A study about geosynthetic – reinforced foundation

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ABSTRACT: In 1998 Madhav and Poorooshasb (1998) published a model for geosynthetic reinforced foundation based in Pasternak (1954) shear layer model and Winkler model to represent granular soil layer and soft soil layer, and a tensioned membrane effect to represent the geosynthetic, which is make possible to predict the settlement response of the reinforced soil. After this Shukla and Chandra (1994) studied the model of Madhav and Poorooshasb (1988) and extended them to analyze the effects of prestressing of geosynthetic, the effects of compressibility of granular soil layer, the effects of consolidation of soft soil, and the effects of compaction in the settlement response. This paper present a brief discussion of this methods, and make a comparison of the results obtained with methods, discussing the agreement of them with finite element analysis.

1 INTRODUCTION

One of the great difficulties in designing of foundations reinforced with geosynthetics is in the correct determination of the structure settlement. The great number of variables involved in the process, such as number of reinforcement layers and its placement, among others (Fabrin, 1999) unable the use of the classic theories on the determination of settlement.

Some proposals for determination of settlement of foundations reinforced with geosynthetic as in Szalatkay (1986) depend on parameters rarely obtained in practice, for example soil stiffness considering the reinforcement effect

Thus, the majority of design methods, such as the proposed by Binquet and Lee (1975), Soni (1994), Das et al. (1996) and Wayne (1998), don't allow the settlement determination nor they consider the reinforcement deformation on their calculation.

In 1988 Madhav and Poorooshasb (1988) published the model for geosynthetic reinforced foundation to predict the settlement response of the reinforced soil. Shukla and Chandra (1994) studied this model and extended them to analyze the effect of prestressing of geosynthetic, the effect of compressibility of granular soil layer, the effects of consolidation of soft soil, and the effect of compaction in the settlement response.

The papers cited above present the parametrical study of the methods, but none of them show a comparison of the methods with finite element analysis or laboratory model tests.

The subject of this paper is to compare the results obtained by the proposed models with results of finite element analysis made using PLAXIS software, which have a special element to simulate geosynthetic considering even the friction of geosynthetic with soil.

2 MADHAV & POOROOSHASB MODEL

2.1 *Theoretical formulation*

The geosynthetic reinforced foundation design method proposed by Madhav and Poorooshasb (1988) considers in its model a soft soil of great thickness on which an incompressible granular layer of finite thickness is placed with geosynthetics inserted within.

The granular soil is represented by the Pasternak (1954) shear layer model and the soft soil is represented by Winkler model. The reinforcement is represented by a tensile stressed membrane. The Figure 1 illustrates the sketch of model.

