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Design of geotextile and geogrid reinforced unpaved roads

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ABSTRACT: A new nonlinear approach to design of geotextile and geogrid reinforced unpaved roads on clay subgrades is presented in this paper. This considers the nonlinear behaviour generally exhibited by most geotextile and geogrid reinforcing materials as well as clayey subgrade soils. The design method is based on a closed form plane strain analysis of the deformation track produced under wheel loading, considering the compatibility of rutting between the reinforcing element and the clayey subgrade soil. Analysis under traffic loading is produced by using the so called "equivalent material" approach to take into consideration the degradation of behaviour of the constituting materials under repeated traffic loading which are modelled using newly developed appropriate constitutive relations.

1 INTRODUCTION

Although several researchers have reported results of model and prototype tests showing beneficial effects of reinforcing on unpaved roads, Giroud and Noiray (1981) should be credited for presenting an elaborate numerical treatment of the problem using soil, reinforcement, loading and the relevant geometrical parameters. However, like traditional AASHTO method (using CBR) in their analyses the elastic and ultimate limiting states of the subgrade clay soil is considered to be developed without considering the state of the accompanying strains and displacements. The only soil parameter which is used is the undrained shear strength (c_u). Moreover, their design is based on a single valued modulus of reinforcing geotextiles or geogrids which normally exhibit nonlinear load-strain behaviour. Again almost all soft clays also show nonlinear deviator stress-strain behaviour. As two materials (clay and reinforcement), with different characteristic properties and load transfer mechanisms, are involved here in sharing the pressure transmitted from the wheels, a sound design method should consider the amounts as well as the degree of mobilization of resistances in the materials under compatible deformation conditions. Therefore, the design method reported in this paper is based on two considerations. These are:

1. Nonlinear behaviour of both the subgrade soil and reinforcing material under repeatative loading conditions.
2. Compatibility of rutting between subgrade soil and reinforcing elements under all serviceability conditions.

Giroud and Noiray did not consider these, which in some cases may lead to unsafe design.

In developing the proposed design method, the deformation of clay subgrade under wheel loading is analysed by considering plane-strain situation. This is performed by using an analytical scheme, incorporating the true deviator stress-strain behaviour of subgrade clay, suggested by Prakash, Saran and Sharan (1984). Hyperbolic (Kondner 1963) stress-strain - number of load repeatations behaviour of subgrade clays as well as reinforcing materials are used. Almost all the clays and reinforcing geotextiles and geogrids were found to obey this law to a reasonable degree of accuracy.

A generalised approach of representation of the nonlinear mechanical behaviour of clayey subgrade soils as well as those for reinforcing elements under repeated loading condition are established and presented. Analysis of behaviour under repeated traffic loading condition is achieved by using the equivalent material approach. This assumes the subgrade soils and reinforcing elements to be new (equivalent) materials possessing degraded mechanical behaviour which is appropriate for the number of load repeatations under consideration.

Design charts showing the required thicknesses of aggregates for different degrees of rutting appropriate for both unreinforced and reinforced conditions are presented.

These are prepared for one type of clay and one type of geotextile reinforcement only. A comparison of the new approach of design with the conventional Giroud and Noiray method is also presented. An example is also produced to show the applicability of the design methodology.

2 DESIGN PARAMETERS AND THEIR CHARACTERIZATION

The geometric and material parameters, relevent to design of reinforced unpaved roads, and their characterization procedure are described briefly in the following sections. To keep uniformity, the notations used by Giroud and Noiray are also used here.

2.1 Geometry and loading condition

A typical cross-section of an unpaved road, with appropriate notations, is shown in Figure 1. These are (Figure 1), ho = thickness (m) of aggregate layer required under unreinforced condition; h = thickness (m) of aggregate layer required under reinforced condition.

