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Research Article

The Significance of Geotextile in Unpaved Roads with Special Reference to Stress Analysis

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Abstract

The technique of ground improvement using geotextile is extensively used in the construction of unpaved roads, fabrication yards, parking spaces, etc. When the underlying soil is soft, having poor consistency and high compressibility, a geotextile layer can be placed over the subgrade followed by a compacted granular fill layer. Tingle and Jersey (2007) categorically pointed out the problems associated with maintaining low-volume unpaved roads with minimum funding and identified geotextile reinforcement as a possible means to deal with this condit ion. Use of geotextile as soil reinforcement has been reported to increase the overall stiffness and bearing capacity of the geotextile - soil composite. Geotextiles are also found helpful in reducing settlement and rutting depth. For a given design conditio n, these improvements lead to a reduced amount of aggregate material and time required for construction and extending of the service life.

***Keywords**: Geotextile, Stress ,Subgrade, Reinforcement, Pavement, Geosynthetics*

Introduction

Geotextile mainly provides separation between base course and subgrade. An analytical approach to the design of geotextile-reinforced unpaved roads was first introduced by Giroud and Noiray (1981). The bearing capacity of the soft subgrade is considered to increase from πc_u to $(\pi + 2)c_u$ with the inclusion of a geotextile; where c_u is the undrained shear strength of the cohesive subgrade. Additional improvement due to membrane action is considered to be a function of the geotextile tensile strength and allowable rut depth.

Reinforcement mechanisms

For roadway applications, geotextiles have been mostly used for separation, drainage, and filtration and woven geotextiles are sometimes used for reinforcement as a tensioned membrane. Lateral confinement, increased bearing capacity, and the tensioned membrane effect have been identified as the major geosynthetic reinforcement mechanisms (Giroud and Han, 2004). The stabilization of unpaved roads on soft ground with a geotextile is primarily attributed to the basic functions of separation of the base course layer from the subgrade soil, and a reinforcement of the composite system. Although field trafficking studies have consistently shown that the geotextile reduces rutting , there does not seem to be a consistent relationship between improved trafficability and tensile strength of the geotextile. The relationship appears to be good in some trials and poor in others. Analytical models have been proposed for the improved bearing capacity of a geotextile reinforced system that account for contributions from (1) a greater load distribution in the stabilized base course layer; (2) a larger bearing capacity factor due to confinement of the subgrade leading to a plastic, rather than elastic, yield; and (3) a tensioned-membrane effect in the deformed geotextile at large ruts. Although the basic functions of the geotextile are reasonably well understood, there are few data from field trials involving traffic loading that allow the relative improvement in performance of a road section with a geotextile to be quantified. This field test describes the performance of an unpaved- road trafficking trial at Vancouver, British Columbia (B.C.). The response to traffic loading of four test sections, each stabilized with a different geotextile, is compared with that of an unreinforced test section. Interpretation of the data addresses the development of ruts, subgrade deformations, strain in the geotextile, and the implications of the field observations for current design methods.

Functions of Geotextile in Unpaved Roads and Areas

Geotextile have been used for sub grade stabilization and base course reinforcement for construction of unpaved structures (roads and areas) since the 1970s. Placed

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between the subgrade and base course, or within the base course, the geotextile improves the performance of unpaved roads carrying channelized traffic and unpaved areas subjected to random traffic. Improved performance consists of increases to the volume of traffic that can be carried by a given thickness of base course, decreases to the base course thickness required to carry a given volume of traffic, or combinations of both increased traffic and thickness reduction. Use of lower quality base course material is another potential benefit provided by geotextile. Geotextile can provide separation between base and subgrade materials and reinforcement of the base course and subgrade. Separation prevents the mixing of subgrade soil and granular base materials and the resulting deterioration of the base course. Reinforcement increases the bearing capacity of the subgrade, stiffens the base layer thereby reducing normal stresses and changing the magnitude and orientation of shear stresses on the subgrade in the loaded area, restricts lateral movement of the base course material and the subgrade soil, and can provide tensioned membrane support where deep rutting occurs. Two types of geosynthetics are typically used in unpaved structures: geotextiles and geogrids. From the viewpoint of unpaved structure reinforcement, there is a significant difference between geogrids and geotextiles. Due to their large apertures, geogrids may interlock with base course aggregate if there is an appropriate relationship between geogrid aperture size and aggregate particle size. While the degree of interlocking depends on the relationship between geogrid aperture size and aggregate particle size, the effectiveness of interlocking depends on the in-plane stiffness of the geogrid and the stability of the geogrid ribs and junctions. As a result of interlocking, the mechanisms of unpaved structure reinforcement are different for geotextiles and geogrids.

Objective

The study presented in this paper is devoted to the use of geotextile in unpaved roads. Therefore geotextiles and unpaved roads (such as trafficked areas) will only be discussed in this papers. The design method developed in this paper can be used for unpaved roads reinforced with geotextiles by neglecting the effects of aggregate interlock and geosynthetic in-plane stiffness. The design method can also be used for unreinforced unpaved structures by neglecting the effect of reinforcement on subgrade bearing capacity. The use of the method for trafficked areas requires some judgment on the part of the design engineer because the number of vehicle passes is difficult to estimate when the traffic is not channelized.

