

CONSTITUTIVE MODELLING FOR TROPICAL RESIDUAL SOILS WITH MODIFIED ROTATIONAL HARDENING PARAMETER

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Abstract

The proposed constitutive model is a generalized model to explain the stress-strain behaviour of soils encompassing unsaturated and saturated states of natural soils and also soft clays. The previously proposed models had treated the unsaturated and saturated states separately. The model is intended for tropical residual soils. The explicit nature of the stress-strain response of tropical residual soil mostly depends on the fabric and nature of cement bonding in addition to the usual factors such as current state, stress history, stress path, and drainage conditions. Despite the availability of quite a good number of constitutive relations concerning the behaviour of clays, the strain softening behaviour of tropical residual soils during the deformation process has not been satisfactorily tackled. The model is proposed within the framework of hardening plasticity. The rotational hardening parameter of modified Wheeler's model is modified in the proposed model. The method allows for calibrating model parameters adopted in the model in addition to being an independent method to model the strain softening behaviour. This approach has the twin advantages of combining the conventional hyperbolic approach to the critical state concept and calibrating the model parameters adopted in the model for use in other stress paths. The model was validated by the analysis of data available in the literature to reproduce quantitatively, the observed behavior. The concordance between observed and computed results is regarded as satisfactory and supports the likelihood of duplicating the key characteristics of soils found in many states using a simple general framework. After formulation of the model for isotropic and triaxial stress states, typical predictions are provided and compared qualitatively with typical trends of tropical soil behaviour.

Keywords: Critical state; Hyperbolic approach; Strain softening; Stress-Strain; Wheeler model

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Introduction

Tropical soils appear in large regions of the world and have been less studied particularly with respect to critical state and limit state conditions. Most geomaterials are structured in nature and this natural structure affects the behavior of tropical soils. Structural features affecting soil behavior include soil cementation and soil fabric.

The basic concept and methodology of critical state soil mechanics (Roscoe and Burland, 1968; Schofield and Wroth, 1968) has underlined most soil constitutive models are based on an elastoplasticity framework. Notably, most isotropic models do exhibit one way or another critical state concept. The more difficult task of developing anisotropic models for both initial and induced anisotropy has not been simply related to the critical state derivation approach (Dafalias, 1987). Many critical state models did not foresee the post peak softening in soil, which is caused by the presence of natural cementation and structure between soil grains (Sandhya Rani *et al.*, 2014a). Qualitative stress paths for a number of field situations would indicate that soil elements would be loaded and deformed differently depending on the geotechnical construction. The constitutive relation in soils is dependent on stress path followed and on rate of loading as indicated by the model tests conducted for different stress paths (Sandhya Rani *et al.*, 2014b). Accordingly, Dafalias (1987) extended the basic a methodology of critical state soil mechanics from isotropy to anisotropy. In this work emphasis was given to the triaxial formulation only. The yield surface expression was obtained by integrating the relevant differential equation in closed form in q - p space starting with a rate of plastic work dissipation equation that notably features an anisotropic variable.

The anisotropic variable evolution law was proposed, introducing two additional model constants which control the demise of anisotropy.

The General Elastoplasticity in Triaxial Space

The triaxial variables are defined by Equations (1) and (2).

$$p = \frac{\sigma_1 + 2\sigma_3}{3} \text{ and } q = \sigma_1 - \sigma_3 \quad (1)$$

$$\epsilon_v = \epsilon_1 + 2\epsilon_3 \text{ and } \epsilon_s = \frac{2}{3}(\epsilon_1 - \epsilon_3) \quad (2)$$

The strain rate decomposition will be from Equation (3).

$$\delta\epsilon_v = \delta\epsilon_v^e + \delta\epsilon_v^p \text{ and } \delta\epsilon_s = \delta\epsilon_s^e + \delta\epsilon_s^p \quad (3)$$

Where

σ_1 and σ_3 = Axial and Radial stress,
 p and q = Mean Principal stress and Deviator stress,
 ϵ_1 and ϵ_3 = Axial and Radial stress,
 ϵ_v and ϵ_s = Volumetric and Shear strain. The subscripts v and s denote volumetric and shear components, and the superscripts e and p denote elastic and plastic components

Elastic Rate Relations shall be Defined as Follows

It is assumed that any variations in the mean effective stress are accompanied by recoverable volume changes according to Equation (4)

$$\delta\epsilon_v^e = \kappa \frac{\delta p}{vp} \quad (4)$$

implying a linear relationship, between the logarithm of mean stress p and specific volume v in the compression plane for elastic unloading reloading of the soil (Figure 1). Where κ = Slope of over consolidation line. The critical state line on p - q plane is shown in Figure 2. It is assumed that any changes in deviator stress q are accompanied by recoverable shear strains according to Equation (5)

$$\delta\epsilon_s^e = \frac{\delta q}{3G} \quad (5)$$

with constant shear modulus G (Figure 3).