The geometrical description of wheel contact area of American-British standard equivalent axle may be described as follows (Figure 2). For on-highway trucks: $L = B/\sqrt{2}$ and $B = \sqrt{P/pc}$ and for off-highway trucks $L = B/2$ and $B = \sqrt{P\sqrt{2}/pc}$. Where, P = axle load (≐ 80 kN) and pc = tyre inflation pressure (= 620 kPa). B and L are the equivalent dimensions of dual wheel contact areas and associated area between them.

2.2 Properties of aggregates and subgrade soils

The aggregates are assumed to be well graded, to provide sufficient interlock and effective load distribution. These should have CBR (California Bearing Ratio) larger than 80.

The design philosophy presented here considers different degrees of rutting. Therefore, a single limiting value description, using either undrained shear strength (c_u) or California Bearing Ratio (CBR) for the subgrade clay soils which normally exhibit nonlinear stress-strain behaviour is not sufficient. Moreover, a realistic representation of behaviour of clay soils under repeated wheel loading can only be modelled through properties and parameters obtained from repeated loading triaxial tests. Such tests on representative samples under representative confining condition are used here to model the behaviour of the clayey subgrade

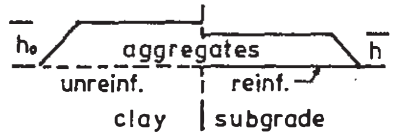

Fig. 1 Typical cross-section

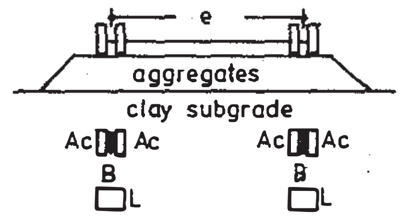

Fig. 2 Wheel contact area

soil. A new nonlinear constitutive relation for stress (σ) - strain(ε) - number of load repeatations (N) behaviour has been developed by the authors (Saha, 1988). Hyperbolic representation forwarded by Kondner (1963) was used to develop these relations. The $\sigma-\varepsilon-N$ relation for a silty clay, which is used to demonstrate the philosophy of the new design method is presented here as follows:

$$\sigma = \varepsilon\,(\sigma 1 - \log N/(c1 + c2\log N))/((a1 + c3\log N)\varepsilon 1 + (1 - a1 - c3\log N)\,\varepsilon) \qquad (1)$$

where $\sigma 1$ (=40 kPa), $\varepsilon 1$(=0.2) and a1(=0.198) are limiting values and c1(=0.105 kPa^{-1}), c2(=0.1098 kPa^{-1}) and c3(=0.02228) are calibration factors.

The $\sigma-\varepsilon$-N relations, for the clay under consideration, obtained from tests as well as those from equation (1) are presented in Figure 3(a) showing reasonable degree of agreement between them.

2.3 Properties of reinforcing elements

The load strain behaviour of most geotextile and geogrid reinforcing materials exhibit nonlinearity under wide ranges of strain levels. Therefore a single modulus representation (2 percent strain level) may lead to considerable error in the analyses. Load (Pr) -strain (εr) - number of load repeatations (N) relations similar to those suggested for clays were also developed using hyperbolic representation. Testing techniques for evaluation of load-strain behaviour of geotextiles and geogrids respectively, have been detailed by McGown, Andrawes and Kabir (1982) and McGown, et. al (1984). Similar appropriate techniques, with slight modification for performing tests under repeated loading was adopted for this study (Saha, 1988).

A woven tape geotextile constructed of polypropylene tapes having weight per unit area of 240 gsm, was used to demonstrate the design philosophy. The Pr - εr - N relation of the geotextile, is presented as follows:

$$Pr = \frac{\varepsilon r(Pr1 - \log N/(r1 + r2\log N))}{(br + r3\log N)\,\varepsilon r1 + (1 - br - r3\log N)\,\varepsilon r} \qquad (2)$$

Where Pr1(=54.5 kN/m), εr1(=0.1) and br(= 0.55) are limiting values and r1(=0.3354 m/kN), r2(=-0.03357 m/kN) and r3(=0.0406) are calibration factors.