Brief review

Geotextiles have been used for subgrade improvement and base reinforcement since 1970s. Geosynthetics research in unpaved roads has evolved over the last few decades. Early studies were concerned with the effect the geosynthetic inclusion would have on the bearing capacity of such roads; later, attention was focused on how geosynthetics might slow the development of ruts. Over the years, various design methods have been put forward, and to date, they deal typically with the relationship between rut depth and traffic, and the effect the inclusion of a geosynthetic has on rut development

Table 1 Literature review

Name of the Author	Area of Work	Year
Giroud and Noiray	Planar reinforcement, such as geogrid and geotextile, has been widely used in unpaved roads with established design methods. An analytical approach to the design of geotextile-reinforced unpaved roads was first introduced by them.	1981
M. R. Hausmann	Geotextiles for Unpaved Roads A Review of Design Procedures	1987
Robert A. Douglas	Anchorage and Modulus in Geotextile-Reinforced Unpaved Roads	1989
Robert A. Douglas & Arun J. Valsangkar	Unpaved Stiffness Geosynthetic-Built Resource Access Roads: Rather than Rut Depth as the Key Design Criterion	1990
N. Som and R.B. Sahu	Bearing capacity of a geotextile reinforced unpaved road as a function of deformation: a model study	1998
Marienfeld and Guram	There was evidence of possible benefits related to the use of geosynthetics in road construction.	1999
Beranek	There was evidence of possible benefits related to the use of geosynthetics in road construction.	2003
Rudolf Hufenus, Rudolf Rueegger et al	Full-scale field tests on geosynthetic reinforced unpaved roads on soft subgrade	2005
Tingle and Jersey	Categorically pointed out the problems associated with maintaining low-volume unpaved roads with minimal funding and identified geotextile reinforcement as a possible means to deal with this condition.	2007

G. Basu, A.N. Roy et al	Construction of unpaved rural road using jute–synthetic blended woven geotextile – A case study	2009
Ennio M. Palmeira, Luiz G.S. Antunes	Large scale tests on geosynthetic reinforced unpaved roads subjected to surface maintenance	2010
Palmeira	The presence of the reinforcement in unpaved roads can also markedly improve the performance these roads when built on weak subgrades	1981
Ramalho-Ortigao and Palmeira		1982
Love et al.		1987
Palmeira and Cunha		1993
Palmeira and Ferreira		1994
Fannin and Sigurdsson		1996
Palmeira		1998
Som and Sahu		1999
Hufenus et al.		2003
USACE		2003
Zhou and Wen		2008
Basu et al.		2009

Some others who have worked in this field are Milligan & Love, 1984; DeGaridel & Javor, 1986; DeGaridel & Morel, 1986; Resl & Werner, 1986; DeGaridel & Javor, 1986; Delmas *et al.*, 1986; Khay *et al.*, 1986; Holtz & Sivakugan, 1987; Gourc & Riondy, 1984;Delmas *etal.*, 1986; Yasuhara *etal.*, 1986; Holtz & Sivakugan, 1987; Jewell, 1990.

Theory

- The empirical design of geosynthetic-reinforced unpaved roads began with the incorporation of geotextiles at the base–subgrade interface for separation, filtration, and reinforcement. The first notable design procedure for geotextile-reinforced unpaved roads was proposed by Barenberg et al. on the basis of the limit equilibrium bearing capacity theory. The limit equilibrium bearing capacity theory is based on selecting an aggregate base thickness such that the vertical stress applied to the subgrade is below the theoretical limits for subgrade shear failure. This design procedure is based on the bearing capacity theory of a footing under static load, a granular fill, and a soft cohesive subgrade. An additional assumption is that the failure mode of the unreinforced system is characterized by local shear, while the failure mode of a geotextile-reinforced system is characterized by a general shear failure due to additional distribution of the load. Barenberg et al. proposed bearing capacity factors of 3.3 and 6.0 for unreinforced and reinforced systems, respectively. These factors were suggested for roads designed for very low traffic volumes and large deformations. The limit equilibrium bearing capacity theory was modified by Steward et al. by proposing lower bearing capacity factors to account for increased traffic requirements. Steward et al. suggested an unreinforced bearing capacity factor of 2.8 and a geotextile reinforced bearing capacity factor of 5.0 for unpaved roads designed for 1,000 equivalent single-axle loads (ESALs) and 2-in. of rutting. Steward et al. used a Boussinesq solution for calculating the vertical stress beneath a uniform circularly loaded area and the modified bearing capacity factors to construct design curves for single, dual, and dual tandem axle loadings.
- An alternative approach in the design of geosynthetic-reinforced unpaved roads was based on the widespread acceptance of the tensioned membrane effect as the primary reinforcement mechanism responsible for changing shear failure modes from localized shear for unreinforced systems to generalized shear for geotextile-reinforced systems. New design procedures were developed on the basis of the use of large-deformation membrane analysis equations. The most popular design procedure was produced by Giroud and Noiray and was also based on limit equilibrium bearing capacity theory with modifications to include benefits of the tensioned membrane effect.
- More recently Giroud and Han modified the Giroud and Noiray method to consider the stress distribution, base course strength properties, geosynthetic–base interlock, and geosynthetic in-plane stiffness. These additions are combined with previously considered factors: traffic volume, wheel load, tire pressure, subgrade strength, rut depth, and influence of the type of geosynthetic on the failure mode of the system.
- Giroud and Han's design method is based on determining the stresses at the base–subgrade interface and determining the rut depth as a function of those stresses and the subgrade bearing capacity. The influence of the number of vehicle passes and the properties of the geogrid are accounted for through modificationsof the stress distribution angle of the aggregate base.

