: Discharge of groundwater from Cathode

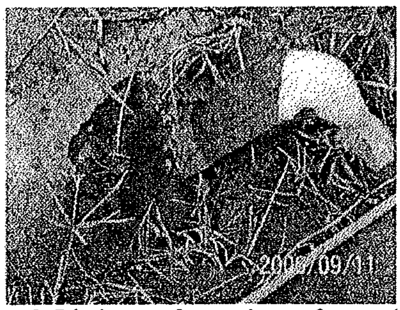


Figure 9: Discharge of groundwater from cathode

Field vane shear tests were carried out a week after completion of the electrification works for the entire treatment area. Figure 10 below shows the results of the shear strength profile. The results show that there is a significant increase in the shear strength profile of the treated soils. Figure 10 below shows the shear strength measurements before and after treatment. The lines shown are computed regression lines. It is clearly evident that significant improvement has been achieved. On the average, shear strength was increased from 9.15 kPa to 31.1 kPa.

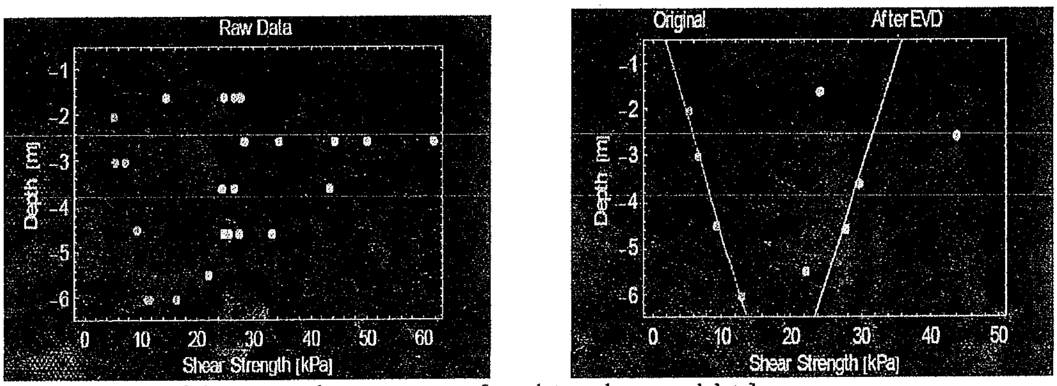


Figure 10: Results of shear strength measurements [raw data and averaged data]

9. CONCLUSION

Two field applications of the EVD have shown significant increases of shear strength within a short period of time. The increased shear strength permits construction of higher embankment heights as compared to conventional PVD ground treatment. This system of ground improvement has proved effective in increasing the shear strength of the soft soils within a short period.

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