The Pr- εr-N relations for the geotextile, from tests as well as those from equation (2) are presented in Figure 3(b), showing reasonable degree of agreement between them.

3 ANALYSIS OF BEHAVIOUR OF ROAD STRUCTURE

Giroud and Noiray (1981) used two separate analyses, one is for very light traffic and the other is appropriate for heavy traffic. For light traffic they used the so called quasi-static analysis in which soil and reinforcement data from monotonic loading tests are used. For analysis under heavy traffic loading they used a modification of the results obtained from quasi-static analysis by using empirical equation given by Webster and Alford (1978), based on in-situ test data produced by Hammit (1970). In contrast, the analyses poduced here uses unified approach for any intensity of traffic loading. These are based on newly developed soil and reinforcement constitutive relations obtained from repeatative loading tests which were described in section 2.

3.1 Load distribution through aggregate layer

A pyramidal distribution of load from the wheels through aggregate layer is assumed. The load distribution pattern and the geometrical representation of the problem for cases with or without reinforcement are presented in Figure 4. The pressure at the base of the aggregate layer, due to wheel loading from off-highway trucks, may be represented as follows (Figure 4). For cases without reinforcement, the pressure is:

$$po = P/2(B+2ho\ \tan\alpha o)(B/2+2ho\ \tan\alpha o) + \gamma ho \qquad (3)$$

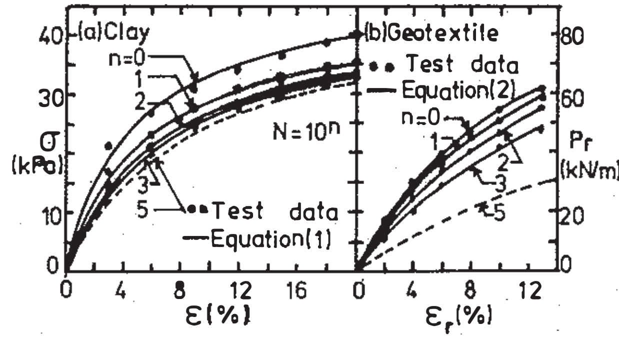

Fig. 3 Mechanical behaviour

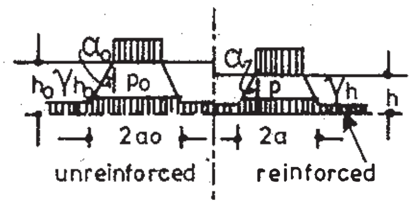

Fig. 4 Pressure distribution

For cases with reinforcement, the pressure is:

$$p = P/2(B+2h\ \tan\alpha)(B/2+2h\ \tan\alpha) + \gamma h \qquad (4)$$

In line with Giroud and Noiray recommendation it is assumed that tan α o=tan α =0.6. These values are used in all subsequent calculations presented here.

3.2 Behaviour of clay subgrade

The clay subgrade is assumed to be semi-infinite, homogeneous and isotropic. The load deformation behaviour of clay subgrade under loading from wheels transfered through the aggregate layer is obtained by using the calculation scheme suggested by Prakash, Saran and Sharan (1984). The method utilizes two parameter hyperbolic deviator stress-strain relationship for clay soils. These are obtained from the constitutive relations σ - ε -N described earlier. In this method the clay layer beneath the loaded area is divided into a number of horizontal slices. The total deformation is then obtained as the summation of deformations of each slice which is denoted as 'S'. The calculations are repeated for different values of width of loaded area (2a) and prssure on loaded area (po). The plot of pressure, po and the ratio δ (=S/2a) is normally found to trace a unique curve. Prakash, Saran and Sharan found very good agreement amongst results obtained from, this method, finite element analysis and test data.

The po - δ curves for, different values of load repeatation (N), for the clay, described earlier, are presented in Figure 5(b). The hyperbolic representation of the po - δ relations are also presented in Figure 5(b).