Features of the Existing Plasticity Model

The modified wheeler model (Rao, 2012) is considered for the present study. The model is a development of critical state models, with

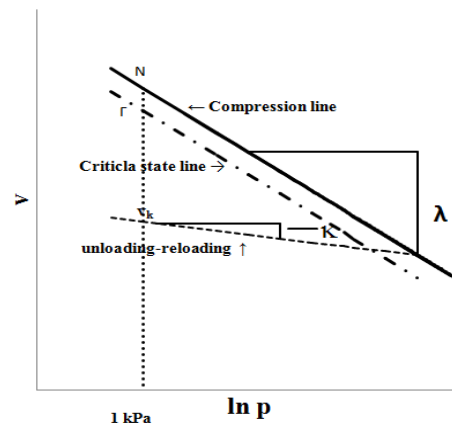


Figure 1. Loading and unloading-reloading line in compression plane (model parameters: λ , κ , Γ & N)

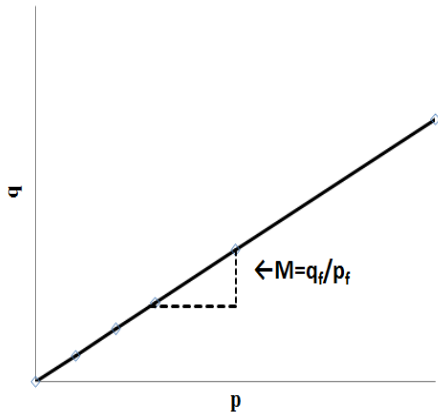


Figure 2. Critical state line on p-q plane

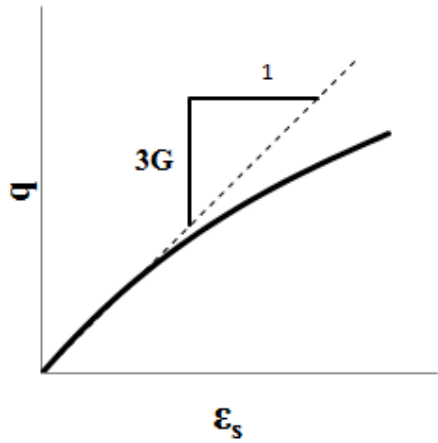


Figure 3. Shear Modulus, G

a rotational component of hardening used to represent the anisotropy of plastic behaviour and the intercept on the q axis in p-q space for the applicability of model for tropical soils. The model presented here is of the triaxial test for simplified stress space. The most important feature of the model for the partially saturated soils is that the critical state line will not pass through the origin and has an intercept μ on the p-q plane. This effect is considered in the yield curve along with anisotropy of plastic behaviour through the rotational component of hardening α . In tropical residual soils, where plastic deformations predominate, the model is suitable. The yield curve is a sheared ellipse with intercept μ on the q-axis and the slope of the critical state line in p-q space is defined by (Rao, 2012) using Equations (6) to (8).

$$f = (q - \alpha p)^2 - (M_*^2 - \alpha^2)(p_m - p)(p_m + p - 2p_x) = 0 \quad (6)$$

$$\frac{p_x}{p_m} = \frac{\Gamma - \ln p_m - v}{\lambda - \kappa} \quad (7)$$

$$\frac{p_m}{p_x} = X \quad (8)$$

Where aspect ratio M_* which is a function of X and the current size of the yield curve will be given by Equation (9)

$$M_* = \frac{(Mp_x + \mu)}{p_m - p_x} \quad (9)$$

M is the slope of the critical state line in q-p space μ is the intercept of the critical state line on the q-axis, p_x is stress where the critical state line and yield curve meet and p_m is the tip stress defining the size of the ellipse, where X varies from 2 to 2.5η (where η is the stress ratio and is equal to q/p) and the parameters α and p_m define the inclination and size of the yield curve respectively as shown in Figure 4. The parameter α is a measure of the degree of plastic anisotropy of the soil.

