GUIDELINES FOR THE DESIGN AND CONSTRUCTION OF GEOSYNTHETIC REINFORCED EMBANKMENTS ON SOFT SUBSOILS

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GUIDELINES FOR THE DESIGN AND CONSTRUCTION OF GEOSYNTHETIC REINFORCED EMBANKMENTS ON SOFT SUBSOILS

1 INTRODUCTION

Construction of embankments over soft subsoil often causes difficulties in both design and construction because of the low shear strength and high compressibility of such soils. Embankments on soft soil may fail due to:

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

Various ground improvement techniques are currently being used by the practicing engineers to safely construct the embankments over soft subsoils, such as;

- Part or full replacement of soft subsoil with soil of better load bearing characteristics
- Construction of the embankment in stages
- Deep stabilization of subsoil using admixtures, such as use of lime columns
- Use of Prefabricated Vertical Drains with preloading
- Use of stone columns to improve the bearing capacity of soft subsoil
- Use of reinforcing elements, Metallic or Polymer, at the base level and above
- Combination of the above

The use of Geosynthetic reinforcements in the construction of embankments on soft subsoils has been adopted by engineers from as far back as 1980's.The behaviour of such embankments over soft soils has been presented in various publications (Humphrey and Holtz (1987), Jewell (1988), Rowe (1997), Leroueil and Rowe (2001), Silvestri (1983), Bonaparte and Christopher (1987).

The present document provides Guidelines for the Design of Embankments Using Geosynthetic Reinforcement at the base. In situations where the use of basal reinforcement has to be combined with other ground improvement methods, reference has to be made to the following publications. In general, these publications also provide guidance for the design and constructions guidelines of embankments.

- HRB SR No. 13 "State of the Art: High Embankments on Soft Ground, Part A – Stage Construction"
- HRB SR No. 14 "State of the Art: High Embankments on Soft Grounds, Part B – Ground Improvement".
- IRC:75 "Guidelines for the Design of High Embankments".
- IS:15284: Part 1 "Code of Practice for Design and Construction for Ground Improvement – Guidelines : Stone Columns"

- IS:15284: Part 2 “Code of Practice for Design and Construction for Ground Improvement – Guidelines : Preconsolidation Using Vertical Drains”
- BS: 8006 “Code of Practice for Strengthened/Reinforced Soils and Other Fills”
- FHWA NHI-95-038 “Geosynthetic Design and Construction Guidelines” Participant Notebook for NHI Course No. 13213

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

The use of reinforcing layer serves the following functions:

- Construction is facilitated as machinery can move easily above the basal mattress for placing the fill
- Basal mattress as above provides good drainage
- The tensile reinforcement provides improvement in the rotational stability of the embankment
- General experience shows a partial control of differential settlement
- Geosynthetic elements are chemically inactive, non-biodegradable and hence durable

The Embankment, Ground Improvement and Drainage Committee (H-4) formed a sub-group comprising S/Shri P.J. Rao, M.S. Verma, P.S. Prasad, Mrs. Minimol Korulla and Ms. Shabana Khan for preparation of draft guidelines. The draft document prepared by the sub-group was discussed by the Committee in series of meetings. The H-4 Committee approved the draft document in its meeting held on 12th September 2011 for placing before HSS Committee. The Highways Specifications & Standards Committee (HSS) approved this document in its meeting held on 23rd September, 2011. The Executive Committee in its meeting held on the 7th October, 2011 approved the document for placing it before the Council. The Council in its 195th meeting held at Lucknow on 3rd November, 2011 approved the draft 'Guidelines for the Design and Construction of Geosynthetic Reinforced Embankments on Soft Subsoils' subject to some modifications in the light of comments offered by the Council members. The modified document duly incorporating comments was approved by the Embankment, Ground Improvement & Drainage Committee (H-4) in its meeting held on 22nd June, 2013 for placing before the HSS Committee. The HSS Committee in its meeting held on 19th July, 2013 approved the "Guidelines for the Design and Construction of Geosynthetic Reinforced Embankments on Soft Subsoils" for publishing.

The Composition of H-4 Committee is as given below:

Mahesh Kumar	--------	Convenor
P.J. Rao	--------	Co-convenor
U.K. Guru Vittal	--------	Member Secretary

2 LOCATION OF SOFT SUBSOILS IN INDIA

India has about 6000 kms long coast line. All along the coast line and in the nearby delta areas, deposits of soft clayey soils are present. These subsoils are generally soft, silt clays which vary from 10 to 30 m in thickness near the coast line. Soft clays cover vast areas of the Gulf of Kutch, river delta areas, shores of the Gulf of Cambay etc. and in general the entire coastal belt.

Typical geotechnical properties of these soft clays, reproduced from HRB SR No. 13 (1994) “State of the Art: High Embankments on Soft Ground, Part A – Stage Construction” are summarized in **Table 1**. The data reveals that these deposits are soft, saturated, plastic and highly compressible in nature. The natural moisture content of these soft soils varies from 60 to 100 per cent. These clayey soils are highly plastic with liquid limit ranging from 50 to 150 per cent. Most of these clayey soils are normally consolidated. The in-situ shear strength generally varies from 7-20 kPa. Other geotechnical properties of relevance in soft ground construction are also included in **Table 1**.

Increased construction activities in the coastal cities and harbours in the soft soil deposit areas pose a challenging task to engineers to meet the needs of rapid development. The problems relate to the construction and maintenance as well as stability of embankments, in the short term, and to the settlement of the ground, in the long term.The stability of an embankment on soft clay depends on interaction of the following parameters (a) Height of embankment (b) Base width of embankment (c) Depth of soft clay (d) Shear strength of soft clay. Depending on the values of the above parameters, failures have occurred where the height of embankment is as low as 1.5 m.

Table 1 : Geotechnical Properties of Soft Clays from Different Parts of India

Properties	Bombay	Outer Harbour Visakhapatnam	Kandla Port Kandla	Willington Island Cochin	Ran of Kutch
Depth of soft clay, m	1.0-20.0	12.0-18.0	12.0-20.0	21.0-28.0	3.0-17.0
Physical Properties					
Liquid Limit, w_L %	30-144	65-97	55-80	105-120	43-73
Plastic Limit, w_P %	18-55	40-45	20-35	40-45	18-45
Natural Water content, w %	40-139	80-90	35-75	65-102	40-80
Plasticity Index I_P	15-89	24-55	20-50	65-75	18-45
Specific Gravity	2.32-2.88	2.65	2.72	2.53-2.60	2.61-2.78
Clay Content	54-100	40-70	30-35	50-65	10-47
Engineering Properties					
Undrained shear strength kN/m^2	15-45	20-40	17-35	5-15	5-20
Natural void Ratio, e_0	1.96-2.81	2.47-2.57	1.1-1.5	2.18-2.30	1.5-2.0
Compression Index	0.37-1.32	0.82-0.88	0.3-0.55	0.65-0.90	0.30-0.56
Coefficient of Consolidation cm^2/sec	$1.23*10^{-4}$	$1.06*10^{-4}$	$8.8*10^{-4}$	$2.54*10^{-4}$	
IS classification	CH-MH	CH-MH	CH-MH	CH-MH	CH-MH

Fine grained soil sare classified on the basis of undrained shear strength as ranging from very soft to hard, as shown in **Table 2.** A broad correlation between the undrained shear strength with SPT and SCPT is also included in the Table. The use of basal reinforcement is most advantageous where soft to very soft soils with undrained shear strength about 50 kPa and less are present.

Table 2 : Classification of Soft Soils Based on Shear Strength

Consistency	Unconfined Compressive Strength (kPa)	SPT Value (N)	SCPT Value (kPa) (acc. to correlation given by Akca (2003))
Hard	>400	>30	>6000
Very stiff	200-400	15-30	3000-6000
Stiff	100-200	8-15	1600-3000
Medium	50-100	4-8	800-1600
Soft	25-50	2-4	400-800
Very Soft	<25	0-2	0-400

3 DESIGN OF GEOSYNTHETIC REINFORCED EMBANKMENT OVER SOFT SUBSOIL

The design of embankment on soft ground is generally governed by the shear resistance of the foundation. The inclusion of Geosynthetic reinforcement at the foundation level (**Fig. 1**) could enhance the performance of embankment as it resists the shear failure in the embankment as well as in the soft soil. Basal reinforcement stabilizes an embankment over soft ground by preventing lateral spreading of the fill, extrusion of the foundation and rotational failure. This stabilizing force is generated in the reinforcement by shear stresses transmitted from the foundation soil and fill, which place the reinforcement in tension. It has also been experienced that the reinforcement can also partially reduce the differential settlement due to better distribution of stress over the soft soil (Rowe and Li (2005)).

The required tensile strength of reinforcement varies with time because of improvement of soft soil shear strength during consolidation. Usually the maximum design working load is experienced during construction. The design life of the reinforcement may be considered in general as equal to the time required to achieve 90 percent consolidation (**Fig. 2**), in most cases.

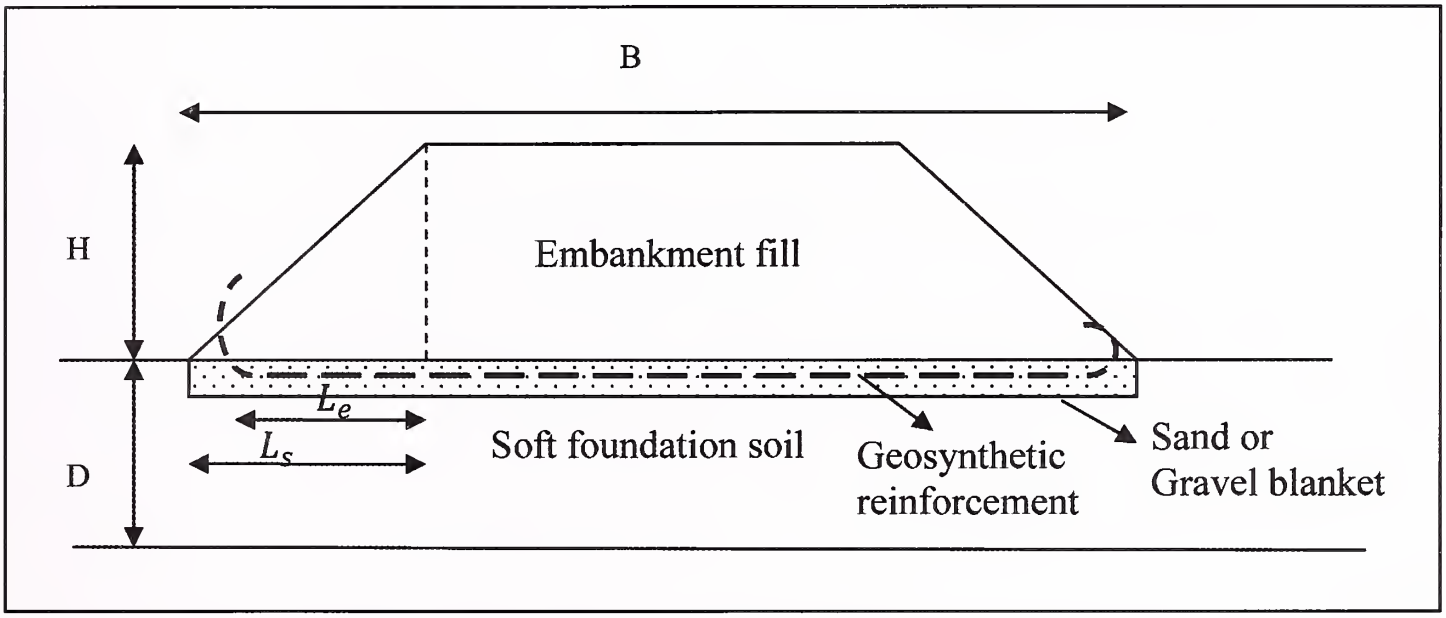


Fig. 1 : Embankment Reinforced with Geosynthetics

Reference can also be made to IRC:75 and HRB SR: 14 for general design procedure of embankments. The design of Geosynthetic reinforced embankment should consider following stability checks:

- Rotational stability of embankment
- Bearing Capacity failure
- Lateral sliding stability

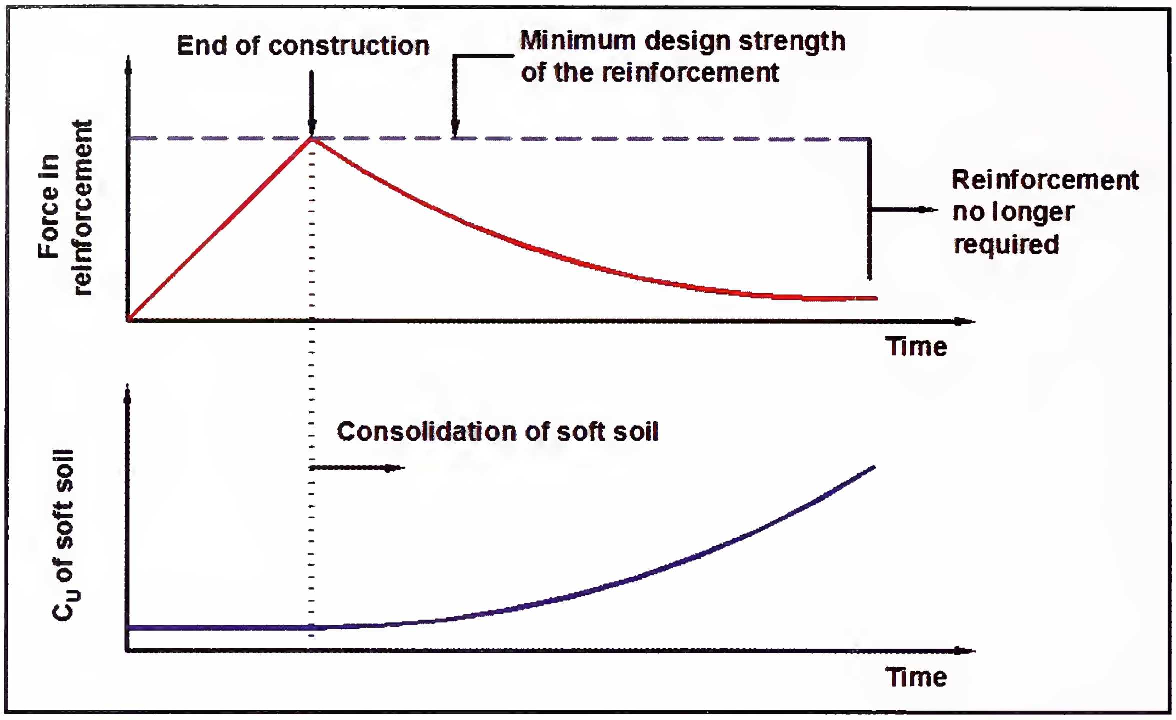


Fig. 2 : Behaviour of Reinforcement and Soft Subsoil with Time

3.1 Rotational Stability

Rotational stability of embankment is evaluated most commonly using method of slices. Bishop's modified method gives the factors of safety that are sufficiently accurate for practical purposes. The factor of safety of the embankment without reinforcement is evaluated first. If the factor of safety is not adequate, then rotational stability check with reinforcement is carried out. The design tensile force required from the reinforcement is thus obtained. From the design tensile force, the ultimate tensile strength requirement of the geosynthetic reinforcing material can be worked out taking into consideration the various reduction factors applicable (Section 3.7). The factor of safety of a reinforced embankment may be expressed as:

$$FS_R = \frac{\sum M_r + T_g R}{\sum M_d} \quad \text{... (1)}$$

Where

ΣM_r = summation of resisting moment of all slices in kN-m/m

ΣM_d = summation of driving moment of all slices in kN-m/m

T_g = tensile force needed in the reinforcement kN/m

R = distance from the center of slip circle to the reinforcement layer in m

Factor of safety for various failure surfaces has to be analyzed and minimum factor of safety should be considered as critical. A validated computer programme or suitable software is generally used to analyze the circular failure surface. A minimum factor of safety of 1.4 may be adopted for rotational stability analysis.