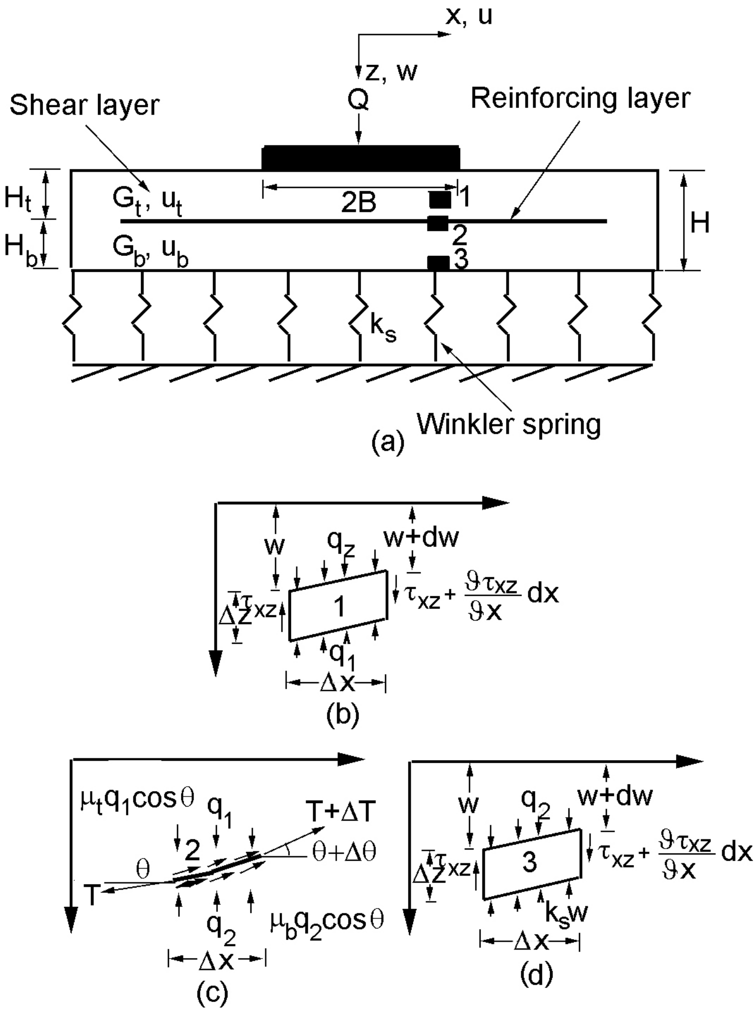

Figure 1. Definition sketch (Ghosh and Madhav, 1988)

Doing the balance of vertical forces for each element of the Figure 1 the following equations are obtained:

$$q = q_1 - G_t H_t \frac{d^2 w}{dx^2} \tag{1}$$

$$q_1 = q_2 - T\cos\theta \frac{d^2w}{dx^2} \tag{2}$$

$$\frac{dT}{dx} = -(\mu_t q_1 + \mu_b q_2)\sec\theta - (q_1 - q_2)\sin\theta \tag{3}$$

$$q_2 = q_3 - G_b H_b \frac{d^2w}{dx^2} \tag{4}$$

$$q_3 = kw \tag{5}$$

where μ_t and μ_b are the soil reinforcement friction coefficients of the upper and lower faces, respectively; w the surface settlement; q the applied overload; G_t and G_b the upper and lower soil shearing reinforcement moduli, respectively; k the soil reaction modulus; and θ is defined by:

$$\tan\theta = \frac{dw}{dx} \tag{6}$$

Combining the equations 1 through 5 general equations that govern the model is given:

$$q = kw - (G_t H_t + T\cos\theta + G_b H_b)\frac{d^2w}{dx^2} \tag{7}$$

$$\frac{dT}{dx} = -(\mu_t \sec\theta + \mathrm{sen}\theta)\left(q + G_t H_t \frac{d^2w}{dx^2}\right) - (\mu_b \sec\theta - \mathrm{sen}\theta)\left(kw - G_{bt} H_b \frac{d^2w}{dx^2}\right) \tag{8}$$

The equations 7 and 8 should be numerically solved in an interactive manner, obeying the following boundary conditions:

$x = 0$, $d_w/d_x = 0$ and $x = L$, $d_w/d_x = 0$ and $T = 0$.

2.2 *Model analysis*

In the analyses of the model the following criteria and parameters were used: reinforcement length equal to two times the foundation width, G_t=G_b=10,000kN/m², μ_t=μ_b=0.5, k_s=5,000kN/m³, H_t=0.5m, H_b=0, q=100kN/m².

The parametric analysis of the Madhav and Poorooshasb (1988) method reveals that variation on soil strength parameters, i.e. shearing modulus G and soil reaction modulus k, have little influence on the obtained values of stress in the reinforcement, as well as the thickness of the granular layer H. The Figure 2 shows the variation of the stress in the reinforcement with the shearing modulus G and the thickness of the granular layer H. This type of behavior doesn't reflect the real structure behavior, because according to Fabrin (1999), variations in the soil stiffness have great influence in the reinforcement solicitation.

It is verified by parametric analysis that the parameters that influence the tensile stress values acting on the reinforcement, obtained through the analyzed models are: the applied overload and the soil-reinforcement friction coefficient, taking into account that a criterious attitude should be taken when the last one is chosen, due to the great influence that this one has upon the stress value, as shown on Table 1.