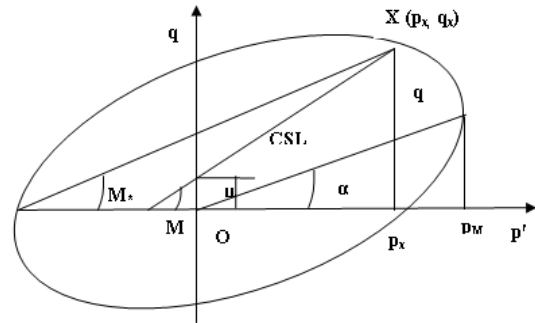


Figure 4. Yield curve of Modified Wheeler Model (Rao, 2012)

When the soil is yielding, the variation in the yield locus size p_0 is linked with the variations in the effective stresses p and $q = \eta p$, through the differential form of equation 6 as shown in Equation (10).

$$df = (q - \alpha p)dq + \left(M_*^2 p - \alpha q - \frac{M_*^2 p_m}{X} + \frac{\alpha^2 p_m}{X}\right)dp + \left(\alpha p_m^2 \left(1 - \frac{2}{X}\right) + \frac{2\alpha p p_m}{X} - pq\right)d\alpha - \left((M_*^2 - \alpha^2)\left(\frac{p}{X} + p_m - \frac{2p_m}{X}\right)\right)dp_m = 0 \quad (10)$$

The Model parameters are described in Table 1.

It is assumed that the soil obeys the normality condition. Accordingly, the plastic potentials are

Table 1. The Model Parameters

S. No.	Description	Symbol	Schematic Representation
1	Elastic Shear Modulus	G	Figure 3
2	Slope of Critical state line when it is projected onto a constant volume plane	M	
3	Specific volume of isotropically normally consolidated soil at $p = 1.0 \text{ kN/m}^2$	N	
4	Specific volume of soil at critical state with $p = 1.0 \text{ kN/m}^2$	Γ	Figures 1 and 2
5	Slope of Normal Consolidation line	λ	
6	Slope of over-consolidation line	κ	
7	Initial inclination of the yield curve	α	
8	Rotational hardening parameter, which defines the relative effectiveness of plastic shear and volumetric strains	β	Figure 4
9	Intercept on deviator stress axis on p-q plot	μ	

automatically given in the $p - q$ plane by the same family of curves (Equation (11)).

$$g = f = (q - \alpha p)^2 - (M_*^2 - \alpha^2)(p_o - p)(p_o + p - 2p_x) = 0 \quad (11)$$

Then the vector of plastic strain increments $\delta \epsilon_v^p : \delta \epsilon_s^p$ is in the direction of the outward normal to the yield locus. An associated flow rule is assumed for the sake of simplicity (Equation (12)).

Therefore,

$$\frac{d\epsilon_s^p}{d\epsilon_v^p} = \frac{\eta - \alpha}{(M^2 - \alpha^2)\left(1 - \frac{p_x}{p}\right) + \alpha(\alpha - \eta)} \quad (12)$$

The model's numerical implementation is significantly simpler when an associated flow rule is assumed than when a non-related flow rule is used. This presumption is supported by experimental data from Graham *et al.* (1983) and Korhonen and Lojander, (1987) for a number of soil types.

Two hardening rules are incorporated into the model. The first one, which is comparable to the Wheeler Model, describes variations in the size of yield curve (Equation (13)).

$$dp_m = \frac{vp_m}{\lambda - \kappa} d\epsilon_v^p \quad (13)$$

The second hardening rule predicts the change in yield curve inclination produced by plastic straining, representing the measure of anisotropy with plastic strains. It is assumed that plastic shear strain tries to drag the value of α towards a different instantaneous target value $\chi_d(\eta)$ that is dependent on the current value of η whereas plastic volumetric strain is simultaneously tries to drag the value of α towards an instantaneous target value $\chi_v(\eta)$ also dependent on η . Therefore, the rotational hardening law is shown by equation (14).

$$d\alpha = \mu'[(\chi_v(\eta) - \alpha)d\epsilon_v^p + \beta(\chi_d(\eta) - \alpha)d\epsilon_s^p] \quad (14)$$

The overall current target value of α will fall between $\chi_v(\eta)$ and $\chi_d(\eta)$. The absolute rate at which α approaches its current target value and the relative effectiveness of plastic shear strains and plastic volumetric strains in determining the present target value are controlled, respectively, by the constants β and μ .