3.2 Bearing Capacity Failure

Reinforced embankment over weak foundations is to be analyzed for safety in bearing capacity. The basal reinforcement acts as a rigid layer and helps to distribute the embankment load on to the subsoil evenly. Also the soft soil strength increases through consolidation which is a time dependent phenomenon.

The following expression can be used to calculate the ultimate bearing capacity of the foundation:

$$Q_{ult} = C_u N_c \qquad \text{... (2)}$$

Where

C_u = Undrained shear strength of the soil in kN/m^2

N_c = Bearing capacity factor

For the case of embankments on soft subsoils, N_c may be calculated from Bonaparte and Christopher (1987) as given below:

$$N_c = 5.14 \text{ for } \frac{B}{D} \leq 2 \qquad \text{... (3)}$$

$$N_c = 4.14 + 0.5\frac{B}{D} \text{ for } \frac{B}{D} \geq 2 \qquad \text{... (4)}$$

Where

B = Width of the bottom of the embankment in meter

D = Depth of the soft soil in meter

The equations 3 and 4 take into account the influence of the B/D ratio on the bearing capacity of the soil. It may be seen as the B/D ratio is higher; the N_c value is more than the Terzaghi's bearing capacity equation. This increase is given as the function of B/D ratio only. The above formula for the calculation of bearing capacity is also been referred in FHWA NHI-95-038 (1998) "Geosynthetic Design and Construction Guidelines" Participant Notebook for NHI Course No. 13213.

It is advisable to have a minimum value of factor of safety as 1.5 for bearing capacity at the end of construction where no other ground improvement is used.

It should be noted that if the bearing capacity of soft soil is not sufficient then ground improvement techniques like stage construction, vertical drains can also be used along with basal reinforcement to increase the bearing capacity of the soft soil. In these methods of ground improvement, increase in strength of subsoil can be achieved within a specified time frame. Hence the factor of safety of 1.25 for bearing capacity at the end of construction may be considered satisfactory. It should be checked that the same increases to 1.5 at the end of consolidation period adopted for the ground improvement technique. This requirement has to be checked for each of the stages adopted for construction. Where stone columns are adopted for ground improvement purpose, the factor of safety in bearing capacity has to be 2.0 at the end of construction to meet the requirements of IS:15284, "Design and Construction for Ground Improvement Guidelines – Part 1 – Stone Columns".

Case studies for the combined use of basal reinforcement with stone columns or vertical drains are also included in the Section 12.

3.3 Lateral Sliding

The lateral sliding stability of the embankment may be checked by considering that the embankment fill should not slide over the reinforcement. The basal reinforcement must resist the outward horizontal thrust of the embankment fill (**Fig. 3 and 4**). The minimum tensile strength required to resist lateral sliding is given below.

$$T_{ls} = (0.5K_a\gamma H^2 - 2c\sqrt{K_a}H) + K_a qH \qquad \text{... (5)}$$

It is advisable to have a minimum value of critical factor of safety as 1.5 for lateral stability. It may be assumed that load in the reinforcement may be assumed to be maximum at the edge of the crest of the embankment.

$$FS_s = \frac{\sum L_e W \alpha \tan(\phi)}{((0.5K_a\gamma H^2 - 2c\sqrt{K_a}H) + K_a qH)} \qquad \text{... (6)}$$

Where

W	=	γ h
γ	=	Density of embankment fills in kN/m^3
h	=	average height of the embankment fill above the reinforcement length L_e in meter
L_s	=	Horizontal projection of the lateral slope (Fig. 1)
L_e	=	Reinforcement Bond Length in meter (Fig. 1)
α	=	Interaction coefficient relating the embankment fill and reinforcement material bond angle, which should be specified in the certification document (e.g. BBA, NTPEP (AASHTO)). In case of absence of certification, it should be limited to 0.5
ϕ	=	Angle of internal friction for embankment fill
c	=	Cohesion of the embankment fill
K_a	=	Active earth pressure coefficient for embankment fill
H	=	Height of embankment in meter
q	=	Surcharge intensity over the embankment in

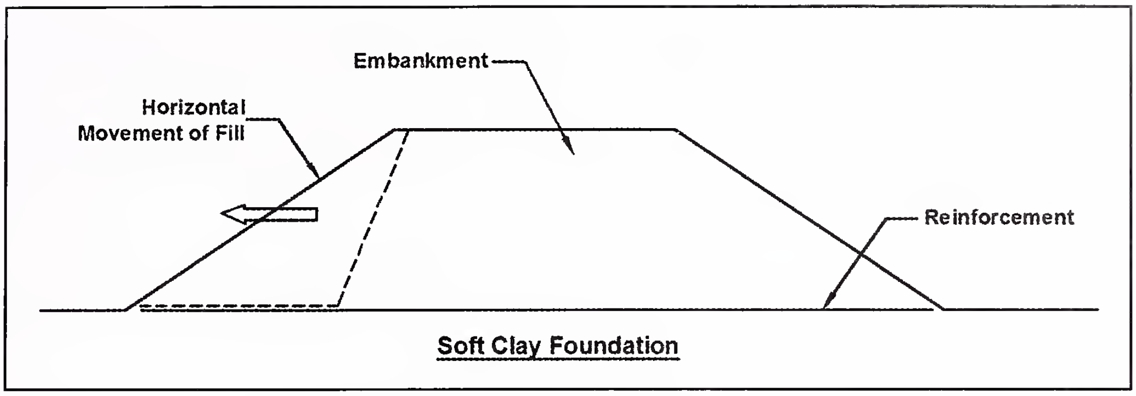


Fig. 3 : Lateral Sliding (BS:8006)

In case of the factor of safety against lateral sliding failure is less, then end anchor blocks may be provided. Anchorage blocks can be made of sandbags, gabion, concrete etc. as shown in **Fig 4**.

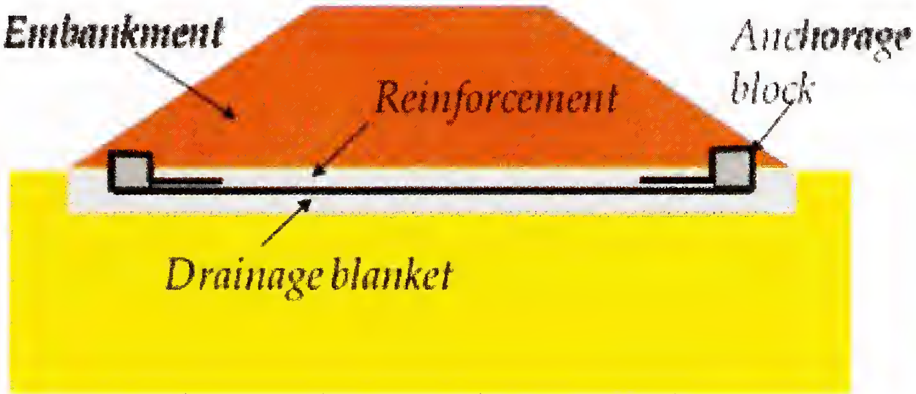


Fig. 4 : Typical Cross Section of Reinforcement Embankments with End Anchorages

3.4 Considerations of Seismic Conditions

The stability of the embankment for its full height shall be checked under seismic conditions as per IRC:75 guidelines. The relevant seismic zone and ground acceleration shall be worked out as per IS:1893(Part 1)-2002.

3.5 Allowable Strains in the Reinforcement

In general BS: 8006 Clause 8.3.2.11 suggests that maximum strain in the reinforcement should not exceed 5% for short term applications and 5% to 10% for long term applications. When choosing the maximum allowable strain of reinforcement, strain compatibility of the reinforcement with the soft soil must be ensured.

3.6 Settlement Analysis

The inclusion of reinforcement alone does not reduce the total settlement of the foundation. Hence conventional settlement analysis needs to be done to calculate the final settlement and should be checked that it is within permissible limits. The tolerable settlements for the embankments may also be referred from IRC:75, Section 4.6. There is a partial improvement in the differential settlement because of the relatively uniform distribution of the stress on the foundation layer due to the inclusion of the reinforcement. If settlement is not within the limits, other ground improvement techniques like stage construction, prefabricated vertical drains or stone columns may need to be adopted.

The settlement of foundation increases the tensile strain in the reinforcement and hence increases the load in reinforcement. The allowable strains in the reinforcement should be checked according to the values given in previous section.

The acceptable settlement value is a serviceability function in the case of pavements on embankments. In this context, due to the settlement of the subsoil, pavement layers including surface experience the stress and surfacing may be repaired at frequent intervals. It is advisable that the high quality bituminous layers for pavement surfacing are not laid in these cases, till the rate of settlement stabilizes.

3.7 Design Tensile Strength of Reinforcement

The design tensile strength shall be calculated by applying the specified reduction factors to the ultimate tensile strength of the reinforcement. Thus the long term tensile strength of the reinforcement, which has an ultimate tensile strength T_{ult}, is obtained as follows:

$$T_{al} = \frac{T_{ult}}{RF_{CR} * RF_{ID} * RF_{W} * RF_{CH} * f_s} \qquad \text{... (7)}$$

Where

T_{al} = tensile strength of the reinforcement (long term strength) in *kN/m*

T_{ult} = ultimate tensile strength (also called short term strength/characteristic strength) from a standard in-isolation wide-width tensile test in *kN/m*

RF_{ID} = Reduction factor for installation damage

RF_{CR} = Reduction factor for creep

RF_{CH} = Reduction factor against chemical/environmental effects

RF_{W} = Reduction factor to allow for weathering during exposure prior to installation or of permanently exposed material

f_s = Factor for the extrapolation of data

The cumulative reduction factor obtained as ($RF_{CH} \times RF_{W}$) is also referred to as reduction factor for durability (RF_D).

All the above reduction factors shall be determined as per ISO/TR – 20432: "Guidelines for Determination of Long Term Strength of Geosynthetics for Soil Reinforcement". It is necessary to consider each item individually and make a conscious decision as to how important it is for the site specific situation. Since ambient temperatures in India are high, creep reduction factors at 30°C and 40°C shall also be provided besides the reduction factors at 20°C.

Table 3 gives typical range of tested values of the reduction factors for geosynthetic made from commonly used polymers. The table is included only to serve as a guideline.

Table 3 : Typical Range of Certified Values of Reduction Factors

Polymer Type	RF_{CR}	RF_{ID}	$RF_{CH} \times RF_W = RF_D$	f_s
PET	1.36-1.59	1-1.31	1-1.3	1-1.37
PVA	1.42	1.06-1.31	1-1.3	1-1.37
HDPE	2.59-2.63	1.02-1.12	1-1.3	1-1.37

Only certified values (refer **Annexure 2**) of reduction factors are acceptable. **Table 4** gives the generally suggested factors of safety for the stability checks at the end of construction. The factors of safety to be suggested are mainly site specific.

Table 4 : Summary of Suggested Factors of Safety at the End of Construction

	With Only Basal Reinforced Mattress	Ground Improvement		Seismic Condition
		PVD'S with Stage Construction (IS:15284-Part 2 (2004))	Stone Columns (IS:15284-Part 1 (2003))	
Rotational stability	1.4	1.4	1.4	1.05
Bearing capacity	1.5	1.25 (at the end of construction of a particular stage) 1.5 (at the end of waiting period specified for the stage)	2.0	
Lateral sliding	1.5	1.5	1.5	1.125

4 NUMERICAL EXAMPLES

A 3 m high embankment with 10 m crest width is to be constructed on a 5 m thick deposit of clay, for a road with a traffic load 20 kN/m^2. The soft clay layer is underlain by medium dense sand. The clay deposit (C_c = 0.3, e_0 = 0.8) has an undrained shear strength of 10.9 kPa and bulk density 18 kN/m^3. The embankment fill is moorum with bulk density 18k N/m^3 and c = 25 *kPa*, φ = 20°.