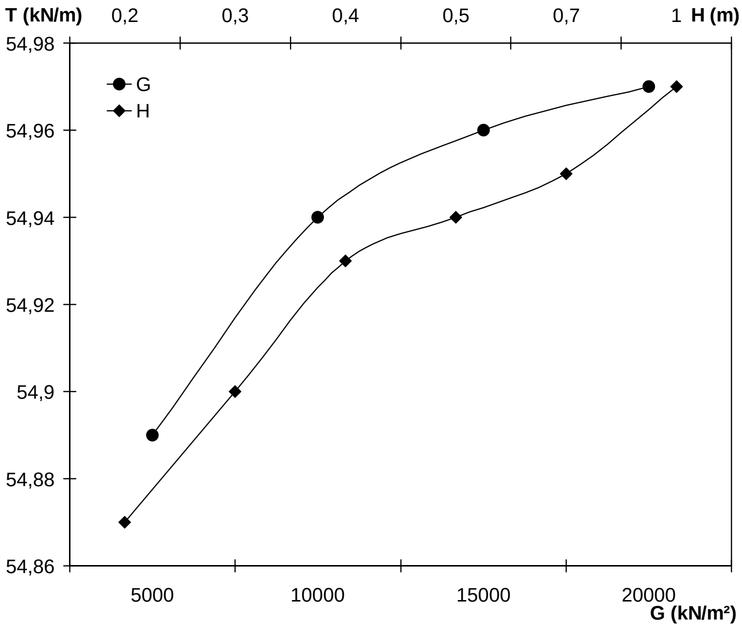

Figure 2. Reinforcement tensile stress variation function of shear modulus and granular layer thickness.

Table 1. Reinforcement tensile stress variation function of soil/reinforcement friction angle.

μ	T (kN/m)
0.2	22.0
0.5	54.9
0.7	76.7
1.0	109.5

It is interesting to point out that the inclusion of the reinforcement doesn't change the settlement on soil surface, such behavior is contrary to what is in the literature (Watari et al., 1986; Dawson and Lee, 1988; Omar et al., 1993; Rao et al., 1994; Kenny, 1998) that relates the reinforcement inclusion to a reduction of the surface settlement.

3 SHUKLA & CHANDRA MODEL

3.1 *Theoretical formulation*

The Shukla and Chandra (1994) model is an extension of the Madhav and Poorooshasb (1988) model in which the authors had included the pre-stressing effects on the reinforcement, compressibility and compaction of the granular layer, and the effects of consolidation of soft soil.

The granular soil is represented by Pasternak model (1954). The compressibility of this soil layer is represented by a Winkler springs range, positioned in the bottom of the layer, with stiffness higher than the one of the lower layer, as shown on Figure 3.

The soft soil is considered saturated and obeying the Terzaghi's (1943) consolidation theory hypothesis, modeled by Winkler springs (constant value with soil depth and time), and small pistons, where the springs represent the soil structure, and the pistons the dissipation of pore pressure (Figure 3).

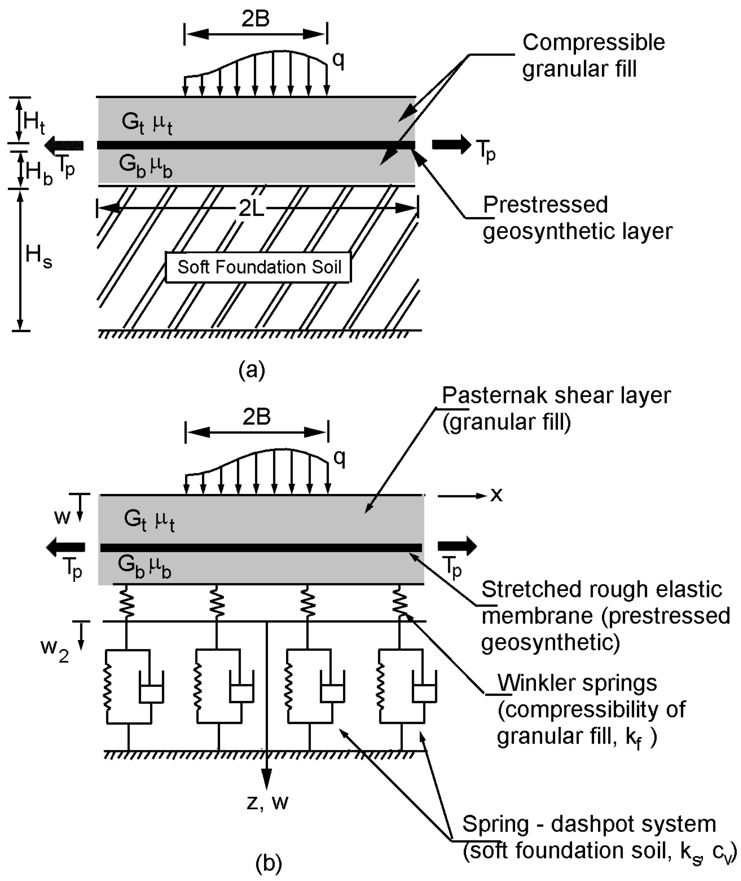

Figure 3. Layout of model concept by Shukla and Chandra (1994).