The following expressions for $\chi_v(\eta)$ and $\chi_d(\eta)$ were proposed by Näättänen *et al.* (1999) based on Wheeler's initial yield curve.

$$\chi_v(\eta) = \frac{2}{3}\eta \quad (15)$$

$$\chi_d(\eta) = \frac{1}{3}\eta \quad (16)$$

η = Stress ratio ($=q/p$). Plastic volumetric strains attempt to align the yield curve approximately about the current stress point in practice, according to the expression for $\chi_v(\eta)$ in Equation (15) (Wheeler, 1997). According to Näättänen *et al.* (1999), the suggested value for

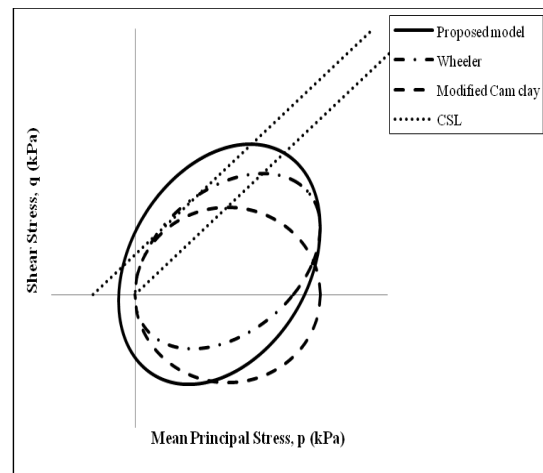


Figure 5. Schematic diagram showing modified Cam-clay, Wheeler model, and proposed model yield curves

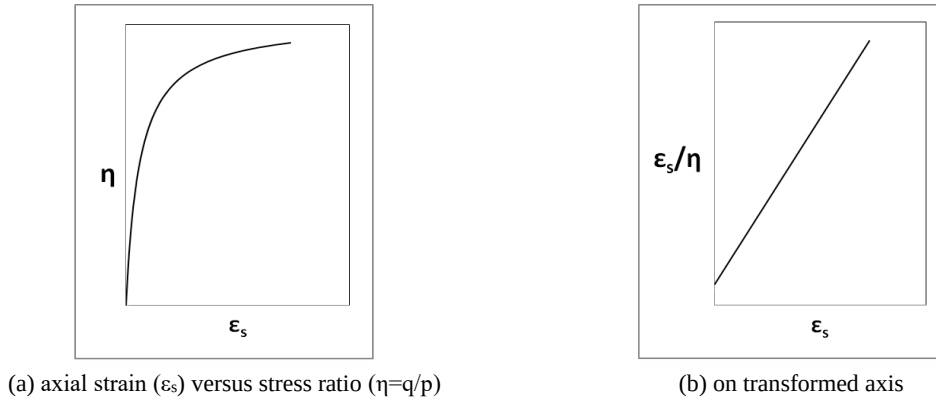


Figure 6. Typical response of natural soil under undrained triaxial compression

$\chi_d(\eta)$ in Equation (16) corresponds to a significant degree of anisotropy at critical states $\alpha = M/3$ at $\eta = M$.

The yield curve for the proposed model in comparison with modified Cam-clay and Wheelers model is shown in Figure 5.

Features of the Model Proposed

The proposed method is based on the Dafalias (1987) which was later extended by Wheeler *et al.* (1999) and Rao (2012). The rotational hardening parameter da of modified Wheeler's model is modified in the proposed model. The method allows for calibrating model parameters adopted in the model in addition to being an independent method to model the strain softening behaviour. This approach has the twin advantages of combining the conventional hyperbolic approach to the critical state concept and calibrating the model parameters adopted in the model for use in other stress paths.

Although it would be ideal to have a comprehensive model that could describe the behaviour of soil under any type of loading and was based on sound principles of continuum mechanics, most often such a generalized model may not be required (Nagendra Prasad *et al.*, 1999). It may be possible to analyse the problem using simpler models, depending on the particular field situation. Hyperbolic nonlinear elastic models developed (Duncan and Chang, 1970) are used in finite element applications even to this date owing to simplicity. However, Kodner's approach does not address the strain softening behaviour which is typical of natural geo-materials. A simple procedure for representing the nonlinear, stress dependent stress-strain behaviour of sensitive clay during undrained shear was proposed (Nagendra Prasad

et al., 1999). The hyperbolic response of stress ratio ($\eta = q/p$) shear strain has been used to describe the strain softening behaviour. Recently, Nagendra Prasad and Sulochana, (2013) has extended the hyperbolic response using the above approach, by incorporating the functional relationship between model parameters and yield stress in oedometer compression, in relation to tropical clayey soils.