Let's choose a slope of 3H:1V

Length of side slope L_s = 9 m

Therefore, base width of the embankment, B = 10+2L_s = 28 m

Traffic load was taken as 20 kPa

Without any Reinforcement

Check for Rotational Stability

By conducting analysis of rotational failure along the circular slip surface analysis by Bishop's method, using a validated computer programme, the embankment without any reinforcement shows a factor of safety of 1.21, which is less than 1.3. Since there is a necessity to curtail rotational failure, geogrid basal reinforcement can be adopted.

Check for Bearing Capacity

Bearing capacity of the soft foundation soil = B.C = $C_u N_c$ = 10.9 * 6.94 = 75.65 kPa

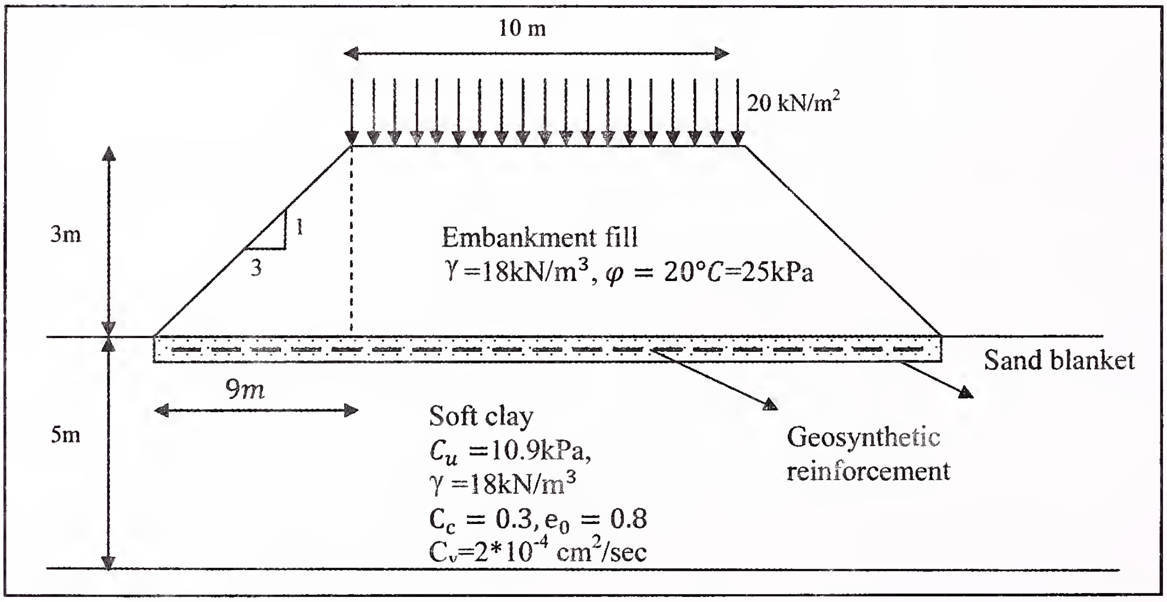
($N_c = 4.14 + 0.5 * \frac{B}{D} = 6.94$)

Without any reinforcement, maximum stress below the embankment

$$P = 18 * 3 + 20 = 74\ kN/m^2$$

Thus factor of safety against bearing capacity failure,

$$FS_{BC} = \frac{75.65}{74} = 1.05 < 1.5$$



With Reinforcement

Let's provide a reinforcement length below side slope L_e of 9 m

Therefore, total reinforcement length, b = 10 + 2 L_e = 28 m

Check for Rotational Stability

Analysis of rotational failure along the circular slip surface analysis by Bishop's method, using a validated computer programme was done. The design tensile strength needed for the reinforcement to improve the factor of safety to 1.3 has been found to be 65 kN/m. OK

Check for Bearing Capacity

As basal reinforcement is provided, it is assumed that the overburden pressure is uniformly distributed over the foundation layer as suggested by FHWA NHI-95-038 (1998) "Geosynthetic Design and Construction Guidelines" Participant Notebook for NHI Course No. 13213, Section 7.5. Thus,

$$P = \frac{(18*3+20)*\left(\frac{10+28}{2}\right)}{28} = 50.21\ kN/m^2$$

Thus factor of safety against bearing capacity failure,

$$FS_{BC} = \frac{75.65}{50.21} = 1.51 > 1.5 \quad \text{OK}$$

Check for Lateral Sliding

$$T_{ls} = (0.5K_a\gamma H^2 - 2c\sqrt{K_a H}) + K_a qH$$

$$T_{ls} = 0.5 * 0.49 * 18 * 3^2 - 2 * 25 * \sqrt{0.49} * 3 + 0.49 * 20 * 3 = -35.91 \text{ kN/m}$$

Since cohesive embankment fill is used, the slope is stable without any reinforcement. Lateral sliding is not the governing criteria in this case.

Tensile Strength of Reinforcement

Long term strength required from rotational stability analysis = 65 kN/m

The reinforcement chosen is bonded geogrid and having an ultimate tensile strength of T_{ult} = 130 kN/m

Incorporating the reduction factors, according to certified product specification

Long term strength of the reinforcement

$$T_{al} = \frac{T_{ult}}{RF_{CR} * RF_{ID} * RF_W * RF_{CH} * f_s} = \frac{130}{1.2*1.05*1.39*1.1*1} = 67.47 \text{ kN/m}$$

Check for Settlement

Initial pressure at the center of the soft clay layer P_0 = 18*2.5 = 45 kN/m^2

Increase in overburden pressure due to embankment fill, $P = \frac{36.64*28}{28+2.5} = 33.64\ kN/m^{2*}$

Settlement after 100% consolidation, $S_f = \frac{C_c}{1+e_o} D \log_{10} \frac{P_0+P}{P_0} = 0.202$ m

Time for the 25% consolidation, $t = \frac{T*H^2}{C_v} = \frac{0.055*\left(\frac{5}{2}\right)^2}{2*10^{-8}} = 0.54$ yrs

Time for the 50% consolidation, $t = \frac{T*H^2}{C_v} = \frac{0.197*\left(\frac{5}{2}\right)^2}{2*10^{-8}}$ = 1.95 yrs

Time for the 90% consolidation, t = $\frac{T*H^2}{C_v} = \frac{0.848*\left(\frac{5}{2}\right)^2}{2*10^{-8}}$ = 8.40 yrs

Settlement is just within limits as specified by IRC-75. However, if the settlement continues to be in the same level, that may cause detrimental performance of the bituminous paved surface. In this case lower quality of the surface may be adopted, but frequent resurfacing will be needed. Alternate methods to accelerate consolidation may also be adopted. The use of basal mattress ensures the stability of the embankment but does not control the settlement.

* refer Braja M. Das : Principles of Foundation Engineering ; Section 5.6 ; Fig. 5.5

Need for Ground Improvement Techniques

In many instances, in construction of embankments on soft subsoils, it is a common experience that the settlement and their time rate of progress are critical factors and hence other ground improvement techniques may be required along with basal reinforcement.

5 SELECTION OF GEOSYNTHETIC AND FILL MATERIAL

5.1 Geosynthetic Material

The geosynthetic, used for basal reinforcement has the main function of reinforcing the soil. The different modes of failure for the geosynthetics used as basal reinforcement are – failure by rupture, failure in bond and failure by excessive strain in the reinforcement. Consequently the requirements to be satisfied by the reinforcement are as follows:

- The reinforcement should have adequate long-term design strength.
- The reinforcement should develop sufficient bond with the soil so as to prevent the sliding of the embankment along the surface of the reinforcement.
- The strains developed in the reinforcement should not exceed the values given in Section 3.5.

To meet the above requirement, a product with high tensile strength, low elongation and low creep is required. Suitable products are polyester geogrids of different types or high density polyethylene (HDPE) geogrids or high strength woven polyester geotextiles. Geocomposite, in which the reinforcing as well as separating and draining materials are bonded together, can be used for basal reinforcement where drainage function is required. A nonwoven geotextile bonded to a geogrid provides in-plane drainage while the geogrid provides tensile reinforcement. Such geotextile-geogrid composites are used for better drainage of low-permeable soils. Since for embankments, length is much greater than width, reinforcement is normally required only in the direction perpendicular to the longitudinal axis of embankment. Hence, reinforcement needs to have high strengths in the transverse (width of embankment) direction, with minimum possible strength in the longitudinal (length of embankment) direction for holding the longitudinal elements and easy handling. Currently products with tensile strength as high as 1300 kN/m are available in the market. Single layer of high strength reinforcement has proven to be more efficient than multiple layers having the same combined total strength (Rowe and Li (2003)). **Fig. 5 to 10** presents different types of geosynthetic basal reinforcements. For information regarding geosynthetics which may be used as reinforcing material, reference may be taken from MORTH Clauses 700 and 3100.

Another form of basal mattress that may be used in embankments is a geocell mattress which is a three dimensional honeycomb structure formed from a series of interlocking cells and it should have adequate tensile strength. However, benefits and design for using geocell mattress basal reinforcement need to be critically evaluated. Biodegradable materials cannot be adopted as basal reinforcement.

Fig. 5 : Bonded Geogrid

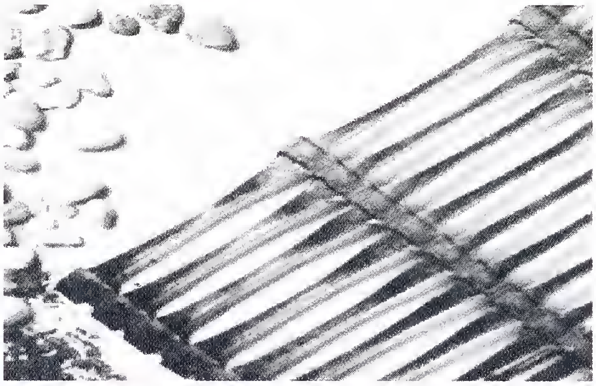


Fig. 6 : Extruded Geogrid

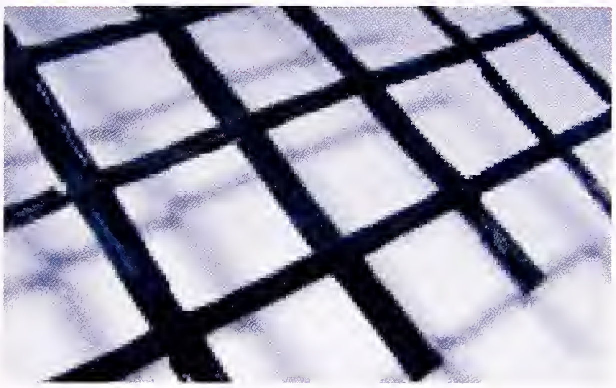


Fig. 7 : Woven Geogrid

Fig. 8 : Woven Geotextile

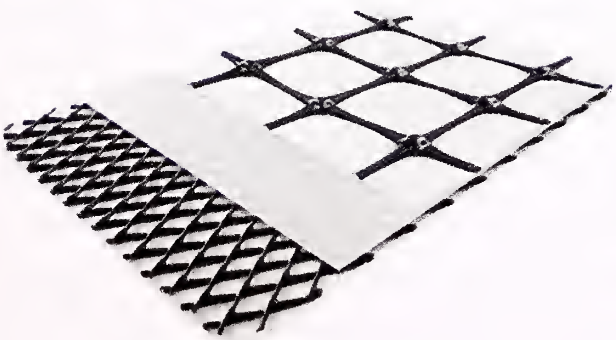


Fig. 9 : Geocomposite

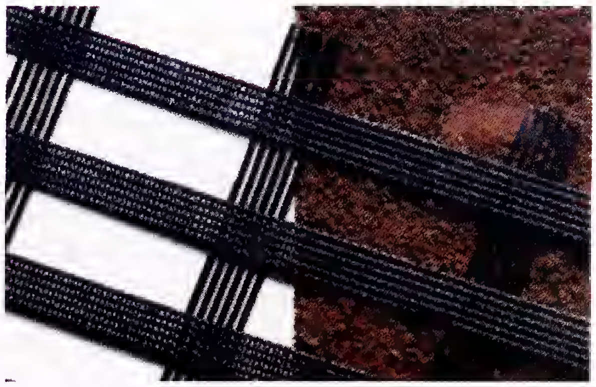


Fig. 10 : Knitted Geogrids

5.2 Strength Requirements

The major properties and its requirement for different types of geosynthetics are discussed in Section 11.

5.3 Drainage Requirements

The geosynthetic should allow free drainage of the soft soils to reduce pore pressure build-up below the embankment. If geotextiles are used for reinforcement, hydraulic properties to be checked are opening size and permeability. The opening size should be such that there is enough resistance against piping at the same time reduces the risk of clogging and should have enough interaction with the embankment fill as well as underlying soft soil. It is recommended that the permeability of the geotextile be at least 10 times that of the underlying soil. The minimum requirement for hydraulic parameters should be according to the specification given in Section 11 for different materials. The regular practice is to give a drainage gravel or sand layer above the soft soil and above which reinforcement is laid.

5.4 Environmental Considerations

The resistance to chemical and biological attacks will mainly depends upon the material with which, geosynthetics are made and the environmental conditions prevailing. Geosynthetics have a very high resistance to chemical and biological attacks. However, in unusual situations such as very low (< 3) or very high (> 9) pH soils, or other unusual chemical environments, such as in industrial areas or near mine or other waste dumps, the chemical compatibility of the polymer(s) in the geosynthetic should be checked to assure that it will retain the design strength at least until the underlying subsoil is strong enough to support the structure without reinforcement.

5.5 Survivability Requirements

The geosynthetics used for reinforcing embankments should have sufficient strength, in addition to the design strength, to withstand the damages occurring during installation. The degree to which this occurs depends on handling of reinforcements prior to installation, the structure of the reinforcements, the nature of the soil in which the reinforcements are installed (mainly the particle size) and the compaction forces applied. This will depend upon the type of installation method adopting for reinforcement, embankment fill construction as well. The survivability requirement for geotextiles is higher compared to geogrids. If the geosynthetic is ripped, punctured, or torn during construction, support strength for the embankment structure will be reduced and failure could result. If the installation damage factor for a particular reinforcement is less, naturally, the short term strength requirement will also be less. Reinforcement manufacturers publish the certified reduction factors for specified ranges of fill types and methods of compaction. When the nature or grading of the fill, or the compaction method fall outside these ranges, these factors should be found out according to ISO/TR – 20432 or from accredited testing laboratories. Minimum strength requirements for puncture, grab, tear and burst, which are very much critical for geotextile, are given in Section 11.