Three critical assumptions regarding the subgrade bearing capacity factors are made by Giroud and Han . First, they select a bearing capacity factor of 3.14 for unreinforced unpaved roads, which is the elastic limit for a saturated undrained subgrade (zero shear strength,a conservative assumption). Second, a bearing capacity factor of 5.14 is selected for the case of a geotextile-reinforced unpaved road on the basis of the assumption that the geotextile provides a separation function resulting in a condition of zero shear strength at the base–subgrade interface. Finally, a bearing capacity factor of 5.71 (theoretical ultimate bearing capacity factor with maximum inward stress on the subgrade) is used for the geogrid-reinforced unpavedroads because of the expectation of maximum inward shear stress at the base–subgrade interface resulting from geogrid–aggregate interlock.

The restrained horizontal movement of the base material dueto the geogrid is expected to result in zero outward shear stress being applied to the subgrade surface. Interesting adaptations include the use of a mobilization coefficient to account for the fact that only a fraction of the maximum bearing capacity of the subgrade is mobilized during loading.

In summary, the common empirical design methods of reinforced unpaved roads are based on the limit equilibrium bearing capacitytheory. These design methods range from the original work of Barenberg et al. to the most recent adaptation by Giroud and Han .

Table 2: shows the critical assumptions for the three design:

Engineering Technical Letter 1110-1-189	Failure in subgrade Fine-grained subgrade soils with undrained loading conditions 2-in. rut failure criterion 1,000-pass failure criterion with linear extrapolations to higher traffic levels Geotextile primary function: separation rather than reinforcement Minimum aggregate thickness of 6 in. (0.15 m)
Giroud and Han (2004)	Uniform base course thickness Channelized traffic for nontraffic areas Minimum base course thickness of 4 in. (0.1 m) for constructability and anchorage purposes Fine-grained subgrade soils with undrained loading conditions Reinforcement allowing loads in the elastic zone while acting as though the subgrade is in the plastic zone Reorientation of shear stress at the subgrade interface Resilient moduli of base course and subgrade used Upper bound of base to subgrade modulus ratio: 5 Limited to less than 10,000 vehicle passes Minimum aggregate thickness of 4 in. (0.10 m)
Giroud and Noiray (1981)	Fine-grained subgrade soils with undrained loading conditions Limited to less than 10,000 vehicle passes Elliptical contact area from wheel replaced with rectangular area associated with dual tire Geotextile roughness preventing failure of the aggregate layer by sliding along the geotextile Pyramidal distribution of load in aggregate layer Assumed angle of load distribution pyramid Reinforcement allowing loads in the elastic zone while acting as though the subgrade is in the plastic zone Induced settlement under load assumed to be parabolic No minimum aggregate depth

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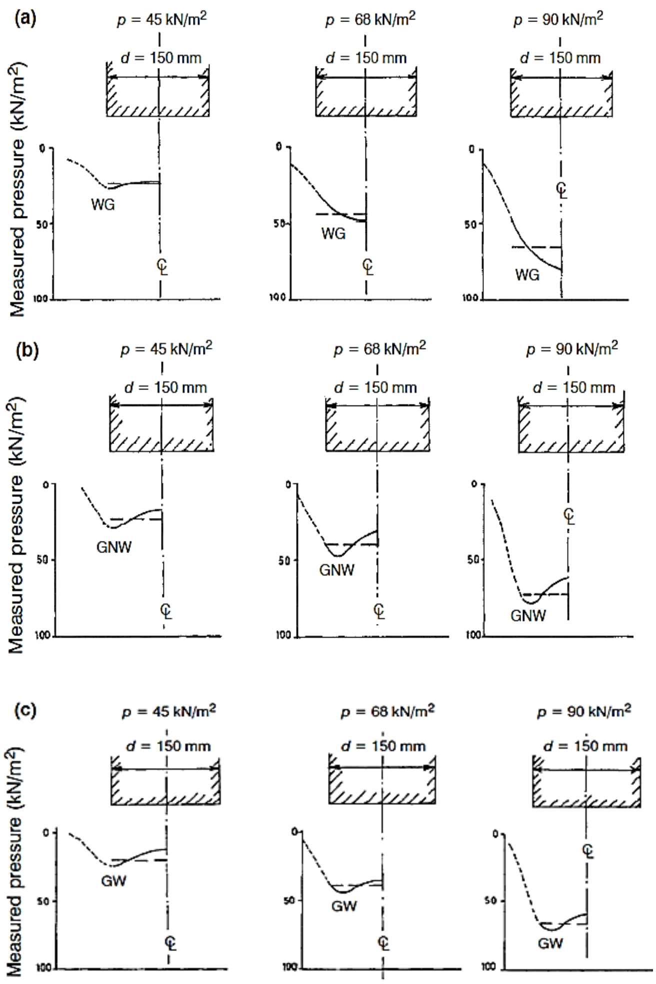
Stresses on the kaolinite layer were not recorded for footing pressures greater than 90 kN/m^2. At a footing pressure of 45 kN/m2, the observed pressure distributions indicate rigid footing behaviour (Das 1985) for both with and without a geotextile. At footing pressures of 68 and 90 kN/m2, the reinforced soil layer acts as a rigid footing, while the unreinforced layer pressure distribution is similar to a flexible footing. The average pressures on the kaolinite layer, obtained from the measured interface pressure distributions, for different fill thickness values, are presented in Table.

Load Dispersion

From the average pressure on the kaolinite layer for different fill thicknesses, the load dispersion values have been assessed and are presented in Table8. From Table, it is seen that the load dispersion is on the order of 4:1 for fill thicknesses of 40 and 75mm at an average footing pressure of 45 kN/m2.Beyond this footing pressure, it is found that the load dispersion varies between 7:1 to 12:1 (vertical:horizontal). Similarly, for a fill thickness of 110 mm, the load dispersion ranges from 3:1 to 5:1, up to an average footing pressure of 68 kN/m^2. For a fill thickness of 150 mm, the maximum load dispersion within the range of measured pressures is found to be 3:1.