An examination of data from experimental findings of tropical residual soils available in the literature indicates a typical relationship of η versus ϵ_s (Figure 6) is hyperbolic. Variation of shear strain with stress ratio (q/p) in terms of hyperbolic relation takes the form as

$$\frac{\epsilon_s}{\eta} = a\epsilon_s + b \quad (17)$$

Differentiating Equation (17) to get Equation (18)

$$d\eta = \frac{d\epsilon_s(1-a\eta)}{b+a\epsilon_s} \quad (18)$$

a and b are Hyperbolic constants.

The rotational hardening parameter da is replaced by $d\eta$ equation computed from the hyperbolic approach as this expression represents the proportionate evolution of de-structuring concomitant with the current state of stress.

The model's description is now completed. The elastic stress: strain response in the matrix equation by combining the Equations (4) and (5) can be summarized as

$$\begin{bmatrix} d\epsilon_v^e \\ d\epsilon_s^e \end{bmatrix} = \begin{bmatrix} \frac{\kappa}{vp} & 0 \\ 0 & \frac{1}{3G} \end{bmatrix} \begin{bmatrix} dp \\ dq \end{bmatrix} \quad (19)$$

Table 2. Method to deduce the model parameters

S. No.	Description	Equation	Reference
Data Required			
1	Poisson's ratio	U	
2	Angle of Internal Friction	Φ	
3	Specific Gravity	G	
4	Liquid Limit	LL	
Functional Relations			
5	Initial Void Ratio, e_0	$e_o = \frac{G * LL}{100}$	
6	Specific volume v_o	$v_o = 1 + e_o$	
7	C_c	$C_c = 0.009 * (LL - 10)$	Wood, 1991
Modified Cam-clay Parameters			
8	G	$G = \frac{3(1 - 2\theta)vp}{2\kappa(1 + \theta)}$	
9	M	$M = \frac{6\sin\phi}{3 - \sin\phi}$	
10	N	$N = v_o - \lambda \ln(5)$	
11	Γ	$\Gamma = N - \lambda + \kappa$	
12	λ	$\lambda = \frac{C_c}{2.303}$	
Wheeler Model Parameters and Functional Relations			
13	Earth Pressure coefficient at rest, K_0	$K_o = 1 - \sin\phi$	
14	Stress ratio, η_0 for 1-D compression	$\eta_{K_o} = \frac{3(1 - K_o)}{1 + 2K_o}$	
15	α	$\alpha = \frac{\eta_{K_o}^2 + 3\eta_{K_o} - M^2}{3}$	
16	β	$\beta = \frac{3(M^2 - \eta_{K_o}^2 - \frac{3\eta_{K_o}}{4})}{2(\eta_{K_o}^2 - M^2 + 2\eta_{K_o})}$	Wheeler <i>et al.</i> (1999)
17	μ	Experimental determination is difficult. The ideal action is to choose a default value based on model simulation.	

$-d\epsilon_v^e$ and $d\epsilon_s^e$ are elastic volumetric and shear strain increments respectively. Substituting Equations (13) and (18) in Equation (10) and on simplification, the plastic stress: strain response can be summarized in the matrix form as:

$$\begin{bmatrix} d\epsilon_v^p \\ d\epsilon_s^p \end{bmatrix} = \begin{bmatrix} p_{11} & p_{12} \\ p_{21} & p_{22} \end{bmatrix} \begin{bmatrix} dp \\ dq \end{bmatrix} \quad (20)$$

$d\epsilon_v^p$ and $d\epsilon_s^p$ are plastic volumetric and shear strain increments respectively. Equations (19) and (20) are absolutely general and are relevant to all effective stress paths in the p:q plane. The model parameters involve α , β , and μ in addition to the modified Cam-clay model. The parameters can be determined referring to the expression presented in Table 2.

p_{11} , p_{12} , p_{21} , and p_{22} are defined in Equations (21) to (24) respectively.