5.6 Fill Requirements

Fill material should conform to the requirement IRC:36 “Recommended Practice for the Construction of Earth Embankments for Road Works” and the relevant Specification of the Ministry of Road Transport and Highways Section 305.

6 SUBSOIL INVESTIGATION AND TESTING

High quality subsoil investigation and testing shall form an essential part of any project involving design and construction of embankments on soft ground. It is recommended that one borehole shall be taken for every 100 m length of embankment. The borehole shall extend to the full depth of the soft soil layer. Shear strength variation with depth shall be established in each borehole. This may be done by in-situ vane shear tests or by collecting undisturbed samples and testing for unconfined compressive strength in the laboratory. These test methods provide shear strength values having high reliability. Static cone penetration test (SCPT) may also be used, however, it is desirable that a correlation is first established at the particular site between SCPT and vane shear values. The use of Standard Penetration Test (SPT) in soft cohesive soils may only be used sparingly, as they do not reflect sufficiently accurately the narrow range of strength increase in soft clayey soils which is the case in the current document. Compressibility characteristics (coefficient of consolidation (C_V) and compression index (C_C)) as well as the liquid limit, plastic limit, natural moisture content, void ratio etc. of the soft subsoil shall also be determined.

Where stage construction is adopted, the increase in shear strength specified for each stage has to be checked in the field, by vane shear tests or laboratory tests on undisturbed samples. Work on next stage filling can be permitted only after it is ascertained that the strength gain needed for building the next stage has been reached. Formulae for determining the increase in shear strength may be referred from HRB SR No. 13 “State of the Art: High Embankments on Soft Ground, Part A – Stage Construction”. However, the actual values have to be checked in field. It would be desirable that laboratory tests on increase in shear strength due to consolidation under different stage loads (preloads) are carried out and these values may be used for design calculations.

7 INSTRUMENTATION AND MONITORING

As may be noted from the Section 3, the embankments on soft ground are designed with low initial factor of safety, since the soft subsoil shear strength is expected to increase with time. Because of the low initial factor of safety, monitoring of the behaviour of the embankment is essential. The parameters to be monitored are:

i) Increase in shear strength (Section 6)

ii) Decrease or change in pore water pressure of the soft subsoil, using piezometers

iii) Settlements, by installing settlement gauges at the center line of the embankment as well as near the shoulder. These gauges shall also be installed at different depths in the soft subsoils.

iv) It is desirable that in cases where large lateral displacements are anticipated, inclinometers are installed at the toe

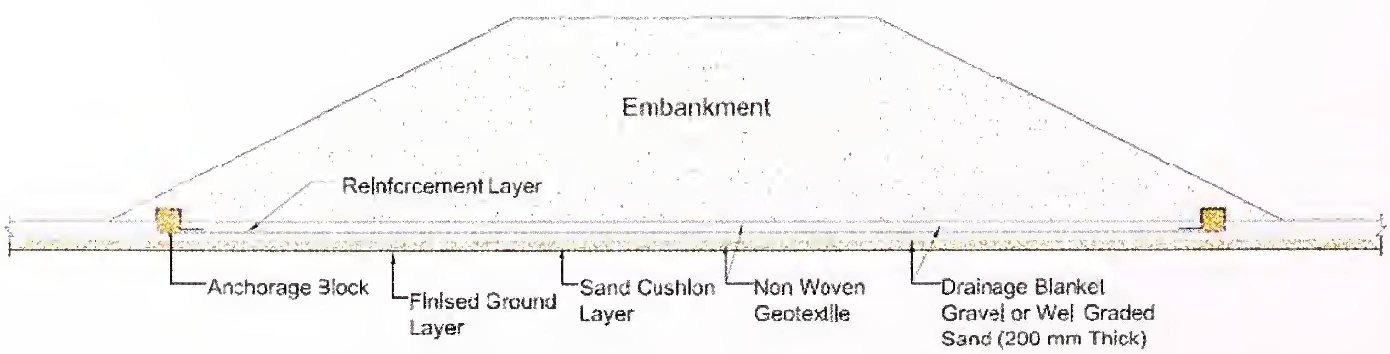
A complete instrumentation programme, including location of installations, monitoring programme etc. shall be worked out as a part of design of the embankment. Trained personnel shall be provided for data collection and interpretation. For details of instrumentation, reference can be made to IRC:75 (1979) and HRB SR: 14 (1995).

8 BASAL MATTRESS WITH GEOSYNTHETIC REINFORCEMENT LAYERING SEQUENCE (Fig. 11)

The basal layers of the embankments generally consist of the following components:

i) Prepared ground level
ii) Sand cushion layer (if required)
iii) Nonwoven geotextile separating layer
iv) Drainage blanket (gravel or well graded sand of at least 200 mm thickness)
v) Geosynthetic reinforcement
vi) Drainage blanket (gravel or well graded sand of at least 200 mm thickness)
vii) Nonwoven geotextile separating layer (optional cases - if cohesive embankment fill is used)
viii) Embankment fill

The geosynthetic reinforcement is placed in between the two layers of gravel or well graded sand which serves as the drainage layer as well as the frictional layer that fully mobilizes the tensile strength of the reinforcement. The geosynthetic reinforcement along with the granular fill acts as a stiff supporting layer between the embankment and the soft soil.



Drainage Blanket should extend beyond the toe of the embandment on either side for a distance of 50 cm (minimum)

Fig. 11 : Geosynthetic Basal Mattress Layering Sequence

9 CONSTRUCTION ASPECTS

The construction procedures for reinforced embankments on soft foundations require special attention to the difficulties that can arise from site access, site clearance and fill placement. Improper fill placement can lead to geosynthetic damage, non-uniform settlements and embankment failure. Construction rate should be preferably slow enough to ensure that there is enough dissipation of excess pore pressure. The following aspects shall be followed for efficient handling, safe storage and placement of geosynthetic materials.

9.1 Site Preparation

- Clearing and grubbing should conform to the requirement of 'Specification for Road and Bridge Works, MORTH'.
- The top foundation soil shall be free from undulations and prepared to the level as indicated in the construction drawings or as directed by the Engineer.
- Materials causing damage to geosynthetic like debris etc. should be removed from site.
- Ground should be excavated or brought to the desired level as per approved drawings.
- A drainage (fine to well graded sand or gravel) layer of thickness shown in the approved drawings shall be placed.

9.2 Reinforcement Storage

- During storage, reinforcement rolls shall be elevated off the ground and adequately covered to protect them from site construction damage, precipitation, extended ultraviolet radiation including sunlight, chemicals that are strong acids or strong bases, flames including welding sparks, and any other environmental condition that may damage the physical property values of the reinforcement.
- Proper illumination shall be provided at important places to facilitate loading and unloading operations of materials during the night.
- Special instructions for the handling of each component shall be provided to the Stores department.
- The equipment used for loading and unloading shall be as per the manufacture's guidelines for the particular component. The arrangements for the same shall be made at the yard and project site.
- The open storage area in the yard shall be levelled properly and capable of taking the load of the material stacks. The ground shall be developed such that even during rainy season the material stacks are accessible. All the materials shall be kept on wooden planks or pipes to avoid direct contact with

the ground. The wooden planks or pipes shall also ensure that the stacks remain stable.

- Fire can inflict huge loss and damages to the materials stored in the yard. Therefore, adequate numbers of fire extinguishers shall be available inside the warehouse. The entire storage yard shall be considered a 'No Smoking' zone.
- Ensure physical verification of the whole consignment as per packing details & check for any visual damage. The damaged or defective materials shall be immediately segregated and shifted to a separate area dedicated for such storage.
- Storing of material should be done by item category, size and grade. Adequate space (1 m) shall be made available between the stacks; sufficient enough to allow movement of store/project staff for routine inspection.
- Routine activities such as sweeping, cleaning, dusting shall be undertaken and daily check list to be filled shall be kept at each covered area.
- Water and moisture acts as catalyst for deterioration of material. Storage place shall be free from any water leakages. Security arrangements with a gate for restriction of unauthorized entry are also recommended.
- Entrances, exits, pathways shall be kept clean and free from material. Height and weight restrictions should be considered while stacking material and damage due to tear and shear.
- Provide proper drainage system and the yard shall be cleared of debris.
- The storage yard shall be provided with a permanent constructed or fabricated mobile ramp.

9.3 Placing of Reinforcement

- The reinforcement shall be laid at the proper elevation and alignment as shown on the construction drawings. It shall be placed with main strength direction oriented perpendicular to centerline of the embankment.
- The reinforcement shall be installed in accordance with the installation guidelines provided by the manufacturer or as directed by the Engineer. It may be temporarily secured in place with ties, staples, pins, sand bags as required by fill properties, fill placement procedures or weather conditions or as directed by the Engineer.
- If geotextiles or low strength geogrids are used, a cushion layer of sand must be given for minimizing installation damages. Sand layer shall be compacted to specified design modified proctor density. Above this layer as per design at a specified distance a layer of reinforcement shall be placed.

- Care should be taken in the handling, lifting and positioning of reinforcement rolls. If, the weight of the rolls is such that mechanical lifting arrangements are necessary, the use of a lifting beam is recommended.
- Slack/wrinkles in the reinforcement layer shall be removed manually. Direct movement of vehicles on the reinforcement shall be prevented.
- The reinforcement should not be exposed to sunlight for more than the maximum duration permitted in the approved drawing/installation methodology. In the absence of any specific provision in the drawings/ installation methodology, the reinforcement should be covered within a day of installation.

9.4 Reinforcement Jointing

- Required overlapping length must be detailed in the drawing by the designer.
- An overlap of 300 mm or as indicated by Engineer shall be provided between the adjacent rolls. There should be no joints or seams along the principal strength direction of the basal reinforcement. However, if unavoidable, the overlap should have sufficient anchorage length so that overlaps are strong enough to carry design loads.

9.5 End Anchorages

- The roll should be unwound a small amount by pushing the roll in the direction of the reinforcement run. The base end of the reinforcement now exposed should be secured by weighting or pinning it to the formation. When the roll is completely unwound, the free end of the reinforcement should be hand tightened and secured by weighting or pinning.
- Where reinforcement is to be anchored by passing it round an anchorage block (thrust block), such as a gabion basket, and back on itself, then the reinforcement should be pulled tight around the block and secured by pinning or weighting until fill around the block has been placed. Fill should not be placed on the return length of reinforcement until the length around the anchorage block has been secured by pulling it tight.

9.6 Fill Considerations

- Reinforcement layer should be covered with well graded sand having angle of internal friction as per approved drawings.
- All filling shall be done in layers of 200 mm thickness. If ground water table is encountered proper dewatering arrangement shall be arranged.
- Fill in immediate contact with the reinforcement should be placed and spread in the longitudinal direction of the reinforcement only.

- Under no circumstances should tracked vehicles be allowed to traffic over the laid, unprotected reinforcement.
- The sequence of fill placement should be considered with care, particularly over very poor soft soil where bearing capacity can be exceeded with small loadings.
- Two placement techniques have been used successfully (Holtz 1990)- For less severe foundation conditions the technique is to place the fill symmetrically from the center outward in an inverted U type construction (**Fig. 12**).
- The second technique involves the initial construction end dump fill along edges of geosynthetic to form access roads. After access roads, the fill should be spread between each toe and placement should be parallel to the alignment and symmetrical from the toe inward toward the center to maintain a U shape (**Fig. 13**).
- Use lightweight dozers and/or graders to spread the fill.

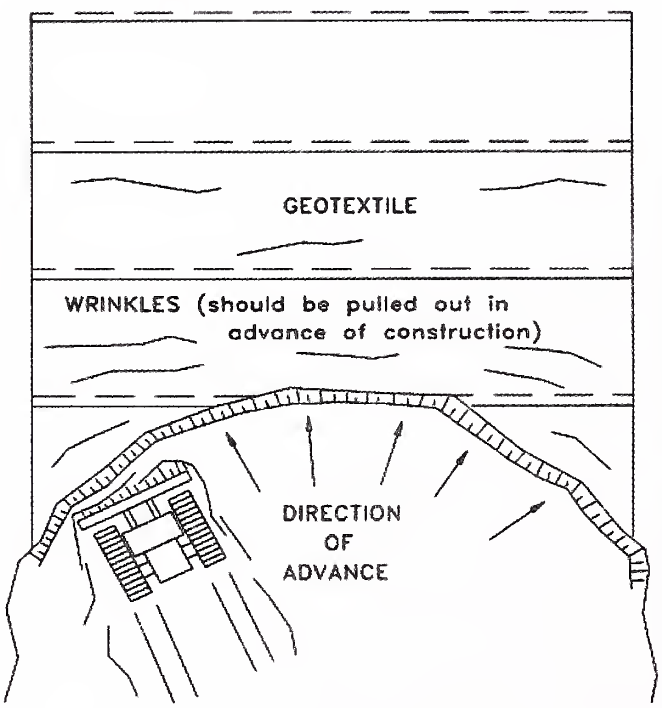


Fig. 12 : Inverted U Construction
(FHWA NHI-95-038, BS:8006)

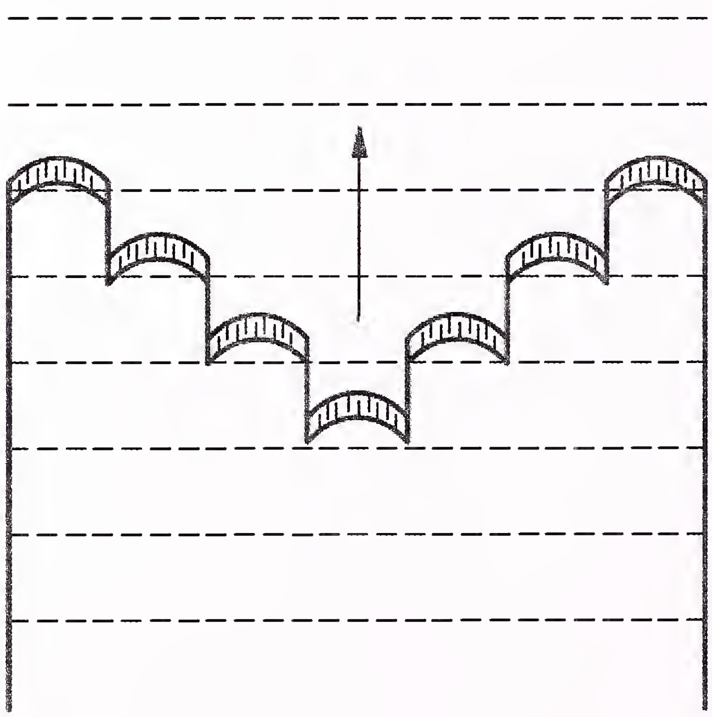


Fig. 13 : U Shaped Construction
(FHWA NHI-95-038, BS:8006)

10 EMBANKMENT WIDENING

Where embankment widening is to be carried out using geosynthetic reinforcement, relative settlement of the existing and new fill plays a major role. The reinforcing element has to be anchored into the existing embankment. For this purpose, part of the embankment may need to be excavated, reinforcing elements are then placed and backfilled. The traffic flow should also be considered during widening. The traffic flow through the existing fill must be completely avoided or only through single lane, can be adopted, depending upon the design. There will be a substantial difference in the settlement of existing and new embankment, the existing fill also settles because of the influence of the new, adjacent fill loads on their foundation soils. Usually for soft soils with high compressibility, sufficient consideration during design and construction shall be made for reducing the settlement of new fill. Curtailment of the settlement using piles along with reinforcement may be preferred in case of highly compressible soft subsoils. In moderately compressible soft soils, effective methods to accelerate the settlement, like prefabricated vertical drains may be adopted.