The equations that govern the problem are:

$$q = \overline{X}_1 \frac{k_f}{k_s + k_f} \frac{k_s w}{U} - \left[G_t H_t + \overline{X}_2 \left(T_p + T\right)\cos\theta + \overline{X}_1 G_b H_b\right] \frac{d^2 w}{dx^2} \tag{9}$$

$$\frac{dT}{dx} = -\overline{X}_3 \left(q + G_t H_t \frac{d^2 w}{dx^2}\right) - \overline{X}_4 \left(\frac{k_f}{k_s + k_f} \frac{k_s w}{U} - G_b H_b \frac{d^2 w}{dx^2}\right) \tag{10}$$

with:

$$\overline{X}_1 = \frac{1 + K_{0R} \tan^2\theta - \left(1 - K_{0R}\right)\mu_b \tan\theta}{1 + K_{0R} \tan^2\theta - \left(1 - K_{0R}\right)\mu_t \tan\theta} \tag{11}$$

$$\overline{X}_2 = \frac{1}{1 + K_{0R} \tan^2\theta - \left(1 - K_{0R}\right)\mu_t \tan\theta} \tag{12}$$

$$\overline{X}_3 = \mu_t \cos\theta\left(1 + K_{0R} \tan^2\theta\right) - \left(1 - K_{0R}\right)\mathrm{sen}\theta \tag{13}$$

$$\overline{X}_4 = \mu_b \cos\theta\left(1 + K_{0R} \tan^2\theta\right) - \left(1 - K_{0R}\right)\mathrm{sen}\theta \tag{14}$$

$$K_{0R} = (1 - \text{sen}\phi)(OCR)^{\text{sen}\phi} \tag{15}$$

where k_f is the granular soil reaction modulus, k_s the soft soil reaction modulus, U the soil consolidation ratio, T_p reinforcement pretension, K_0 coefficient of lateral stress at rest, OCR overconsolidation ratio and ϕ the friction angle of the soft soil.

3.2 *Model analysis*

In the analysis of the Shukla and Chandra (1994) model besides the parameters used in the Madhav and Poorooshasb (1988) analysis the following parameters were used: U=0, K_{0R}=0.7; T_P=0 e k_f=20,000kN/m³.

In the analysis of the model proposed by Shukla and Chandra (1994) the same problems of the model of Madhav and Poorooshasb (1988) is detected. It is verified that variations in the reinforcement pretension, consolidation rate of the soil, reaction modulus of the granular layer and K_{0R} values practically don't alter the values of the reinforcement mobilized tension, presenting a behavior contrary to the expected.

In terms of settlement, K_{0R} and reinforce pretension variations don't change the values of it, reinforcing the hypothesis that the settlement behavior on the proposed models is only governed by the models of Pasternak (1954) and Winkler, not suffering any influence of the tensioned membrane model.

Tables 2 and 3 present the settlement and reinforcement stress behavior at the center of the foundation for some values of reinforcement pretension and K_{0R}.

Table 2. Settlement and reinforcement tensile stress function of pretesion obtained by the Shukla and Chandra (1994) model.

Tp (kN/m)	w (cm)	T (kN/m)
0	1.4	58.7
20	1.4	58.7
50	1.4	58.7
100	1.4	58.7

Table 3. Settlement and reinforcement tensile stress function of K_{0R} obtained by the Shukla and Chandra (1994) model.

K_{0R}	W (cm)	T (kN/m)
0	1.4	58.6
0.4	1.4	58.7
1.0	1.4	58.7
1.5	1.4	58.7
2.0	1.4	58.8

A more detailed analysis of both methods reveals that the sensibility of both to the input parameters is related to the relative stiffness (G/k). It is observed that these methods only begin to present sensibility in the input parameters when the relationship:

$$G^* = \frac{GH}{kB^2} \leq 1 \tag{16}$$

where B is equal the half width of foundation.

To satisfy this relationship it is necessary to take into account the following situation, usually not verified in practice:

- Great foundation widths;
- Extremely thick granular layers (smaller than B);
- Stiffness of the granular material equal or lower than the soft soil stiffness.