3.3 Geometry of deformation

The wavy shape of the deformed unpaved road structure results from incompressibility of the saturated clay subgrade. Therefore, the volume of soil displaced downwards due to wheel loading must be equal to that displaced upward due to heaving. Schematic diagrams of unreinforced and reinforced unpaved road structures are presented in Figures 5(a) and 6(a) respectively. The wavy concave (AB) and convex (BB) parts were found to resemble arcs of parabolae by Webster and Watkins (1977), from full scale tests. The geometry of the three possible modes of deformation may be represented mathematically as follows (Figures 5(a) and 6(a)). Where, r = rut depth.

Model: $a' > a,\ S = ra'/(a+a')$ (5)

Mode2: $a' < a,\ S = 2ra^2/(2a^2+3aa'-a'^2)$ (6)

Mode3: $a' = a,\ S = r/2$ (7)

3.4 Unpaved road without reinforcement

The design of unpaved road without reinforcement involve determination of thickness of aggregate layer for different degrees of rutting. It has been discussed earlier that po-δ relation for clay subgrades may be represented by hyperbolic functions. Therefore, the resistance, po, of the clay subgrade may be represented as:

$$po = \delta/(s1+s2\,\delta)+\gamma\,ho \quad (8)$$

where, s1 and s2 are hyperbolic parameters (Figure 5(b)) for clay subgrade appropriate for the number of load repeatation (N). Equating equations (3) and (8) and putting $\delta = S/2.ao$.

$$P/(2(\sqrt{P\sqrt{2}/pc}+1.2ho)(\sqrt{P/2pc\sqrt{2}}+1.2ho))$$

$$= S/(2\,ao.s1+s2S) \quad (9)$$

Values of 'S' as a function of 'r' may be obtained from equations (5), (6) and (7) depending on the mode of deformation. Design charts giving thicknesses (ho) of agreegate layer for different degrees of rutting may now be produced from equation (9) using appropriate soil parameters s1 and s2. Such chart for unpaved road on the clay subgrade

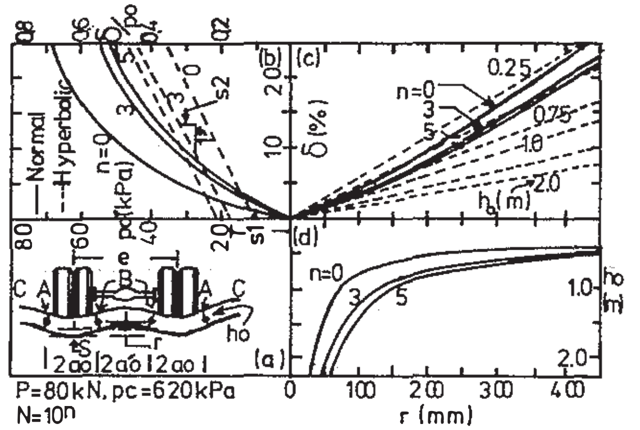

Fig. 5 Design chart for unreinforced unpaved road

for different intensities of traffic loading (N) conditions are produced in Figures 5(c) and (d).

3.5 Unpaved road with reinforcement

The use of reinforcement in an unpaved road structure may be described as producing two beneficial effects. These are:

1. Confinement of subgrade soils between and beyond the wheels, resulting in a system offering additional resistance against rutting.
2. Releaving the subgrade soil of some pressure transmitted from the wheels, thereby increasing the overall carrying capacity.

In this analysis, like that by Giroud and Noiray, the pressure releaving effect is only considered.

From equilibrium consideration, the pressure, p, transmitted from the wheels through the aggregates on part AB (Figure 6(a)) of the reinforced road should be equal to:

$$p = ps + pr \quad (10)$$

where, ps and pr are mobilised reactions (pressure) in the clay subgrade and the reinforcement respectively, appropriate for the intensity of traffic loading (N), under the same degree of rutting. Therefore, determination of ps and pr should be based on analysis of the kinematics of the deformation track produced due to wheel loading. The compatibility of rutting was not considered by Giroud and Noiray in developing their design charts.