11 TECHNICAL SPECIFICATIONS FOR GEOSYNTHETICS

a) Specifications of Geogrids

1) **Material** – Geogrid is a planar polymeric structure consisting of a regular open network of connected tensile elements, which may be linked by extrusion, bonding, weaving or knitting, whose openings are larger than the constituents and used in contact with soil or any other geotechnical material in civil engineering applications.

2) **Ultraviolet and Chemical Inertness** – The geogrid shall be UV stabilized (ASTM D4355) and should be inert to all chemicals existing in soil $4 \leq pH \leq 9$ and suitable for applications in soil with pH up to 11 if necessary with appropriate reduction factors.

3) **Specifications** – Geogrids shall be dimensionally stable and able to retain their geometry under manufacture, transport, and installation. The geogrid shall be selected depending upon the project requirement. Manufacturers shall submit the certified values of the properties of the geogrids required for use as reinforcement. The list of such properties is given in **Table 5, 6 and 7**. The ultimate tensile strength values indicated by the manufacturers needs to be reduced to long term strength for design purpose by applying suitable Reduction Factors (RF) as explained in Section 3.7. The selection of the geogrid need to be strictly based on the long term design strength values for a specific design life. The geogrid shall fulfill the following requirements:

    a) Shall have ISO (ISO-9001) or CE certification for manufacturing process and quality control

    b) Manufacturers shall provide certified values for all reduction factors from a competent independent agency such as British Board of Accreditation (BBA) or National Transportation Product Evaluation Programme (NTPEP) or shall furnish test reports from an independent laboratory with valid accreditation from Geosynthetic Accreditation Institute – Laboratory Accreditation Programme (GAI-LAP) or United Kingdom Accreditation Service (UKAS) for all the required tests to substantiate all the reduction factors.

Table 5 : List of Certified Values of Properties of Bonded Geogrids for Embankment Reinforcement to be Submitted by the Manufacturer (Fig: 5)

Property		Unit
Ultimate Tensile Strength (UTS)		kN/m
Typical Strain at UTS		%
Tensile Strength at	2% strain	kN/m
	5% strain	kN/m
Single Strip Tensile Strength		kN
Single Strip Width		mm
Roll Length		m
Roll Width		m
Long term Design strength 60 yr design life (after applying all the 4 reduction factors as per Section 3.7)		kN/m
Long term Design strength 120 yr design life (after applying all the 4 reduction factors as per Section 3.7)		kN/m

Table 6 : List of Certified Values of Properties of Extruded Geogrids for Embankment Reinforcement to be Submitted by the Manufacturer (Fig: 6)

Property		Unit
Ultimate Tensile Strength (UTS)		kN/m
Typical Strain at UTS		%
Tensile Strength at	2% strain	kN/m
	5% strain	kN/m
Carbon Black Content		%
Roll Length		M
Roll Width		M
Long Term Design strength 60 yr Design life (after applying all the 4 reduction factors as per Section 3.7)		kN/m
Long Term Design Strength 120 yr Design life (after applying all the 4 reduction factors as per Section 3.7)		kN/m

Table 7 : List of Certified Values of Properties of Woven/Knitted Geogrids for Embankment Reinforcement to be Submitted by the Manufacturer (Fig. 7)

Property		Unit
Tensile Strength – MD		kN/m
Strain at max. Strength – MD		%
Tensile Strength (MD) at	2% strain	kN/m
	5% strain	kN/m
Tensile Strength – CMD (Required if geogrid is used as bidirectional)		kN/m
Strain at max. Strength – CMD (Required if geogrid is used as bidirectional)		kN/m
Roll Length		m
Roll Width		m
Long Term Design Strength 60 yr Design life (after applying all the 4 reduction factors as per Section 3.7)		kN/m
Long Term Design Strength 120 yr Design life (after applying all the 4 reduction factors as per Section 3.7)		kN/m

b) Specifications of Geotextile

1) **Material** – Geotextiles are planer structures manufactured from polyester or polypropylene multifilament yarns by weaving in the warp and the weft direction (woven geotextile).

2) **Ultraviolet and Chemical Inertness** – The geotextile must have a high resistance to ultraviolet degradation (ASTM D4355) and to biological & chemical environments normally found in soil.

3) **Specifications** – Geotextile shall be dimensionally stable and able to retain their geometry under manufacture, transport, and installation. The geotextile shall be selected depending upon the project requirement. Manufacturers shall submit the certified values of the properties of geotextiles which are required for design as reinforcement.

The list of such properties is given in **Table 8**. The Ultimate tensile strength values submitted by the manufacturer needs to be reduced to obtain long term strength for design purpose by applying suitable Reduction Factors (RF) as explained in Section 3.7. The selection of the geotextile need to be strictly based on the long term design strength values for a specific design life. The geotextile shall fulfill the following requirements:

a) Shall have ISO (EN ISO-9001) or CE certification for manufacturing process and quality control

b) Manufacturers shall provide certified values for all reduction factors from a competent independent agency like British Board of Accreditation (BBA) or National Transportation Product Evaluation Programme (NTPEP) or shall furnish test reports from an independent laboratory with valid accreditation from Geosynthetic Accreditation Institute – Laboratory Accreditation Programme (GAI-LAP) or United Kingdom Accreditation Service (UKAS) for all the required tests to substantiate all the reduction factors.

c) Specifications of Geocomposite

1) **Material** - In Geocomposite, the reinforcing as well as separating or draining material are bonded together. A needle punched non-woven geotextile bonded to a geogrid provides in-plane drainage while the geogrid provides tensile reinforcement. Such geotextile-geogrid composites are used for better drainage of low-permeable soils.

2) **Ultraviolet and Chemical Inertness** - The geocomposite must have a high resistance to ultraviolet degradation (ASTM D4355) and to biological and chemical environments normally found in soil.

Table 8 : List of Certified Values of Properties of Woven Geotextile for Embankment Reinforcement to be Submitted by the Manufacturer (Fig: 8)

Property	Unit
Tensile Strength - MD	kN/m
Strain at Max. Strength - MD	%
Tensile Strength - CMD	kN/m
Strain at Max. Strength - CMD	%
Puncture Strength	N
Wide Width Tensile Strength (MD)	kN/m
Wide Width Tensile Strength (CMD)	kN/m
Trapezoidal Tear Strength	N
Apparent Opening Size	mm
Permittivity	Sec^{-1}
Roll Length	m
Roll Width	m
Ultraviolet Stability at 500 h, Retained Strength	%
Long Term Design Strength 60 yr Design life (after applying all the 4 reduction factors as per Section 3.7)	kN/m
Long Term Design Strength 120 yr Design life (after applying all the 4 reduction factors as per Section 3.7)	kN/m

3) **Specifications** – Geocomposite shall be dimensionally stable and able to retain their geometry under manufacture, transport, and installation. The geocomposite shall be selected depending upon the project requirement. Manufacturers shall submit the certified values of the properties of geocomposite which are required for design as reinforcement. The list of such properties is given in **Table 9.** The Ultimate tensile strength values provided by the manufacturer needs to be reduced to long term strength for design purpose by applying suitable Reduction Factors (RF) as explained in Section 3.7. The selection of the geocomposite need to be strictly based on the long term design strength values for a specific design life.

The geocomposite shall fulfill the following requirements:

a) Shall have ISO (EN ISO-9001) or CE certification for manufacturing process and quality control

b) Manufacturers shall provide certified values for all reduction factors from a competent independent agency like British Board of Accreditation (BBA) or National Transportation Product Evaluation Programme (NTPEP) or shall furnish test reports from an independent laboratory with valid accreditation from Geosynthetic Accreditation Institute – Laboratory Accreditation Programme (GAI-LAP) or United Kingdom Accreditation Service (UKAS) for all the required tests to substantiate all the reduction factors.

Table 9 : List of Certified Values of Properties of Geocomposite for Embankment Reinforcement to be Submitted by the Manufacturer (Fig: 9)

<table>
<tr><th colspan="2">Property</th><th>Unit</th></tr>
<tr><td colspan="2">Tensile Strength - MD (Geogrid)</td><td>kN/m</td></tr>
<tr><td colspan="2">Strain at max. Strength-MD (Geogrid)</td><td>%</td></tr>
<tr><td colspan="2">Tensile Strength - CMD (Geogrid)</td><td>kN/m</td></tr>
<tr><td colspan="2">Strain at Max. Strength-CMD (Geogrid)</td><td>%</td></tr>
<tr><td rowspan="2">Tensile Strength(Geogrid) at</td><td>2% strain</td><td>kN/m</td></tr>
<tr><td>5% strain</td><td>kN/m</td></tr>
<tr><td colspan="2">Single Strip Tensile Strength (geogrid)</td><td>kN</td></tr>
<tr><td colspan="2">Single Strip Width (geogrid)</td><td>mm</td></tr>
<tr><td colspan="2">Puncture Strength (geotextile)</td><td>N</td></tr>
<tr><td colspan="2">Apparent opening size (geotextile)</td><td>mm</td></tr>
<tr><td colspan="2">Permittivity(geotextile)</td><td>Sec^{-1}</td></tr>
<tr><td colspan="2">Roll Length</td><td>m</td></tr>
<tr><td colspan="2">Roll Width</td><td>m</td></tr>
<tr><td colspan="2">Long Term Design Strength 60 yr Design life (after applying all the 4 reduction factors as per Section 3.7)</td><td>kN/m</td></tr>
<tr><td colspan="2">Long Term Design Strength 120 yr Design life (after applying all the 4 reduction factors as per Section 3.7)</td><td>kN/m</td></tr>
</table>

d) Specifications of Separation Layer

1) **Material** - Generally a non-woven geotextile layer is used as the separation layer between the soft subsoil and drainage layer of sand or gravel. Nonwoven geotextile (**Fig. 14**) is a planar and essentially random textile structure produced by bonding, interlocking of fibers, or both, accomplished by mechanical, chemical, thermal or solvent means and combination of thereof. Mechanically bonded nonwoven geotextile made of high tenacity polypropylene staple fibers or thermally bonded nonwoven geotextile made of polypropylene and polyethylene are the most common types.

Fig. 14 : Nonwoven Geotextile

2) **Specification** - Non-woven geotextile shall be dimensionally stable and able to retain their geometry under manufacture, transport, and installation. The range of typical values of most geotextiles is from 100 to 1000 gm/m^2. The functions and properties of nonwoven geotextile as separating layer can be referred form Table 4.1 and 4.2 of IRC:SP:59-2002 – "Guidelines for Use of Geotextiles in Road Pavements and Associated Works". The selection of type of non-woven geotextile (Type I, II or III) depends on project requirement and engineer's jurisdiction. Manufacturers shall submit the values of the properties of geotextile which are required for design as separator. These values from the tests which are conducted from accredited laboratories. The list of such properties is given in **Table 10**. A typical BOQ is added as **Annexure 1** for reference where description of reinforcement alternatives, separation layer and drainage layer are included.

Table 10 : List of Tested Values of Properties of Non-Woven Geotextile for Separation to be Submitted by the Manufacturer (Fig: 14)

Property	Unit
Grab Tensile Strength	N
Grab Elongation	%
Puncture Strength	N
Trapezoidal Tear Strength	N
Apparent Opening Size	mm
Permittivity	Sec^{-1}
Water Flow	l/min/m^2
Ultraviolet Stability at 500h, Retained Strength	%
Roll Length	m
Roll Width	m

12 CASE STUDIES

a) Southern Transport Development Project Seethawaka Industrial Park, Avissawella, Srilanka

Southern Transport Development Project (STDP) was Sri Lanka's first major expressway project with a length of 126 km stretching from Colombo to Matara. Part of the project at section JBIC from Dodangoda to Kurundugahahetekma **(Fig. 15)** required construction of embankment over soft foundation soil in a construction period of 130 days. The height of the embankment varies from 4.0 m to 10.5 m. Soil investigation had further shown that the environment in which the embankment is to be constructed is acidic.