4 NUMERICAL ANALYSIS

The numerical analysis was accomplished using Plaxis (Brinkgreve and Vermeer, 1998) finite elements software. The granular soil was represented by the Hardening Soil model and the soft soil by the Linear Elastic model (in order to represent with better accuracy the Winkler model), using the calculation procedure with updating of the mesh coordinates, recommended for cases of great deformations (Brinkgreve and Vermeer, 1998).

The model was configured according to what is presented on Figure 4, with the reinforcement positioned in the interface of the soils, because, as verified by Fabrin (1999) this is the best positioning of the reinforcement dealing with situation of granular soil over soft soil, where, according to the author, the granular material behavior as a flexible beam in the which the geosynthetic works as the beam tension bars.

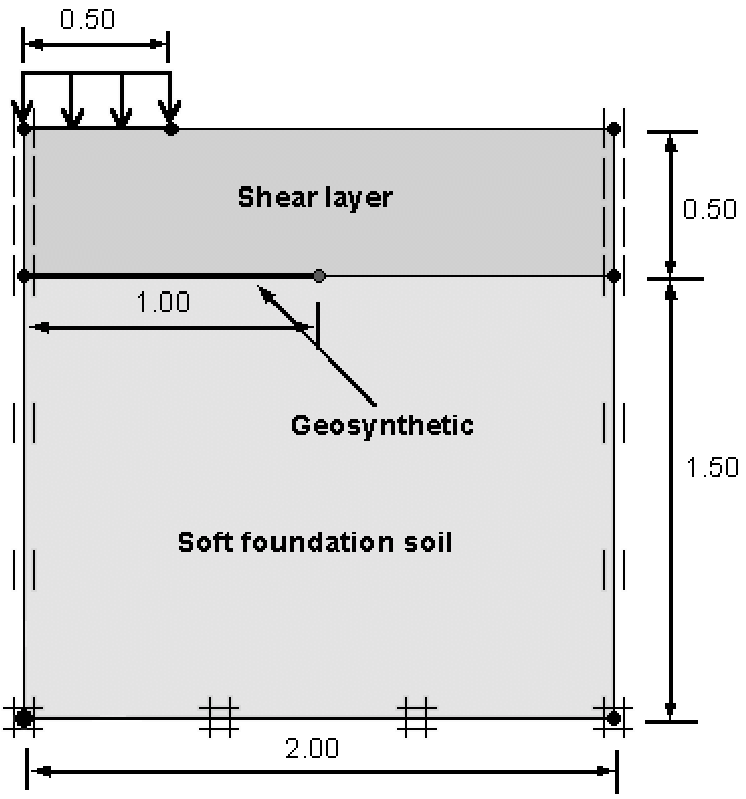

Figure 4. Layout of model discretized by numerical analysis.

The soil parameters used in the analyses are presented on the Table 4 and the reinforcement parameters are presented on the Table 5.

The analyses are cited on the following text using Aijk convention, where "i" refers to the soft soil, "j" to the granular soil and "k" to the reinforcement used in the analysis.

Table 4. Soil parameters used in the numeric analysis

Soil	γ (kN/m³)	E (MPa)	c (kN/m²)	ϕ (°)
SS1	15.8	3	--	--
SS2	17.7	5	--	--
GS1	17.9	33.3	1	27
GS2	20.9	77.7	1	35

SS = soft soil; GS = granular soil

Table 5. Reinforcement parameters used in the numeric analysis

Reinforcement	J (kN/m)*
R1	600
R2	4000

* secant tensile modulus at 5% of deformation

5 COMPARATIVE ANALYSIS

5.1 *Madhav & Poorooshasb model*

Table 6 presents the comparison of the results obtained by numeric analysis with the ones obtained by the Madhav and Poorooshasb (1988) method, for values obtained at the foundation center, with an applied overload of 100kN/m². The surface settlement values (w_s), reinforcement vertical strain (w_r) and reinforcement tensile stress (T) are presented.

Table 6. Comparison of obtained values of w e T.