$$p_{11} = \frac{M_*^2 p - \alpha q - \frac{M_*^2 p m}{X} + \frac{\alpha^2 p m}{X}}{\left(\frac{v p m}{\lambda - \kappa}\right)(M_*^2 - \alpha^2) \left(\frac{p}{X} + p_m - \frac{2 p m}{X}\right) - \left(\frac{1 - a \eta}{b + a \epsilon_s}\right)(\alpha p_m^2 \left(1 - \frac{2}{X}\right) + \frac{2 \alpha p p m}{X} - p q) \left(\frac{\eta - \alpha}{(M^2 - \alpha^2) \left(1 - \frac{p X}{p}\right) + \alpha(\alpha - \eta)}\right)} \quad (21)$$

$$p_{12} = \frac{(q - \alpha p) + \left(\frac{1}{3G}\right) \left(\frac{1 - a \eta}{b + a \epsilon_s}\right) (\alpha p_m^2 \left(1 - \frac{2}{X}\right) + \frac{2 \alpha p p m}{X} - p q)}{\left(\frac{v p m}{\lambda - \kappa}\right)(M_*^2 - \alpha^2) \left(\frac{p}{X} + p_m - \frac{2 p m}{X}\right) - \left(\frac{1 - a \eta}{b + a \epsilon_s}\right)(\alpha p_m^2 \left(1 - \frac{2}{X}\right) + \frac{2 \alpha p p m}{X} - p q) \left(\frac{\eta - \alpha}{(M^2 - \alpha^2) \left(1 - \frac{p X}{p}\right) + \alpha(\alpha - \eta)}\right)} \quad (22)$$

$$p_{21} = \frac{M_*^2 p - \alpha q - \frac{M_*^2 p m}{X} + \frac{\alpha^2 p m}{X}}{\left(\frac{v p m}{\lambda - \kappa}\right)(M_*^2 - \alpha^2) \left(\frac{p}{X} + p_m - \frac{2 p m}{X}\right) \left(\frac{(M^2 - \alpha^2) \left(1 - \frac{p X}{p}\right) + \alpha(\alpha - \eta)}{\eta - \alpha}\right) - \left(\frac{1 - a \eta}{b + a \epsilon_s}\right)(\alpha p_m^2 \left(1 - \frac{2}{X}\right) + \frac{2 \alpha p p m}{X} - p q)} \quad (23)$$

$$p_{22} = \frac{(q - \alpha p) + \left(\frac{1}{3G}\right) \left(\frac{1 - a \eta}{b + a \epsilon_s}\right) (\alpha p_m^2 \left(1 - \frac{2}{X}\right) + \frac{2 \alpha p p m}{X} - p q)}{\left(\frac{v p m}{\lambda - \kappa}\right)(M_*^2 - \alpha^2) \left(\frac{p}{X} + p_m - \frac{2 p m}{X}\right) \left(\frac{(M^2 - \alpha^2) \left(1 - \frac{p X}{p}\right) + \alpha(\alpha - \eta)}{\eta - \alpha}\right) - \left(\frac{1 - a \eta}{b + a \epsilon_s}\right)(\alpha p_m^2 \left(1 - \frac{2}{X}\right) + \frac{2 \alpha p p m}{X} - p q)} \quad (24)$$

Critical Appraisal on Determination of Soil Parameters

If α can be determined, and then to calculate the initial size of the yield curve p_0 , only one point on the yield curve is required. In a triaxial apparatus, this yield point would ideally be determined by isotropic or K_0 consolidation. Alternatively, it would be possible in an oedometer, but it requires measurement of radial stress to fully define the stress state (Wheeler, 1997). Because there are uncertainties regarding the depositional history and because there is a chance that the yield stress will rise above the maximum pressure previously applied due to the effects of ageing or inter-particle bonding, estimation of a point on the yield curve without performing any laboratory tests would rarely be satisfactory (Burland, 1990).

In addition, it would be difficult to come up with a simple direct method for experimentally measuring the value of μ for a particular soil. The only option would seem to be to run model simulations with various values of μ , compare the results to observed behaviour, and then choose the appropriate value. The necessary experimental test would include a significant yield curve rotation. The degree of rotation of the yield curve, which is determined experimentally by unloading and then reloading along a different stress route, as well as the observed pattern of straining, can then be used to compare observed and predicted behaviour.