In reinforced unpaved road, for a known degree of rutting, the pressure shared by the clay subgrade may be determined from the following equation:

$$ps = \delta/(s1+s2\,\delta)+\gamma\,h \quad (11)$$

where $\delta = S/2a$ and the values of 'S' as a function of rut depth may be obtained from equations (5), (6) and (7). The force due to pressure, pr acting on zone AB (Figure 6(a)) is equivalent to the vertical component of the tension Pr in the reinforcing element acting at points A and B. Therefore:

$$a.pr = Pr.\cos\beta \tag{12}$$

From property of parabola $\cos\beta = 4\sqrt{\delta^2/(16\delta^2+1)}$. Again from hyperbolic load strain representation of reinforcing elements:

$$Pr = \varepsilon r/(b1+b2\varepsilon r) \tag{13}$$

The parameters b1 and b2, are dependent on the characteristics of reinforcing element and intensity of traffic loading (N). Values of b1 and b2 may readily be determined from equations (2), (12) and (13). Putting values of $\cos\beta$ and Pr in equation (12):

$$a.pr = (4\varepsilon r/(b1+b2\varepsilon r))(\sqrt{\delta^2/(16\delta^2+1)}) \tag{14}$$

From the assumption of parabolic deformation the strain in the reinforcing element may be expressed as:

$$\varepsilon r = (1/2)(\sqrt{1+16\delta^2}+(1/4\delta)(\ln(4\delta+\sqrt{1+16\delta^2})-2) \tag{15}$$

From equations (14) and (15) it can be found that pr.a may be expressed as a function of δ only for different values of N. pr.a – δ plots for the reinforcing element for different intensities of traffic loading (N), are presented in Figure 6(b). The pressure pr may be obtained from equation (14) as:

$$pr = (4\varepsilon r/a(b1+b2\varepsilon r))(\sqrt{\delta^2/(16\delta^2+1)}) \tag{16}$$

Using equations (4),(11) and (16), equation (10) takes the form:

$$P/2(\sqrt{P\sqrt{2}/pc}+1.2h)(\sqrt{P/2pc\sqrt{2}}+1.2h))=\delta/(s1+s2\delta)$$
$$+(4\varepsilon r/a(b1+b2\varepsilon r))(\sqrt{\delta^2/(16\delta^2+1)}) \tag{17}$$

The parameters εr and δ are functions of the degree of rutting. Design charts giving required thicknesses of aggregates, 'h' due to different degrees of rutting, may now be produced from solution of equation (17).

The design charts for the reinforcing element on the clay subgrade for different intensities of traffic loading (N) are presented in Figure 6(c) and (d).

3.6 Design example

To show the applicability of the suggested method, the following design example is produced. A reinforced unpaved road should be designed for American-British Standard, off-highway truck with, P = 80 kN, pc = 620 kPa, N = 1000 passages and r = 190 mm, on a clay subgrade and reinforcing element of the type described in this paper. Using Figure 6 or 7, for r = 190 mm and N = 1000; the required thickness of aggregate layer, h , are obtained as 0.31 m and 0.60 m by Giroud and Noiray method and the authors' method respectively. This show the Giroud and Noiray method to yield result on the unsafe side.