Pressure distribution on the kaolinite layer for a fill thickness of 110 mm: (a) without a geotextile, WG; (b) with a nonwoven geotextile, GNW; (c) with a wov geotextile, GW.

Figure-3

Comparing these observations with the load-carrying capacity of the soil layer for different fill thicknesses at different footing settlements, it can be concluded that the load dispersion angle for the present investigation is on the order of 4:1 up to a maximum settlement of 10 mm. Beyond 10 mm of settlement, it appears to lie in the range of 7:1 to 12:1 (with an average of 10:1).

Bearing Capacity Factor, po / cu

Bearing capacity factors are available in the literature for estimation of the load-carrying capacity of unreinforced and reinforced unpaved roads, i.e. for soil layers with a granular fill overlying soft soil. These bearing capacity factors multiplied by the cohesion of the soft subgrade, along with the dispersion effect, give

Average pressure on the kaolinite layer from the measured interface pressure distribution values.

Fill thickness (mm)	Average measured pressure on kaolinite layer (kN/m^2)								
	p = 45 kN/m^2			p = 68 kN/m^2			p = 90 kN/m^2		
	WG	GNW	GW	WG	GNW	GW	WG	GNW	GW
40	38	35	34	62	64	63	--	81	81
75	30	29	23	52	54	52	--	75	74
110	24	22	20	42	40	40	66	72	66
150	11	11	13	18	18	18	30	30	32

Note: p = average footing pressure; WG = without geotextile; GNW = with nonwoven geotextile; GW = with woven geotextile.

Predicted load dispersion values for the kaolinite layer.

Fill thickness (mm)	Predicted load dispersion								
	p = 45 kN/m^2			p = 68 kN/m^2			p = 90 kN/m^2		
	WG	GNW	GW	WG	GNW	GW	WG	GNW	GW
40	5:1	4:1	4:1	12:1	12:1	12:1	--	10:1	10:1
75	4:1	4:1	3:1	7:1	8:1	7:1	--	10:1	10:1
110	4:1	3.5:1	3:1	5:1	5:1	5:1	9:1	12:1	9:1
150	2:1	2:1	2:1	2:1	2:1	2:1	3:1	3:1	3:1

Note: p = average footing pressure; WG = without geotextile; GNW = with nonwoven geotextile; GW = with woven geotextile.

the maximum load on the fill layer. No recommendation has been given for the behaviour of a soft soil layer at different levels of deformation. An attempt has, therefore, been made to estimate the soil layer bearing capacity factors, i.e. *po* / *cu* , where *po* is the pressure on the kaolinite layer after dispersion and *cu* is the cohesion of the kaolinite layer, for different footing settlements. The load dispersion angle, was taken as 4:1 for settlements of 5 and 10 mm, and 10:1 for settlements of 15 mm and greater. The bearing capacity factor values, *po* / *cu* , calculated for different fill thicknesses and footing settlements are given in Table 7. The values presented for 5 and 10mmof settlement are the average values for the unreinforced and reinforced soil layers. For a settlement equal to or greater than 15 mm, the values for reinforced soil layers are given. Comparing the values presented in Table 9with the load-carrying capacity proposed by Giroud and Noiray (1981) andMilligan et al. (1989), it is seen that the average value of *po* / *cu* for a settlement of 10 mmis comparable with the bearing capacity factor π and (π/2 + 1) for unreinforced soil layers, while the same bearing capacity factor value for 15 mm of settlement is close to (π + 2) for reinforced layers. Higher values of *po* / *cu* , observed at greater amounts of settlement, are probably due to the lateral restraint mobilized through interface friction at the geotextile-soft soil (i.e. kaolinite) interface and tension induced in the geotextile.

Experimental study

Laboratory model footing tests were performed to study the improvement in behaviour of a geotextile-reinforced soil layer as a function of footing settlement. Tests were carried out in a steel tank having a diameter of 700 mm, a height of 700 mm, and a 150 mm diameter rigid circular footing (Figure 1). The model subgrade was prepared by placing commercial-grade kaolinite, from a slurry, in the test tank and artificially consolidating the kaolinite. The liquid limit and plastic limit of the kaolinite were 45 and 25%, respectively, having silt and clay fractions of 71 and 29%, respectively. The final thickness of the subgrade after consolidation was maintained at 450 mm. The water content of the consolidated kaolinite layer was measured as 32 to 33%.

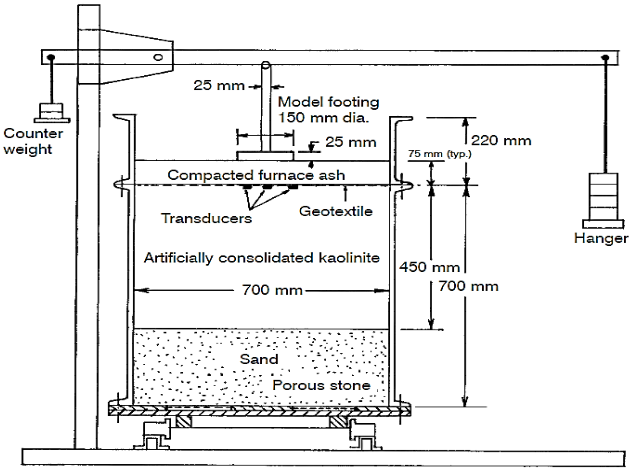


Figure 4: Test set-up for the laboratory model footing test.