Accordingly, there is a need to advantageously make use of the rotational hardening model proposed with the associated flow rule for simplicity, by appropriately regulating the parameters with the observed response. The following method would be one such attempt to propose a method which is

based on observed response and the approach which is widely known to practicing engineers. Most natural clayey deposits, as well as lime/cement treated clayey soils, exhibit strain softening behaviour, which can affect, for example, the stability of an embankment and the bearing capacity of a foundation on this kind of ground. Failure of soil under a foundation normally occurs progressively, and in order to simulate the progressive failure phenomenon, strain-softening material behaviour should be considered. Most existing elasto-plastic soil models for clayey soils have a limited capacity to model strain-softening behaviour. For example, the widely used Modified Cam-clay (MCC) model can only simulate softening for a soil element in a heavily over consolidated state with a relatively high value of the ratio κ/λ .

Validation of the Proposed Approach

The approach proposed is applied to soils available in literature whose stress-strain response is characterized by softening behaviour. The soil data from the literature for lateritic and saprolitic soils (Futai *et al.*, 2004) and five regional soils in and around Tirupati region, Andhra Pradesh (Sulochana *et al.*, 2013) is considered. Table 3 shows the basic properties of above said soils.

Figures 7 to 13 show the experimental data represented in conventional stress-strain-pore pressure response. The presented data demonstrates the occurrence of strain softening in relation to elevated pore water pressures. It is worth noting that the pore water pressure remains positive even in the softening region, suggesting that the behaviour differs from that of over-consolidated soils. The phenomenon of strain softening in relation to

Table 3. Soil Data from Literature

Source	Futai <i>et al.</i> (2004)		Sulochana <i>et al.</i> (2013)				
Typical Name of Soil	Lateritic soil	Saprolitic soil	Vinayaka Nagar soil	Gayatri Nagar soil	Renigunta soil	Munireddy Nagar soil	Tiruchanur soil
Location	Ouro Preto, Brazil		Tirupati, India				
Depth of Sampling (m)	1	5	2.7	3.5	2.5	1.8	2.7
% Gravel			3	1	6.6	0.5	7
% Sand	45	38	46	43	63.4	57.5	48
% Silt	10	52	51	56	30	42	45
% Clay	45	10					
Specific Gravity, G_s	2.63	2.68	2.67	2.67	2.66	2.66	2.66
Natural water content (%)	28	46	16.7	17.73	19.86	16.25	14.92
Liquid Limit (%)	58	42	42	33	92	27	55
Plastic Limit (%)	28	20	30	22	45	21	35
Cohesion (KN/m ²)	7	15					
Angle of internal friction	28.1°	31°					
MDD (KN/m ³)	15.2	17.5					
IS Classification	CH	CI	CI	CL	SC	SC	SC