Preliminary design analysis showed that the soft foundation soil could not support the embankment without treatment at the base of the embankment. High strength geogrids **(Fig. 16 and 17)** with ultimate tensile strengths of 150 kN/m^2 and 200 kN/m^2 were used for basal reinforcement. The geogrid was made from high molecular weight, high tenacity polyester multifilament yarns. The yarns were protected with a polymeric coating making it suitable for acidic soil.

Fig. 15 : Seethawaka Industrial Park, Srilanka Site Before Construction

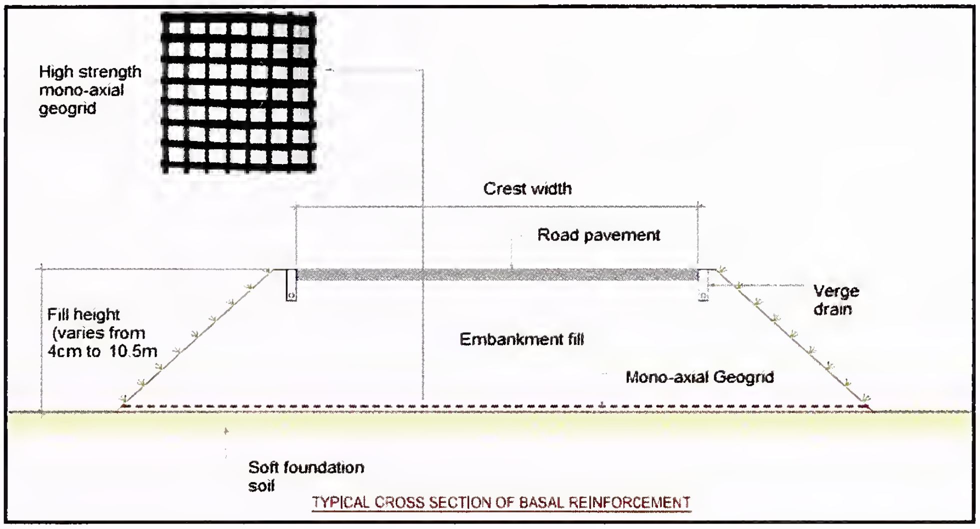


Fig. 16 : Typical Cross Section of Basal Reinforcement Used for Southern Transport Project Srilanka

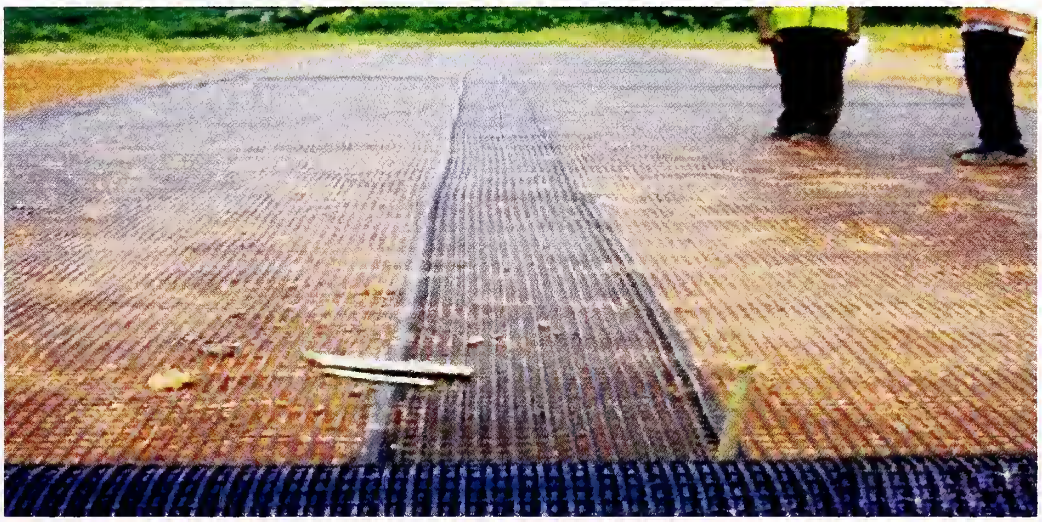


Fig. 17 : Installation of Woven Geogrids over Geotextile Separation Layer

b) Road Project: S.Marco-Argentano Road Italy

During the construction of the 3rd part of the National Highway No. 533, in junction with national motorway A3 at the location S. Marco Argentano in Italy, a marshy land consisting of soft clay was encountered near a bridge abutment. This zone had an approximate thickness of 2.50 m which extends for an area of 8,000 sqm. Anticipating the instability problems associated with the construction of the 7 m (maximum) high embankment, high strength uni-directional geogrid of high tenacity polyester having a tensile strength 400 kN/m^2 was used as a basal reinforcement in between the embankment fill and the soft soil strata as shown in **Fig. 18**. The geogrid **(Fig. 19)** used was having a Linear Low Density Polyethylene (LLDPE) coating to resist chemical attacks. A non-woven geotextile used as a separator between geogrid and the drainage blanket made of sand was also provided. Anchoring blocks were given at higher heights to reduce the base width required for stability. Complete structure is shown in **Fig. 20**.

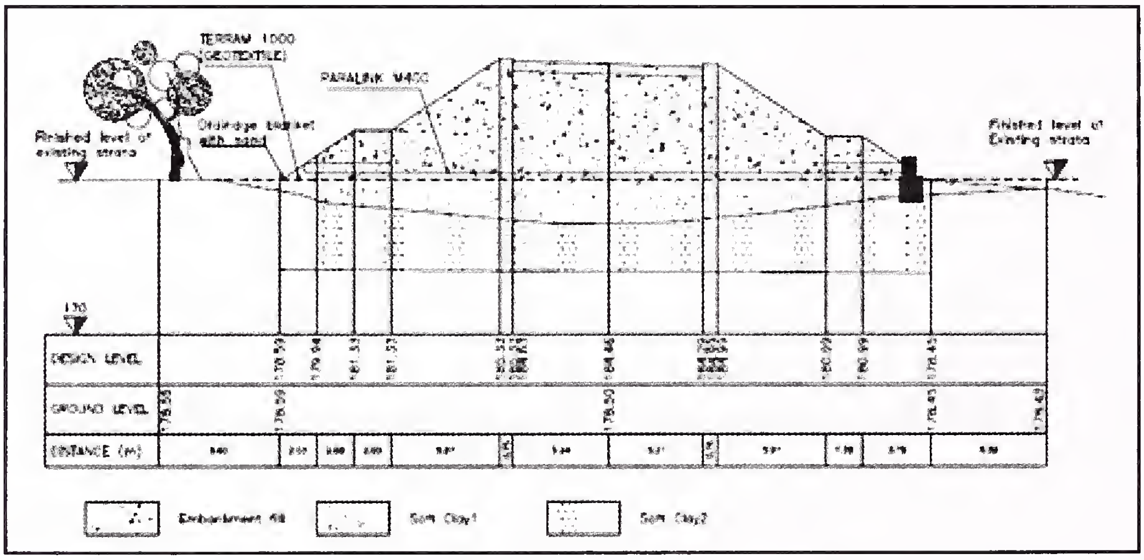


Fig. 18 : Typical Cross Section of S.Marco-Argentano Road Italy

Fig. 19 : Installation of Bonded Geogrids Over Geotextile Separation Layer

Fig. 20 : Completed Structure- S.Marco-Argentano Road Italy

c) Expressway: Bangkok, Thailand

The new Bangkok Expressway was crossing a marshy area with a very soft soil consisting of normal consolidated Bangkok clays, 20 m deep. The soft soil had a bulk density of 16 kN/m^3 and undrained shear strength of 15 kN/m^2. The site investigations showed that the bearing capacity of the foundation soil was too low to support a "traditional" highway embankment. In addition to the above considerations, high stiffness was required for the initial construction phase to allow the heavy plant to operate on site without sinking into the extremely soft soil that was often waterlogged. The design required four horizontal layers of biaxial geogrids for stabilizing this embankment. The geogrid used was of tensile strengths of 32 kN/m in the machine direction and 18 kN/m in the cross machine direction. The geogrids were installed at 300 mm vertical spacing. Tensile creep test results, adequate to determine the design tensile strength of the geogrids for a design of 1 year under constant load (1 year was the anticipated time for the consolidation of the clay soil under the embankment). In places, were water content is as high as 100%, prefabricated vertical drains were installed along with reinforcement.

d) Four Laning of NH-5 between Rajahmundry and Eluru, Andhra Pradesh

The four lane NH-5 project between Rajahmundry and Eluru in the Godavari delta of Andhra Pradesh included five reinforced soil retaining walls. The maximum height of reinforced soil wall was 15 m **(Fig. 21)**. This made them among the highest reinforced soil retaining walls built in India at the time when the project was taken up (in 2001). The reinforced soil walls were provided with high strength geogrid reinforcement. The subsoil formation in these stretches consisted soft deltaic clays of varying depth. To meet the bearing capacity and settlement problems arising from soft clay foundation layers a stepped wall configuration with wraparound geogrid reinforcement **(Fig. 22)** and basal mattress at base of the reinforced soil wall were adapted. The provided basal mattress was 1 m thick. The scheme contained a separation geotextile and three layers of geogrids. The lower layer of geogrid was having strength 200 kN/m and two layers of 40 kN/m geogrids were placed at 50 cm each. The space between the geogrid layers was filled with compacted stone metal. Basal reinforcement **(Fig. 23)** provided a stiff platform on the soft clay which ensured more uniform distribution of load on foundation. **Table 12** shows gradation of fill material used in Basal Mattress. The reinforcement absorbed the outward shear stresses near the edge and hence prevented them from being transferred to the soft soil. Hence, basal reinforcement enabled the soft soil to carry higher embankment loads. Constant monitoring of settlement was done by installing settlement gauges at intervals along the embankment length. The comparison of predicted and observed settlements was found to be satisfactory. View of the Finished Road (NH-5) Section between Rajahmundry and Eluru is shown in **Fig. 24**.

Table 11 : Details of Embankments

S. No	Stretch	Length (m)	Height of Embankment (m)	Depth of Soft Clay Layer
1)	R. Gowthami Eluru side Approach	396.0	15.0	6.00
2)	R. Vasishta Rajahmundry side Approach	338.0	15.0	6.00
3)	R. Vasishta Eluru side Approach	420.0	15.0	9.5
4)	Tanuku ROB Rajahmundry side Approach	443.0	9.0	4.5
5)	Tanuku ROB Eluru side Approach	352.0	9.0	5.5

Note : The National Highway Numbers have recently been changed by NHAI. The then prevailing No.NH-5 is now NH-16

Table 12 : Gradation of Fill Material Used in Basal Mattress

Sieve Size	% Passing
100	100
80	95-100
60	90-100
32	60-80
16	35-45
8	20-35
2	10-20