The engineering properties of the material are given in Table 3. Furnace ash layer thicknesses of 40, 75, 110, and 150 mm were used. The furnace ash was compacted at optimum water content in layers of 35 to 40 mm using a 20 kN rigid circular plate. The unit weight of the compacted furnace ash was 11.6 kN/m3, corresponding to 80% of the maximum dry density. The 1st tests consisted of model footings placed on the compacted furnace ash overlying the consolidated kaolinite layer without a geotextile at the interface. In the 2nd tests, a 700mm diameter geotextile specimen was placed at the interface of the kaolinite and furnace ash layers. Before laying the geotextile, pressure transducers were placed at three positions (centre, edge, and 50 mm from the centre of the footing) on the kaolinite layer in order to measure the kaolinite-geotextile interface stresses with increases in footing pressure. Polypropylene, needle-punched nonwoven and multifilament woven geotextiles were used. After preparing the soil layer, the footing was placed on top and in the centre. A load was applied to the footing in increments using a lever system (Figure 1). For each load increment the settlement was recorded with time. The next load increment was applied when the settlement stabilised. The process was continued until a large soil layer deformation was measured.

Figure 5: Dr. Arvind Dewangan is taking result from an experiment in HIET Kaithal at HCTM Technical Campus Kaithal.

Table-3

Engineering properties of the furnace bottom ash.

Physical property	Value
Specific gravity	2.10
Particle size distribution	
Gravel	2%
Coarse sand	7%
Medium sand	55%
Fine sand	33%
Silt	3%
Uniformity coefficient	2.10
Optimum moisture content	29%
Maximum dry unit weight	11.6 kN/m^3
Internal friction angle	45°

Table-4

Engineering properties of the nonwoven and woven geotextiles.

Physical property		Nonwoven geotextile	Woven geotextile
Mass per unit area (g/m^2)		204	204
Thickness under a 2 kN/m^2 load (mm)		3.40	0.65
Peak wide-width tensile load (kN/m)	MD	31.0	40.8
	CD	13.4	22.3
Elongation at maximum load (%)	MD	60.6	17.6
	CD	75.0	17.8
Secant modulus at 10% elongation (kN/m)	MD	39.2	314.9
	CD	39.2	157.5
Elongation at 50% peak strength (%)	MD	29.0	5.6
	CD	41.0	6.4
Geotextile-kaolinite interface friction angle (°)		22	25

Note: MD = machine direction; CD = cross-machine direction.

Discussion of test result

Load-Settlement Performance

Average footing pressure versus settlement curves for fill thicknesses of 75 and 110 mm are presented. It is seen that for footing deformations less than 10 mm there is practically no change in the load-settlement behaviour with the inclusion of a geotextile at fill-subgrade interface.But,for unreinforced layer it is much higher in comparison with the reinforced layer beyond 10 mm of settlement. Further, the load-settlement behaviour is found to be similar for both woven and nonwoven geotextile.

Load capacity ratio(lcr)

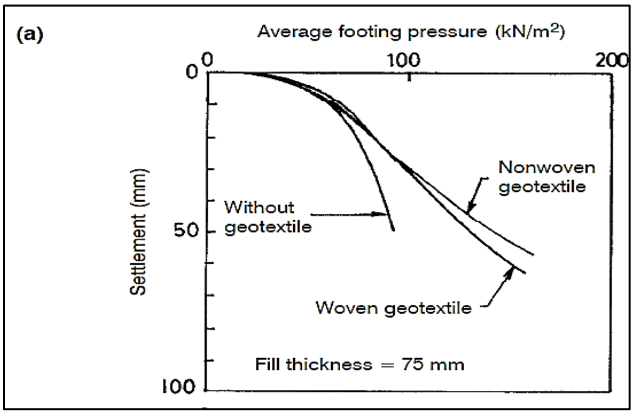
Improvement in the load-carrying capacity of a reinforced soil layer with the inclusion of a geotextile is typically expressed as the ratio of the ultimate load on the reinforced soil to that of the unreinforced soil. The improvement parameter is denoted by the load capacity ratio, *LCR*, and is defined as:

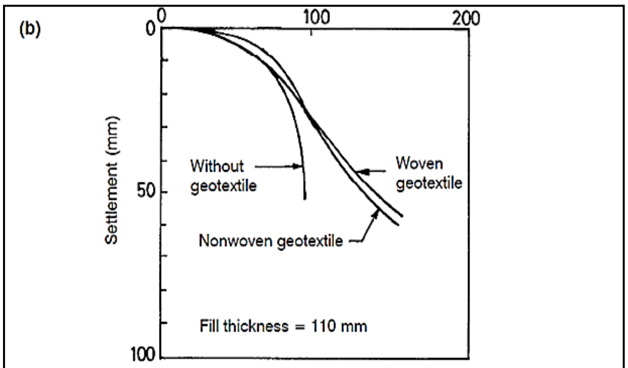
$$LCR = \frac{\text{Footing pressure for reinforced soil bed at a specified settlement}}{\text{Footing pressure for unreinforced soil bed at the same settlement}}$$

Table-5

Bearing capacity factors and improvement ratio.

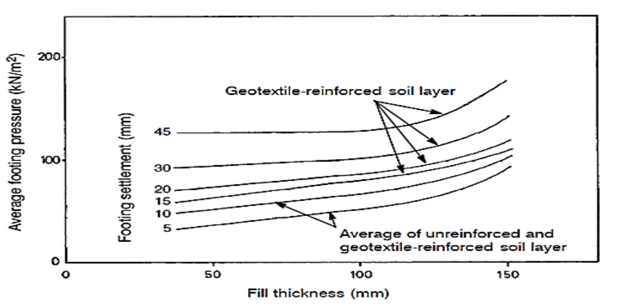
Reference	Unreinforced	Reinforced	Improvement ratio
Giroud and Noiray (1981)	3.14	5.14	1.64
Milligan et al. (1989)	2.57	5.14	2.00
Houlsby and Jewell (1990)	3.07	5.69	1.85





Settlement versus pressure relationships for different furnace ash layer thicknesses: (a) 75 mm; (b) 110 mm.