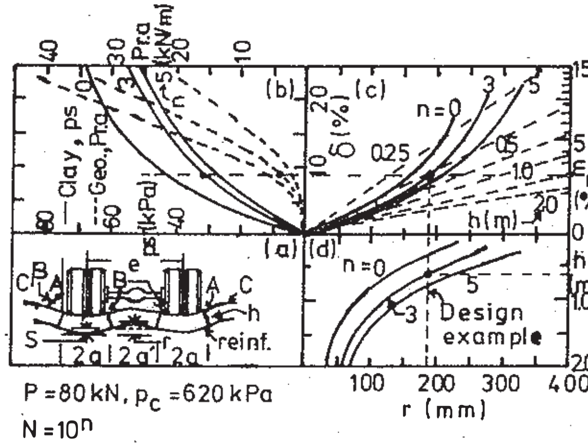

Fig. 6 Design chart for reinforced unpaved road

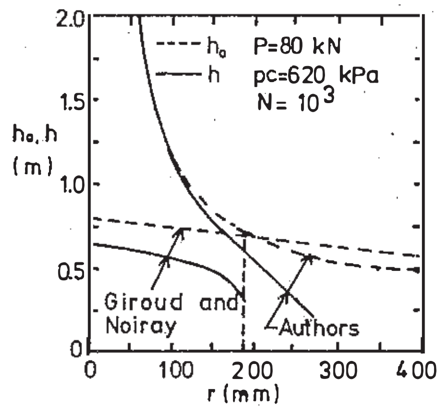

Fig. 7 Designs by authors' and Giroud and Noiray (1981) methods

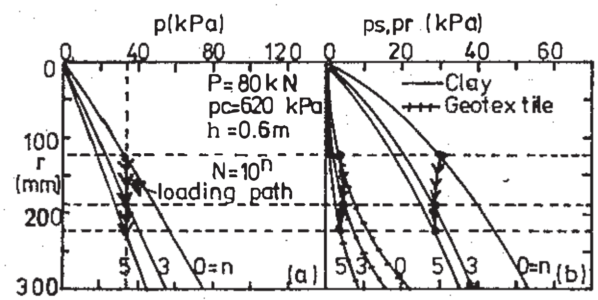

Fig. 8 Load sharing by clay and reinforcement

Table 1. Authors refinements over Giroud and Noiray (1981) method.

Area of refinement	Basic design parameters		Quasi-static (Q.S) analysis			Analysis under traffic loading	
Design method	Subgrade	Reinf.	Unreinf.	Reinf.		Unreinf.	Reinf.
				Subgrade	Reinf.		
Giroud & Noiray (1981)	c_u	K (modulus at εr=2%)	$Po=\pi c_u$ $\neq f(r)$	$ps=(\pi+2)c_u$ $\neq f(r)$	pr= f(K,r)	po, ps, pr from Q.S.+ Webster and Alford (1978) equation.	
Authors	New σ-ε-N relation	New Pr- εr-N relation	For clay: po,ps=f(σ-ε -N,r) obtained from po or ps -δ - N relation using Prakash, Saran and Sharan (1984) method. For reinf.: pr = f(Pr- εr-N, r) from pr-δ-N relation.				

4 DISCUSSION AND CONCLUSIONS

Giroud (1982) in his closing remarks on discussion on the paper by Giroud and Noiray (1981) stated, "one of the goals of the authors of the original paper was to carefully document their approach so it can be used by researchers as a starting point for the development of improved methods of design". In line with Giroud's thought the method of analysis presented here may be considered as an improvement over that presented by Giroud and Noiray. The refinements introduced by the authors on their method are summarised in Table 1. The Giroud and Noiray method would produce results on the unsafe side, which is shown to calculate lower values of h (Figure 7), compared with those by the authors. To depict the load sharing mechanism of the subgrade soil and the reinforcement under repeated loading, the loading path appropriate for the design example is shown in Figure 8(a) and (b). These show the clay subgrade to loose load slightly and the reinforcing element to gain it, relative to their respective initial values.

From the studies presented in this paper the following conclusions could be made:

1. A single parameter representation of behaviour of both clay subgrade and reinforcing element is not adequate for design of reinforced and unreinforced unpaved roads.
2. In design of reinforced and unreinforced unpaved roads, the newly developed constitutive relations may be used to represent the behaviour of both clay subgrade and reinforcing element to a reasonable degree of accuracy.
3. Assumption of mobilization of elastic (in case of unreinforced road) and ultimate (in case of reinforced road) limit states irrespective of degrees of rutting may lead to unsafe design, especially at low values of rutting.
4. Overall, the design method proposed here is based on realistic representation of behaviour of both clay subgrade and reinforcing element under compatible degrees of rutting and therefore, may be adopted.

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