Analyse	FEM			Madhav e Poorooshasb (1988)	
	w_s (cm)	w_r (cm)	T (kN/m)	W_s (cm)	T (kN/m)
A111	3.0	2.5	4.6	1.6	38.5
A112	2.9	2.5	10.7	1.6	38.5
A121	2.6	2.1	3.6	1.6	38.5
A122	2.5	2.1	9.6	1.6	38.5
A211*	0.8	0.5	1.2	0.4	15.4
A212*	0.8	0.5	3.6	0.4	15.4
A221	1.7	1.3	2.5	1.0	38.5
A222	1.7	1.3	7.8	1.0	38.5

* surcharge of 40kN/m²

The Table 6 presents data where can be observed that the variation in the soil foundation parameters doesn't alter the values of settlement and reinforcement stress obtained by the Madhav and Poorooshab (1988) method. In the matter of granular soil variations it is observed that this one affects only settlement, not changing the behavior of reinforcement mobilized tension.

Another point that should be highlighted is the discrepancy of the values obtained by numeric analysis with the ones obtained by analytic method. A tendency of underestimate the settlement values and overestimate the reinforcement stress values is observed

It is observed through the values presented on the Table 6 that the Madhav and Poorooshasb (1988) shows a rigid reinforcements behavior and not the one of flexible reinforcements, modeled according to the theory of tensile membrane. Numeric analyses using SS1 and GS1 soils with reinforcement of 60,000kN/m of stiffness values showed a reinforcement mobilized tension value T of only 14.4kN/m, far below the one obtained by the Madhav and Poorooshasb (1988) that was of 38.5kN/m.

It is verified, through the Figure 5, that the settlement behavior obtained by the Madhav and Poorooshab (1988) comes completely different in comparison with the soil surface settlement and the reinforcement vertical strain obtained through numeric analysis.

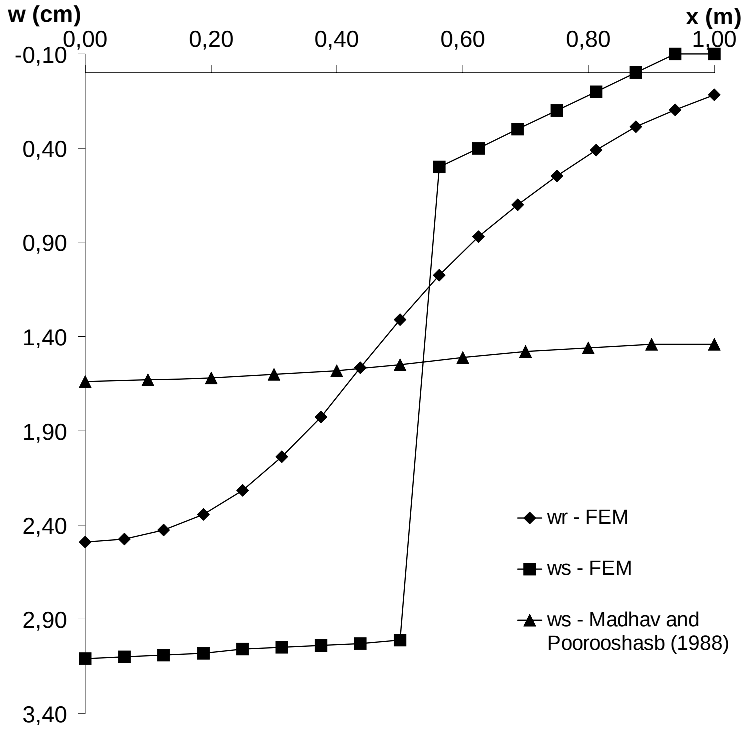

Figure 5. Settlement behavior – case A111.

It is observed that there was rupture by punching of the granular material with failure plan very close to the vertical plan passing through the foundation boundary in the numeric model, agreeing with the Meyerhof (1974) proposal for foundation in granular soil over soft soil.

It is verified that the behavior of settlement as idealized by Madhav and Poorooshasb (1988) only happens for cases in that the relationship (16) is satisfied. The Figure 6 presents the settlement behavior for the situation of granular soil stiffness G=10.000kN/m², soft soil reaction modulus k=8,000kN/m³ and reinforcement R2, with B=H=0.5m and q=40kN/m², what implicates in G*=2.5.

It is verified by the Figure 6 a good approach of the surface settlement values obtained by Madhav and Poorooshasb (1988) with the ones obtained by finite elements for the strip directly bellow the foundation (0 <x <0.5), however a discrepancy of the values for the strip outside the foundation (x>0.5).

It is observed in the comparison of the settlement behavior from Figures 5 and 6 as long there is a G* value reduction (G*=8 in Figure 5 and G*=2.5 in Figure 6), there is a closeness on the settlement behavior and values obtained by Madhav and Poorooshasb (1988) with the one obtained by finite elements.