Figure-2



Variation of the average footing pressure values for different fill thicknesses.

Figure-3

Table-6

Load capacity ratio, *LCR*, values at various settlement values and different furnace ash layer thicknesses.

Furnace ash layer thickness (mm)	Load capacity ratio, *LCR*			
	s = 15 mm (*d*/10)	*s* = 20 mm (2*d*/15)	*s* = 30 mm (*d*/5)	*s* = 45 mm (3*d*/10)
40 = (0.27*d*)	1.09 (1.21)	1.17 (1.46)	1.27 (1.94)	1.41 (2.65)
75 = (0.50*d*)	1.08 (1.22)	1.11 (1.38)	1.21 (1.69)	1.41 (2.21)
110 = (0.73*d*)	1.11 (1.21)	1.09 (1.27)	1.14 (1.49)	1.32 (1.86)
150 = (1.0*d*)	1.02 (1.08)	1.07 (1.18)	1.22 (1.40)	1.44 (1.73)

Notes: *d* = diameter of footing; *s* = settlement. Values in parentheses give the *LCR* values in terms of the bearing capacity of the unreinforced soil layer at a settlement of 10 mm.

The *LCR* values calculated at various settlement and different furnace ash layer thicknesses are presented in Table 6. The values in parentheses are the ratios of the load carrying capacity of the reinforced soil layer at a specified settlement to the unreinforced layer at a settlement of 10 mm.

Stress Distribution

The stress distribution on the kaolinite layer at the kaolinite-geotextile or kaolinitefurnace ash interface was measured with increases in footing pressure in order to assess the load dispersion angle over the soil layer. The predicted load dispersion angle is then used to estimate the bearing capacity factors of the soil layer with increases in footing deformation. Typical vertical stress distributions measured below the interface (on the top surface of the kaolin layer) for a fill thickness of 110 mm and different footing pressures are presented in Figure. Stresses on the kaolinite layer were not recorded for footing pressures greater than 90 kN/m^2. At a footing pressure of 45 kN/m2, the observed pressure distributions indicate rigid footing behaviour (Das 1985) for both with and without a geotextile. At footing pressures of 68 and 90 kN/m2, the reinforced soil layer acts as a rigid footing, while the unreinforced layer pressure distribution is similar to a flexible footing. The average pressures on the kaolinite layer, obtained from the measured interface pressure distributions, for different fill thickness values, are presented in Table.

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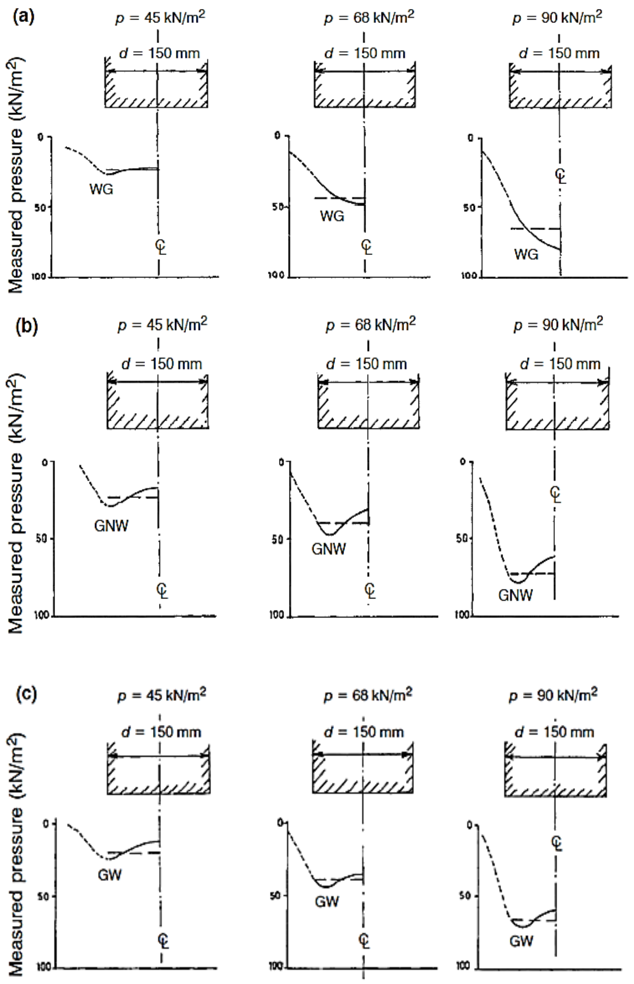

Pressure distribution on the kaolinite layer for a fill thickness of 110 mm: (a) without a geotextile, WG; (b) with a nonwoven geotextile, GNW; (c) with a woven geotextile, GW.

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Bearing Capacity Factor, po / cu

Bearing capacity factors are available in the literature for estimation of the load-carrying capacity of unreinforced and reinforced unpaved roads, i.e. for soil layers with a granular fill overlying soft soil. These bearing capacity factors multiplied by the cohesion of the soft subgrade

Table-7&8

Average pressure on the kaolinite layer from the measured interface pressure distribution values.