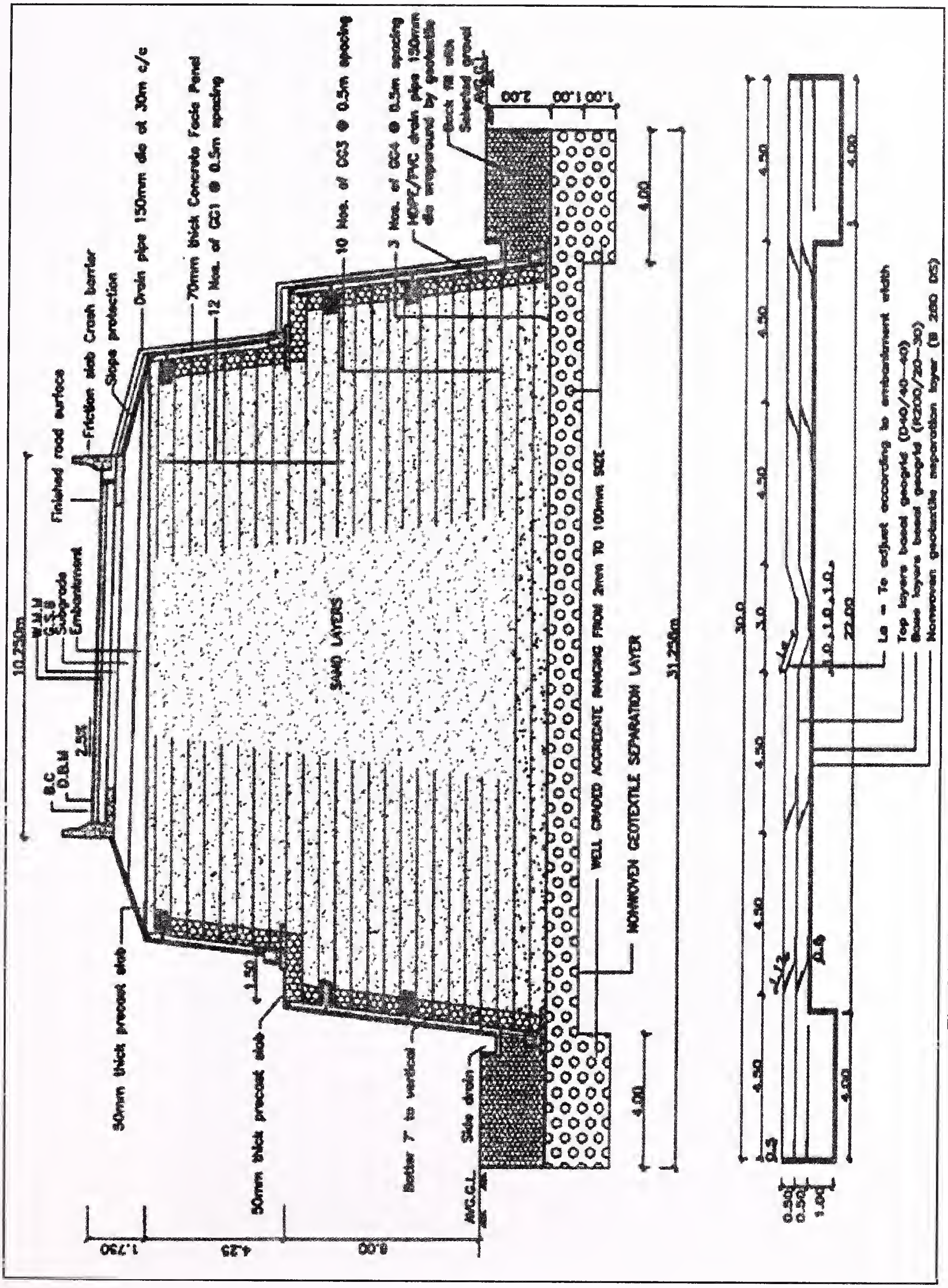


Fig. 21 : Typical Cross Section of Reinforced Soil Wall and Basal Mattress, NH-5

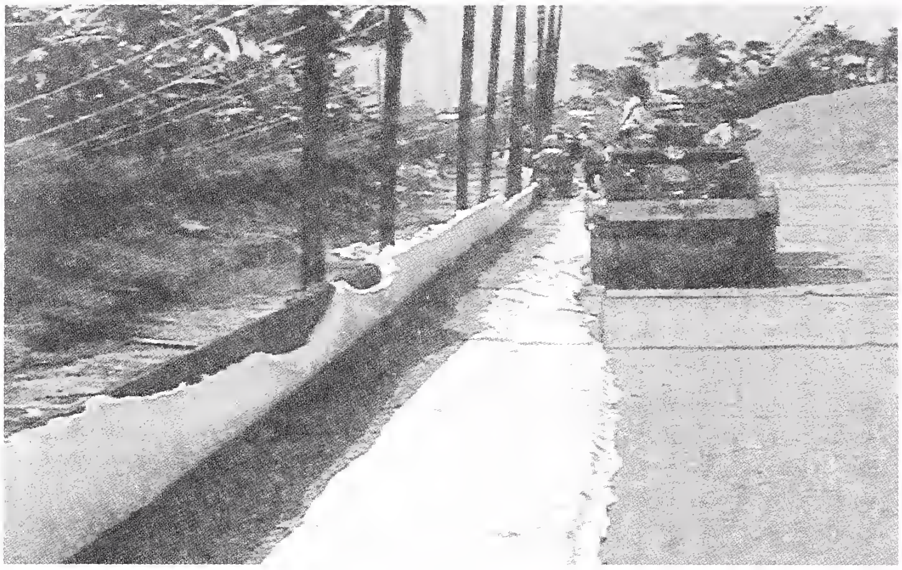


Fig. 22 : Finished Wraparound Layers

Fig. 23 : Construction of Basal Mattress at NH-5

Fig. 24 : View of the Finished Road (NH-5) Section Between Rajahmundry and Eluru

e) Road Project: Visakhapatnam Port Connectivity Road Project, Andhra Pradesh

To provide fast and easy access to Visakhapatnam port from NH–5, NHAI in association with Visakhapatnam Port Trust constructed a new port connectivity road. The length of this project road was about 12.3 km out of which 4.567 km length was provided with ground improvement using prefabricated vertical drains and high strength geotextile **(Fig. 25)** basal reinforcement layer. The thickness of soft marine clay was 10 to 18 m which had undrained shear strength of 5 to 8 kPa. Cc was varying from 0.8 – 1.2. As per IS classification system the soil was classified as CH type. The height of the embankment was 2.5 to 3.2 m.

The embankment construction comprised of laying working platform of 0.7 m thickness over original ground. Prefabricated Vertical Drains were installed at 1.15 m center-to-center in a triangular pattern after laying the initial embankment. Sand drainage layer of 0.6 m thickness was then laid over the initial embankment. High strength polymeric woven geotextile was then spread over the sand drainage layer. The design tensile strength of geotextile used was 230 kN/m. The geotextile was anchored at the ends by using sand filled bags. The embankment construction was then taken up in two stages – 1.75 m (first stage) and then 2.0 m (second stage). Waiting period for each stage was 175 days. A total quantity of about 124,000 sq.m of geotextile was used in this project as reinforcement layer. The project has been completed and opened to traffic in 2007.

Fig. 25 : Geotextile as Basal Reinforcement at Visakhapatnam

f) Road Over Bridge near PMC Building, Port Road, Mundra, Gujarat

Mundra Special Economic Zone (SEZ) is located in Kutch District; Gujarat. This is linked to the National Highway Network through an extension of NH 8A Ext. from Mundra-Anjar-Bhimasar. A railway crosses the port connectivity road (Mundra to NH 8A). ROB (8 lane) was proposed to cross the railway line. The approaches of ROB were proposed to be retained with a reinforced soil wall. (Granular fill soil --unit weight 20 kN/m^2, ϕ = 32°). Maximum height of reinforced soil wall was 9 m. The soil up to 3.0 m depth was clayey silt and followed by sand with silt up to 4.5 m depth. This is underlain by sandy silt with traces of clay till 9.0 m depth. Ground water table was at 1.5 m depth. High strength geogrids having mono-axial array of geosynthetic strips, which has a planar structure were used as basal reinforcement **(Fig. 26)** to improve the strength of the underlying soil with a drainage layer **(Fig. 27)** and geotextile in between.

The uni-directional ultimate strength of the mono-axial geogrid was 200 kN/m. Stone columns were also used to reduce the settlement of the approach road at higher heights. The high strength geogrids **(Fig. 28)** placed were effectively able to distribute the stress uniformly to the foundation soil, thereby decreasing the differential settlement. Maximum tensile load was calculated as the sum of the loads needed to transfer the vertical embankment loading on the stone columns and the load needed to resist lateral sliding. Since the load was transmitted to the stone columns, the settlement of the soil in between the columns was also reduced considerably. **Fig. 26** shows the typical cross section of the reinforced approach road.

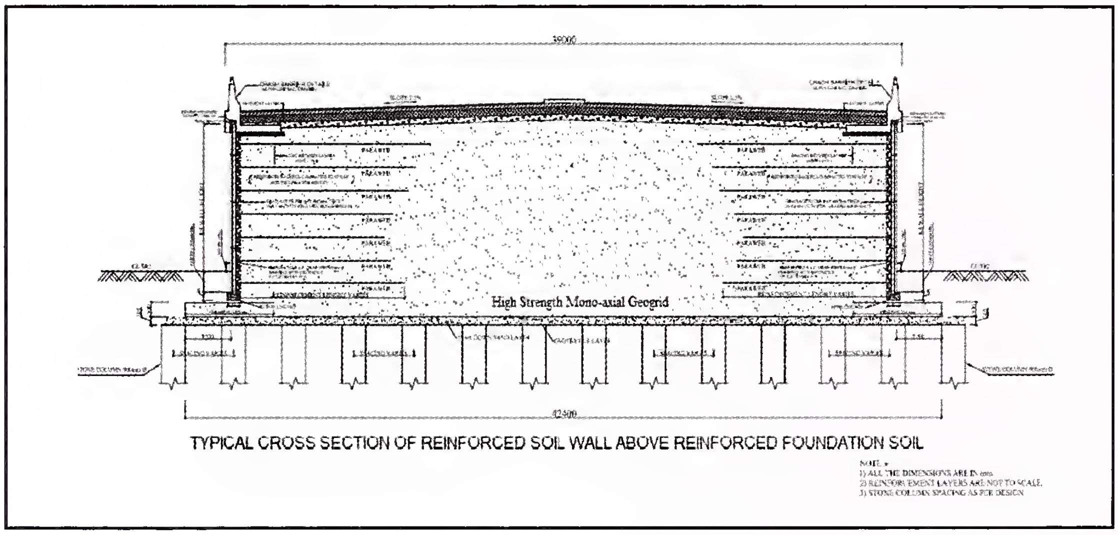


Fig. 26 : Cross Section of Basal Reinforcement with Stone Columns, Mundra, Gujarat

Fig. 27 : Installation of Drainage Layer, Mundra, Gujarat

Fig. 28 : Installation of Bonded Geogrid, Mundra, Gujarat

g) Four lane highway from Vallarpadam to NH-47

The 17 m highway provided four lane links for ICTT Vallarpadam to NH 47. Of this 17 km link nearly 10 km were in backwaters and the highway was built on reclaimed bed placed by hydraulic dredging. The embankment height was varying from 3.00 m to 7.20 m. Up to 3 m, the subsoil was very loose clayey silt sand followed by a very soft to soft highly compressible silty clay layer (Cu=12.5-14 kPa, Cc=1.2) of thickness 15-22 m. The layer is followed by medium dense to stiff silty sand. Due to low shear strength and high compressibility of the clayey sub soil the embankments were found to be unsafe in bearing capacity, rotational stability and could experience large settlements. The side slopes adopted for the embankments were 1V: 2H. Locally available moorum was used as embankment fill material (c=30 kPa and □=27°). Prefabricated vertical drains (PVDs were adopted as the ground improvement for stretch in the reclaimed area. The spacing of PVDs was kept at 1.150 m in a triangular pattern in all the locations. A layer of non-woven geotextile was laid above drainage granular blanket as a separator between embankment fill and drainage material. Three stage constructions for maximum height 7.20 m was designed for a cumulative waiting period of one year, so that the embankment construction was carried with minimum delay. Bearing capacity and stability analysis were conducted for each stage of loading. Based on the gain in shear strength after completion of waiting period of each stage, the height of next stage of fill placing was decided taking into account stability and bearing capacity aspects. In order to open one carriageway early stone columns were adopted for ground improvement at bridge approaches where the height of embankment was maximum. This enabled the full height to be built in a short time, avoiding the multiple stages if PVDs were adopted for ground improvement. Two lanes of highway were thereby opened early for traffic. To achieve the required bearing capacity of subsoil treated with stone columns, a waiting period of 30 days was recommended. At the 16 locations where embankment was built on 1.0 m diameter stone column the spacing of stone column was ranged from 1.65 m-1.75 m in triangular pattern in all the locations. Biaxial geogrids were used to transfer the stress uniformly to the stone columns and PVDs and thus reducing the differential settlement. Geogrids also helped in preventing the embankment from rotational failure as well. The condition of completed carriageway and traffic movement along the corridor is shown in **Fig. 29**.

Fig. 29 : Condition of Completed Carriageway

h) Road Over Bridge: Thane Bhiwandi Vadapa Road over South Kasheli Creek

For a major bridge across Thane Bhiwandi Vadapa Road, the solid approaches were required to be retained using reinforced soil walls. There was an embankment existing for many years. The road had to be widened to the increased width of the bridge. The subsurface comprised of top 4 to 6 m of very soft to soft dark grey clay. From 7.5 m to 10.0 m soil constituted silty clay. This layer was followed by medium dense dark grey medium sand. As the structures were near a creek ground water table was at top. Fill soil properties were considered as: Cohesion – 0 kN/m^2, Angle of friction - 32°, Unit weight of soil – 20 kN/m^3. The maximum height of the embankment was 9.6 m. In order to achieve the required global and bearing stability, basal reinforcement over piles **(Fig. 30)** was proposed for the new embankment. The piled embankment technique allows embankments to be constructed to the required heights

Fig. 30 : Typical Cross Section of Basal Reinforcement with Piles Used for Road Over Bridge, Thane.

without any restraint on construction rate with control on post construction settlements. Basal reinforcement was used to form a geosynthetic raft over piles and transfer the load to the piles, and thus enabling to maximize the economic benefits of the piles installed in soft foundations. The reinforcement also helped in counteracting the horizontal thrust of the embankment fill and the need for raking piles along the extremities of the foundation could be eliminated. In the direction along the length of the embankment the maximum tensile load should be the load needed to transfer the vertical embankment loading onto the pile caps. In the direction across the width of the embankment the maximum tensile load should be the sum of the load needed to transfer the vertical embankment loading onto the pile caps and the load needed to resist lateral sliding. Basal reinforcement proposed here was geogrid which has planar structure consisting of a monoaxial array of composite geosynthetics strips.

Each single longitudinal strip had a core of high tenacity polyester yarns tendons encased in a polyethylene sheath; the single strip was connected by cross laid polyethylene strip which gave a grid like shape to the composite. Two geogrids layers having uniaxial strength of 400 kN/m each, along and across the road were given. The design was carried out according to BS:8006 (1995). The design of piled embankments was not included in the scope of the present document.

i) Visvesvaraya Setu (Okhla Flyover) Project

Delhi PWD in association with CRRI constructed the reinforced fly ash approach embankment on one side of the slip roads adjoining NH-2 at Okhla in Delhi. During design stage, it was noticed that safe bearing capacity of subsoil was only 125 kN/m^2, while bearing pressure due to reinforced fly ash embankment wall was about 193 kN/m^2. Ground improvement was carried out by using two layers of bi-oriented geogrids at a depth of 0.45 m and 1.0 m placed at the bottom of reinforced fly ash embankment. Typical cross section of the embankment is shown in **Fig. 31**. Bottom ash, a waste material from thermal power plants was used as frictional fill in basal reinforcement portion. Bottom ash was filled in two layers up to a height of 0.5 m. Each layer was compacted to 95 percent of proctor density. Bi-orientated geogrid was spread **(Fig. 32)** over the compacted fill. 10 mm diameter rods were pegged down to ensure that geogrid stayed in its place. Compaction was carried out by 8 ton static roller followed by vibratory roller. The flyover was opened to traffic in Jan. 1996 and has been performing well.

j) Restoration of Wharf Road (NH-9) at Vijayawada by Geocell Basal Mattress & Reinforced Soil Wall

During September 1999, the Wharf Road in Vijayawada Municipal limits of NH-9 collapsed due to the failure of the retaining wall on the canal side **Fig. 33**. The restoration work of this road involved using stone filled basal mattress of geocells as foundation over which, geosynthetic reinforced soil wall in two tiers was built. The geotechnical investigations at site revealed that the soil profile is varying, but in general the subsoil in the top 2 m was clayey (CH or CI) and it was very soft. Below this the soil was generally CI and sometimes CH,