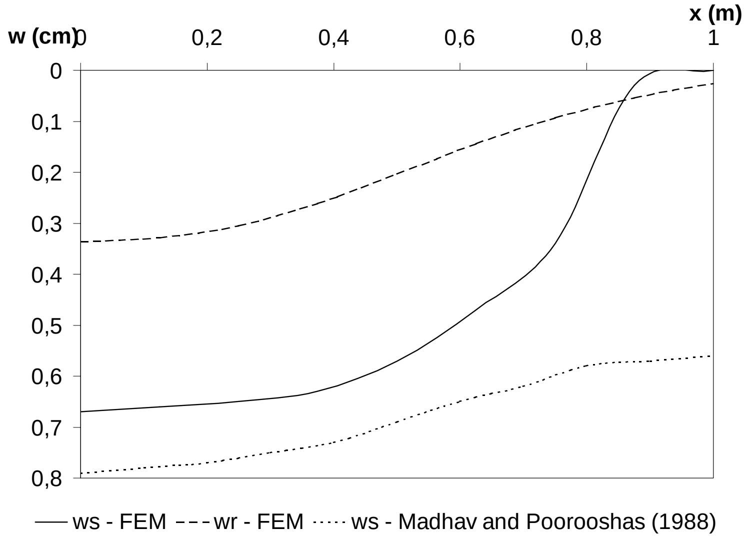

Figure 6. Settlement behavior for G^*=2.5.

5.2 *Shukla & Chandra Model*

In the study of the Shukla and Chandra (1994) method it was just studied the effects of the compressibility of the granular layer in numeric analysis. As presented above , variations of K_{0R} and T_p don't present significant influences on settlement values and on the reinforcement stress, not justifying then, an analysis about the variation on these parameters.

Comparisons between the Shukla and Chandra (1994) method and the finite elements method considering the consolidation effects was not accomplished due to the Shukla and Chandra (1994) method to consider the consolidation according to the Terzaghi's (1943) consolidation theory and the finite elements method to consider bi-directional consolidation (Binkgreve and Vermeer, 1998), not being justified a comparison between both.

Table 7 presents the comparison of the results obtained in numerical analysis with the ones obtained by Shukla and Chandra (1994), where just the variation of the compressibility modulus of the granular layer k_f was consider.

It is observed that the Shukla and Chandra (1994) method doesn't present any variation on the reinforcement tensile stress values and settlement, independently of the analyzed soil. In general, the method presents a tendency to underestimate the settlement values and to overestimate the reinforcement tensile stress values.

Because it is an extension of the Madhav and Poorooshasb (1988) method the Shukla and Chandra (1994) method presents the same behavior of the first method, being here valid the analyses and comments shown on the item 5.1.

Table 7. Comparison of obtained values

Ana-lyze	FEM			Shukla e Chandra (1994)	
	w_s (cm)	w_r (cm)	T (kN/m)	W_s (cm)	T (kN/m)
A111	3.0	2.5	4.6	0.3	40.5
A112	2.9	2.5	10.7	0.3	40.5
A121	2.6	2.1	3.6	0.3	40.5
A122	2.5	2.1	9.6	0.3	40.5
A211*	0.8	0.5	1.2	0.1	16.2
A212*	0.8	0.5	3.6	0.1	16.2
A221	1.7	1.3	2.5	0.3	40.5
A222	1.7	1.3	7.8	0.3	40.5

* surcharge of 40kN/m²

6 CONCLUSION

This paper presents a general overview of design methods for geosynthetic reinforced foundation proposed by Madhav and Poorooshasb (1988) and Shukla and Chandra (1994). Both methods allow the determination of as much the reinforcement tensile stress as the soil settlement values, where one can observe that:

- In terms of reinforcement tensile stresses both methods present very high values comparing to the ones observed in the numerical analysis.
- In terms of settlement, it is observed that the closeness of the values obtained by the analytic methods with the numeric analysis, it is a function of the relative stiffness (G/k) between the granular and soft soils. For situations of low relative stiffness the analytic and numeric values tend to a closeness in the area directly below the foundation.

Therefore, the conclusion is that the methods are not ideals for determination of the reinforcement tensile stress, but they are good tools of easy application for an initial estimate of the foundation settlement, specially on situations of low relative stiffness.

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