Fill thickness (mm)	Average measured pressure on kaolinite layer (kN/m^2)								
	$p = 45$ kN/m^2			$p = 68$ kN/m^2			$p = 90$ kN/m^2		
	WG	GNW	GW	WG	GNW	GW	WG	GNW	GW
40	38	35	34	62	64	63	--	81	81
75	30	29	23	52	54	52	--	75	74
110	24	22	20	42	40	40	66	72	66
150	11	11	13	18	18	18	30	30	32

Note: p = average footing pressure; WG = without geotextile; GNW = with nonwoven geotextile; GW = with woven geotextile.

Predicted load dispersion values for the kaolinite layer.

Fill thickness (mm)	Predicted load dispersion								
	$p = 45$ kN/m^2			$p = 68$ kN/m^2			$p = 90$ kN/m^2		
	WG	GNW	GW	WG	GNW	GW	WG	GNW	GW
40	5:1	4:1	4:1	12:1	12:1	12:1	--	10:1	10:1
75	4:1	4:1	3:1	7:1	8:1	7:1	--	10:1	10:1
110	4:1	3.5:1	3:1	5:1	5:1	5:1	9:1	12:1	9:1
150	2:1	2:1	2:1	2:1	2:1	2:1	3:1	3:1	3:1

Note: p = average footing pressure; WG = without geotextile; GNW = with nonwoven geotextile; GW = with woven geotextile.

along with the dispersion effect, give the maximum load on the fill layer. No recommendation has been given for the behaviour of a soft soil layer at different levels of deformation. An attempt has, therefore, been made to estimate the soil layer bearing capacity factors, i.e. *po* / *cu* , where *po* is the pressure on the kaolinite layer after dispersion and *cu* is the cohesion of the kaolinite layer, for different footing settlements. The load dispersion angle, was taken as 4:1 for settlements of 5 and 10 mm, and 10:1 for settlements of 15 mm and greater. The bearing capacity factor values, *po* / *cu* , calculated for different fill thicknesses and footing settlements are given in Table 7. The values presented for 5 and 10mmof settlement are the average values for the unreinforced and reinforced soil layers. For a settlement equal to or greater than 15 mm, the values for reinforced soil layers are given. Comparing the values presented in Table 9with the load-carrying capacity proposed by Giroud and Noiray (1981) andMilligan et al. (1989), it is seen that the average value of *po* / *cu* for a settlement of 10 mmis comparable with the bearing capacity factor π and $(\pi/2 + 1)$ for unreinforced soil layers, while the same bearing capacity factor value for 15 mm of settlement is close to (π + 2) for reinforced layers. Higher values of po / cu , observed at greater amounts of settlement, are probably due to the lateral restraint mobilized through interface friction at the geotextile-soft soil (i.e. kaolinite) interface and tension induced in the geotextile.

Load Transfer Mechanism

Giroud and Noiray (1981) considered that the load-carrying capacity of a subgrade with the inclusion of a geotextile would improve the bearing capacity of the soft subgrade from πcu to $(\pi + 2)cu$. Additional improvement would be due to the tension induced in the geotextile. Milligan et al. assumed load transfer mechanism for a geotextile-reinforced unpaved road with a geotextile at the fill-subgrade interface that the shear stress developed at the base of the fill would be taken up by the geotextile and only pure vertical stresses would be transferred to the soft subgrade. This consideration is based on the assumption that deformation of the footing is small and the type of geotextile to be used would be sufficiently stiff. Further, the load dispersion angle in both the cases was assumed to be 25 to 45^0 (2:1 to 1:1).

Table-9

Bearing capacity factor, p_o / c_u for the kaolinite layer.

Fill thickness (mm)	Bearing capacity factor, p_o/c_u					
	$s = 5$ (d/30)	$s = 10$ (d/15)	$s = 15$ (d/10)	$s = 20$ (2d/15)	$s = 30$ (d/5)	$s = 45$ (3d/10)
40 = (0.27d)	2.14	3.11	4.35	5.26	6.99	9.54
75 = (0.50d)	2.35	3.09	4.89	5.51	6.75	8.82
110 = (0.73d)	2.45	3.12	5.32	5.64	6.59	8.24
150 = (1.0d)	3.33	3.70	6.25	6.82	8.10	10.0
Average	2.57	3.26	5.20	5.81	7.11	9.15

Note: d = diameter of footing; s = settlement.

From the current study, it is seen that the geotextile-reinforced soil layer has a greater load-carrying capacity when footing deformations are greater than 15mm (0.10d). The behaviour of the low initial modulus nonwoven geotextile and the high initial modulus woven geotextile is found to be similar for the experimental set-up used for the current investigation and is in agreement with the work reported by Burd and Brocklehurst (1990). Furthermore, from the model test data, it was found that the ultimate load-carrying capacity of the soft soil layer was mobilized at a settlement of 0.10 times the diameter of the footing. At higher loads, the deformation of the reinforced soil layer was less than the unreinforced soil layer. This improvement in the reinforced soil layer was caused by lateral restraint of the geotextile, which is the result of geotextile-soft soil interface shear. Interface friction mobilized at greater settlements causes increases in the horizontal stresses leading to an increase in vertical stresses on the soil layer. These horizontal stresses induce tension in the geotextile to maintain equilibrium. If the undrained shear strength remained the same, an increase in the horizontal stress, σh , would cause an increase in the vertical stress, σv. Vertical stresses mobilized on the surface of the kaolinite layer may, therefore, be considered as the sum of the increased horizontal stresses and twice the cohesion of the soil layer.

Table-10

Mobilized vertical stress, σ_{vm} , for different footing settlements.