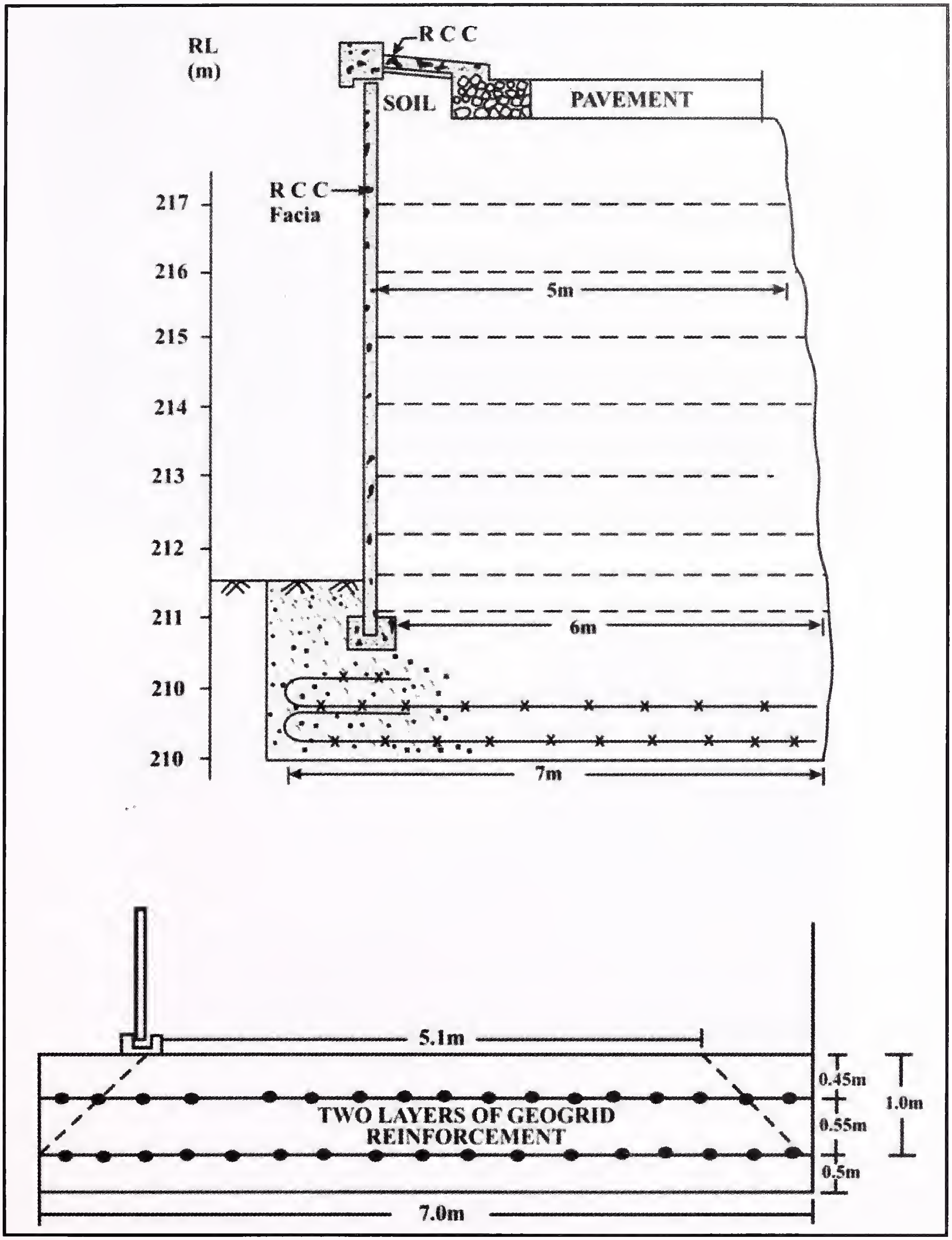


Fig. 31 : Typical Cross Section of the Basal Reinforcement Scheme Provided Under Reinforced Fly Ash Embankment at Okhla

Fig. 32 : Spreading Geogrid as Basal Reinforcement at Okhla

Fig. 33 : The Damaged Portion of Wharf Road Before Restoration

but in between there were few layers of sand (SM). With a wall height of 8.5 m above the canal bed level, and a 2 m thick soft soil near the canal bed, a gravity retaining wall was not feasible. Pile foundations needed to be at least 15 m to 20 m long and hence it would have been prohibitively expensive. As the road is located in a busy commercial area, there was little space available for conventional construction. Also major work of foundation as well as reinforced soil embankment for road restoration, were to be constructed during the 30-day closure of the canal. Keeping in view, soft nature of the soil, time constraints and inefficacy of the vertical drains in black soils, additional costs involved in case of piles, the restriction due to the land acquisition, etc., the best solution was thought to be a 'basal mattress reinforcement'. It has thus been designed to provide a foundation from stability view point only and not from the settlement point of view. Basal mattress used in this project, is a three-dimensional honeycombed structure (geocell), formed from a series of interlocking cells. These cells can be easily assembled at the site by using high strength geogrids, resting on the soft foundation soil and then filled up with granular material. A basal mattress thus formed provides a very stiff foundation platform designed to support the loads. The detailed design has been carried out using BS:8006 (1995) and AASHTO (1998) and the overall principles of geotechnical engineering. For the basal mattress 1.0 m high vertical geocell was formed using uniaxial geogrids laid on biaxial geogrid base. The geocell mattress extended 2.3 m beyond the facia. Provision of this mattress increased bearing capacity at the required level. **Fig. 34** shows sectional view of geosynthetics reinforced soil wall with stone filled geocell basal mattress. The construction of this 200 m stretch was successfully completed during the year 2002, despite difficulties experienced in making a deep cut next to the busy road, and the water level in the canal remaining at least 1m above the bed level. **Fig. 35** shows the road section after completion of restoration works.

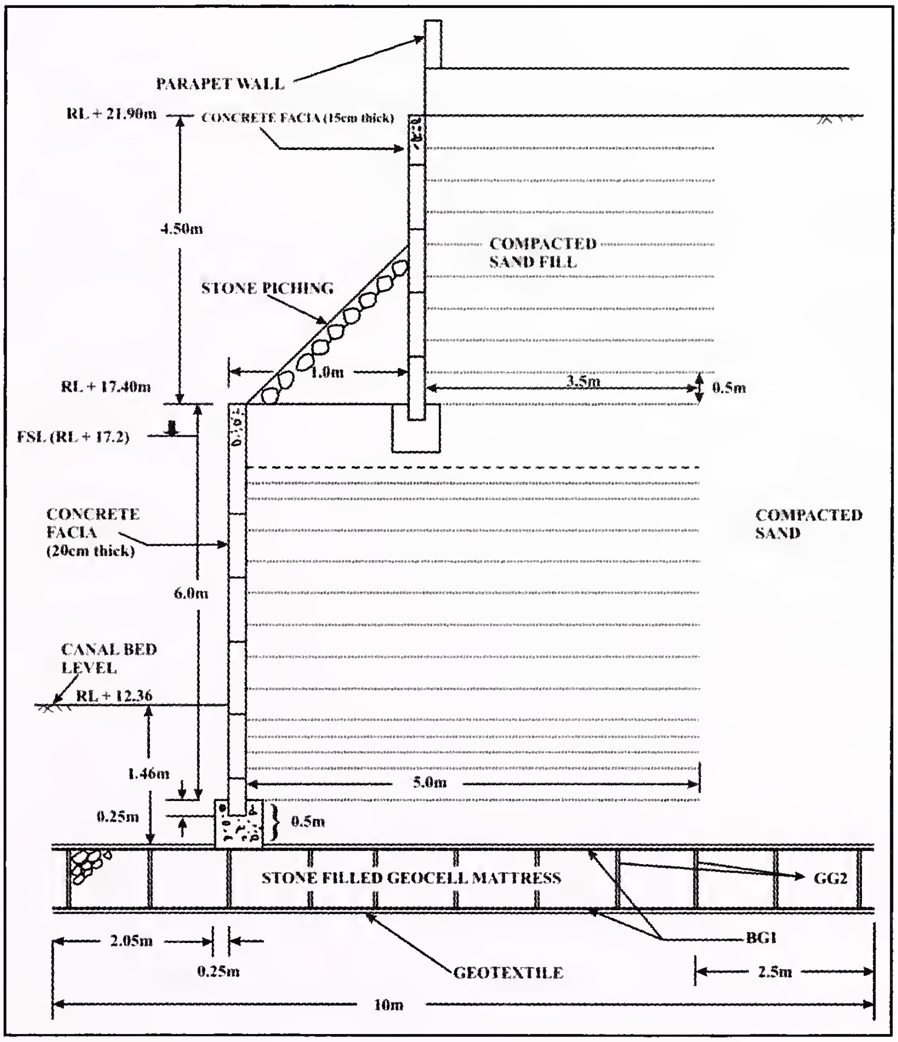


Fig. 34 : The Designed Section Showing Stone Filled Geocell Mattress

Fig. 35 : The Road Section After Completion of Restoration Works

k) Other Road Projects Where in Basal Reinforcement was Adopted

High strength polymeric woven geotextile (approximately 35,000 square meters) was used as basal reinforcement for road project at JNPT Port Connectivity Project of NHAI at Mumbai. High strength polymeric woven geotextile (approximately 14,000 square meters) was used as basal reinforcement for road embankment on soft soil at Amona – Khandola Bridge Project in Goa, for Goa PWD.

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Annexure 1

(Refer Section 10(d) 2)

BOQ ITEMS

Sl. No		Description	Unit of Measurement	Quantity	Rate
1)		**Reinforcement**			
	a)	**Bonded Geogrids**			
		Supply and laying of uniaxial geogrid, manufactured by bonding geosynthetic strips, having ultimate tensile strength of (as specified for the project) kN/m and tensile strengths of (as specified for the project) at 2% and 5% respectively provided in a roll of length (as specified for the project) metre and width (as specified for the project) metre.	Sq.M.		
	b)	**Extruded Geogrid**			
		Supply and laying of uniaxial geogrid, manufactured by extruding, having average tensile strength of (as specified for the project) kN/m in machine direction and tensile strengths of (as specified for the project) at 2% and 5% respectively provided in a roll of length (as specified for the project) metre and width (as specified for the project) metre.	Sq. M		
	c)	**Woven/Knitted Geogrids**			
		Supply and laying of Woven/Knitted geogrids made from high molecular weight, high tenacity multifilament yarns, having tensile strength of (as specified for the project) kN/m in machine direction and (as specified for the project) kN/m in cross machine direction and tensile strengths of (as specified for the project) at 2% and 5% respectively, provided in a roll of length (as specified for the project) metre and width (as specified for the project) metre.	Sq.M		
	d)	**Woven Geotextiles**			
		Supply and laying of woven geotextile made from multifilament yarn, having tensile strength of (as specified for the project) kN/m in machine direction and (as specified for the project) kN/m in cross machine direction with maximum apparent opening size of (as specified for the project) mm, provided in a roll of length (as specified for the project) metre and width (as specified for the project) metre.	Sq.M.		
	e)	**Geo composite**			
		Supply and laying of geocomposite made by bonding geogrid, having tensile strength of (as specified for the project) kN/m in machine direction and tensile strengths of (as specified for the project) at 2% and 5% respectively and non-woven geotextile with maximum apparent opening size of (as specified for the project) mm, permittivity (as specified for the project) sec^{-1} provided in a roll of length (as specified for the project) metre and width (as specified for the project) metre.	Sq.M.		

Sl. No		Description	Unit of Measurement	Quantity	Rate
	f)	**Geocell**			
		Supply and laying of geocell with (or without) perforations having a cell height of (as specified for the project) m, seam strength of (as specified for the project) N/mm, carbon black content of (as specified for the project) %, having expanded cell size (as specified for the project) cm^2.	Sq.M		
		Supply and placing of infill material as per design specification.	Cum		
2)	**Separation Layer**				
	Supply and laying of non-woven geotextile, having a grab tensile strength of (as specified for the project) kN/m, trapezoidal tear strength of (as specified for the project) kN/m, puncture strength of (as specified for the project) kN/m with maximum apparent opening size of (as specified for the project) mm, permittivity of (as specified for the project) sec^{-1} provided in a roll of length (as specified for the project) metre and width (as specified for the project) metre.		Sq.M		
3)	**Drainage Layer**				
	Supply, laying and compaction of drainage layer, (gravel or well graded sand) as per drainage specification		Cum		

Annexure 2
(Refer Clause 3.7)

CERTIFICATION FOR REDUCTION FACTORS OF GEOSYNTHETIC REINFORCING ELEMENTS

The following agencies provide certification for the use of geosynthetic material as reinforcing elements. These certifications are based on the results of the required tests carried out at accredited laboratories. Both the certifications are accepted in many countries of the world.

1) British Board of Agreement (BBA)
2) National Transport Product Evaluation Program (NTPEP)-AASTHO

Table -List of some Accredited Testing Laboratory for Geosynthetic Materials

S. No.	Name of Laboratory	Type	Remarks
1)	TRI/Environmental	Third party independent laboratory	
2)	SGI Testing Services, LLC	Third party independent laboratory	
3)	tBU	Institute	
4)	British Textile Technology Group (BTTG)	Third party independent laboratory	Tests on Durability are performed. Installation and creep tests are not done
5)	The Bombay Textile Research Association (BTRA)	Institute	Has testing facilities for some properties, but not for any of the reduction factors.

Table -List of Accreditation Institutes for Geosynthetic Testing Laboratories

Sr. No.	Name	Location	Email
1)	Geosynthetic Accreditation Institute (GAI) – Laboratory Accreditation Program (LAP)	United States of America (U.S.A.)	gkoerner@dca.net
2)	Deutsches Institut für Bautechnik (DIBt)	Germany	dibt@dibt.de
3)	United Kingdom Accreditation Service	United Kingdom	info@ukas.com

The certification of reduction values may also be based on test certificates issued by laboratories accredited by agencies listed in table above.

The list of some of the accredited laboratories is also available in http://www.geosynthetic-institute.org/gai/lab.htm

(The Official amendments to this document would be published by the IRC in its periodical, 'Indian Highways' which shall be considered as effective and as part of the code/guidelines/manual, etc. from the date specified therein)