Fill thickness (mm)	Settlement, s (mm)	Vertical stress, σ_v (after dispersion) (kN/m^2)	Mobilized vertical stress, σ_{vm} (kN/m^2)	
			NWG	WG
40	20	63	74	78
	30	84	92	97
	45	114	116	123
75	20	66	76	81
	30	81	88	94
	45	106	109	115
110	20	68	78	83
	30	79	88	93
	45	99	104	110
150*	20	66	76	81
	30	79	88	93
	45	97	102	108

Notes: *Load dispersion = 6:1; NWG = nonwoven geotextile; WG = woven geotextile.

From Table 10, it is seen that the mobilized vertical stress values, σvm , for different fill thicknesses are within the range of the pressure applied to the kaolinite layer. The calculated σvm values are similar to the values of σv after dispersion for a settlement of 45 mm for both the nonwoven and woven geotextile. Overestimation of σvm at 20 and 30mmof settlement may be due to the peak interface friction not being fully mobilized. The σvm values are higher for the woven geotextile in comparison with the nonwoven geotextile. This is clear fromFigure 6 where σv values are plotted in terms of po / cu (after dispersion) versus mobilised po / cu . However, considering the range of variation of the predicted σvm or po/cu values, it may be said that the bearing capacity factors given in Table 7 are reasonable to estimate the load-carrying capacity of geotextile-reinforced soil at different footing settlements.

Analysis

Efficiency Calculation

The efficiency of the geosynthetic as a reinforcement in a road can be quantified by the Traffic Benefit Ratio, defined as:

$$\textbf{TBR} = \frac{N_r}{N_u}$$

where TBR is the traffic benefit ratio, Nr is the number of load cycles on the reinforced road for a given rut depth and Nu in the number of load cycles on the unreinforced road for the same rut depth. Koerner (1994) reports values of TBR varying between 2 and 16,depending on the soil and geosynthetic characteristics.

Comparison

Cost of Non-woven and Woven Fabrics as Reinforcement

Properties		Non-wovens		Wovens	
		Imported BIDIM U 34	Indigenous Dinesh GPB 127	Imported LOTRAK 16/15	Indigenous Garware
Ultimate breaking strength	kN/m	13·7	13·7	15·7	15·7
Design strength	kN/m	3·4	3·4	3·9	3·9
Weight of fabric	g/m^2	270	295	120	150
Cost of fabric	US$/m^2	0·77[a]	3·8	0·54[b]	2·3
Weight of fabric per m^3 of soil to induce bearing capacity of 39·5 kN/m^2	kg/m^3	3·4	3·4	1·20	1·20
Cost of reinforcement	US$/m^3	9·6	44·3	5·4	18·5

[a,b]Based on 1984 and 1985 prices, respectively.

Borrowed from:

Geosynthetic Characteristics

Geosynthetic	Mass/unit area (g/m^2)	Tensile modulus[a] (kN/m)	Tensile strength[a] (kN/m)	Elongation at failure[a] (%)
Nonwoven geotextile (test results: ASTM D4595)	150·0	22·0/21·2	4·86/5·26	163/147
Geogrid (manufacturer's literature — Tensar, 1987)	400·0	NA	15·0/15·0	NA

[a]Machine/cross-machine direction.

Borrowed from:

Benefits of Geotextile Reinforcement

The following benefits have been identified as a result of laying a geotextile as a reinforcing layer between the fill and the subsoil.

- Economic advantages of geotextile reinforcement primarily in the possibility to reduce the thickness of the fill layer or to limit the amount of subgrade to be removed.
- This saves on the use of granular materials and the amount of unsuitable material for removal and deposition elsewhere,which has both economical and ecological aspects,and may be useful for construction tracks laid without asphalt surfaces.
- Reduction in the rut formation as a function of the trafficking, increasing the serviceable life of the road.

Advantages of Geotextile Reinforced Unpaved Roads

- Besides reinforcing the system, depending on the geotextile characteristics, separation between a high quality fill material and a poor foundation soil can avoid or minimise the impregnation of the voids of the former by particles of the latter, increasing the life time of the road. Under large strain conditions the membrane effect provides additional benefits for road reinforcement.
- Geosynthetic reinforced unpaved roads are also easier and quicker solutions compared to traditional alternatives, such as the use of greater fill heights or the substitution of the poor foundation soil by a more competent one, which are solutions detrimental to the environment.

Conclusions

- The presence of the reinforcement layer reduced significantly the vertical stress increments transferred to the subgrade and vertical strains in the subgrade.
- Current geosynthetic design methods consist of empirical relationships based on limit equilibrium bearing capacity theory with adaptations to include theoretically rigorous parameters linked to failure mechanisms identified in historical experiments.
- All three design methods show significant benefits in terms of reduced aggregate thickness for geotextile- and geogrid-reinforced unpaved roads.
- Inclusion of a geotextile layer at the fill-soft subgrade interface improves the load-carrying capacity of the soil layer at a greater footing settlement

Future work

- Current design procedures for geotextiles are narrowly based on strength and deformation characteristics of fabrics and subgrade. The theoretical considerations of the effects of fabrics on the bearing capacity--such as control of the subgrade failure mode, enhanced ability of the aggregate to distribute surface loads, and membrane support--are relatively crude. It would really be desirable to obtain more field performance data before accepting these concepts as good approximations of the complex structural interaction of soil and fabrics.
- In addition to proving structural adequacy, proper consideration should be given to criteria related to separation, survivability and workability. In addition, careful construction techniques and good management of construction activities will greatly contribute to successful use of fabrics in unpaved roads and during the equivalent construction phase of paved roads

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