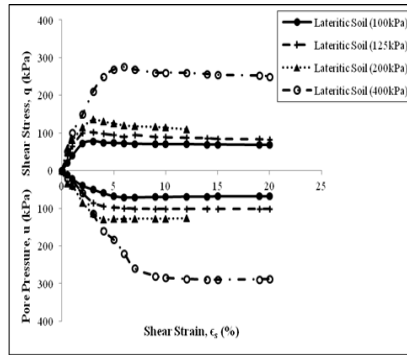


Figure 7. Stress-Strain-Pore pressure response of Lateritic soil

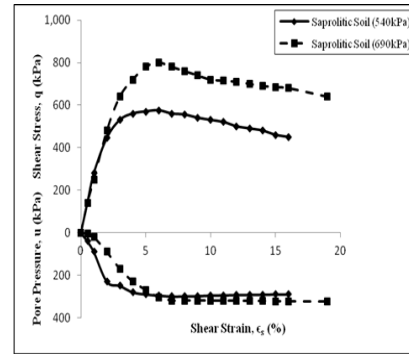


Figure 8. Stress-Strain-Pore pressure response of Saprolitic soil

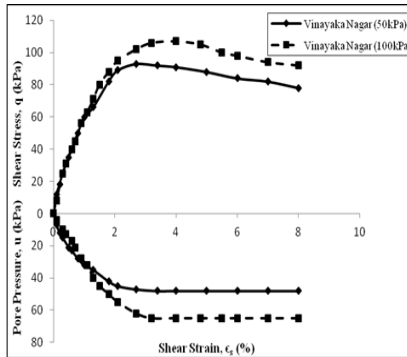


Figure 9. Stress-Strain-Pore pressure response of Vinayaka Nagar soil

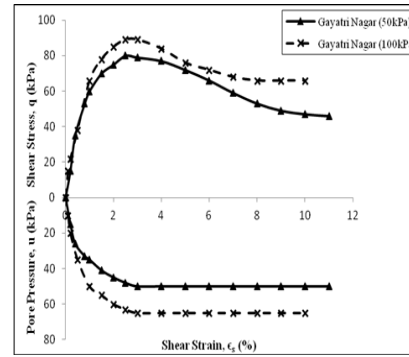


Figure 10. Stress-Strain-Pore pressure response of Gayatri Nagar soil

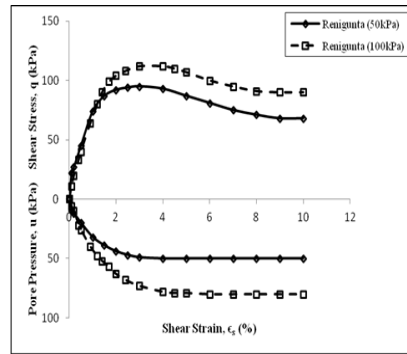


Figure 11. Stress-Strain-Pore pressure response of Renigunta soil

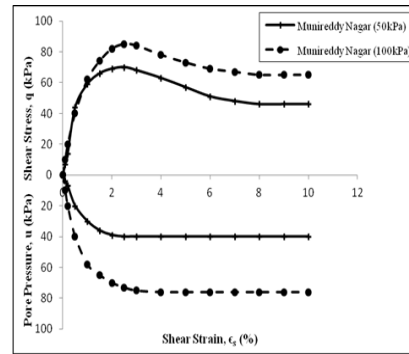


Figure 12. Stress-Strain-Pore pressure response of Munireddy Nagar soil

positive pore pressures is an intriguing characteristic observed in tropical residual soils, as noted by Sulochana (2013). Figures 14 to 20 illustrate the effective stress paths of the samples. The stress paths observed in this study demonstrate a decrease in mean effective stress during the strain softening process.

Conclusions

Based on detailed analysis of modified Wheeler's rotational hardening model in comparison to the hyperbolic approach proposed the following conclusions were made.

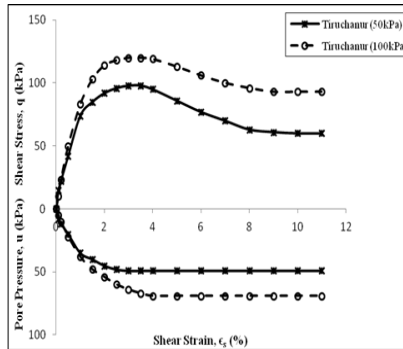


Figure 13. Stress-Strain-Pore pressure response of Tiruchanur soil

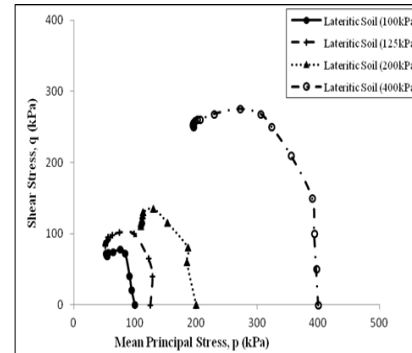


Figure 14. Effective stress paths of Lateritic soil

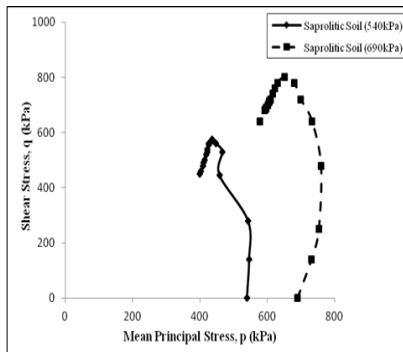


Figure 15. Effective stress paths of Saprolitic soil

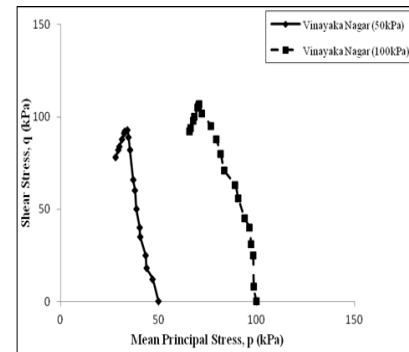


Figure 16. Effective stress paths of Vinayaka Nagar soil

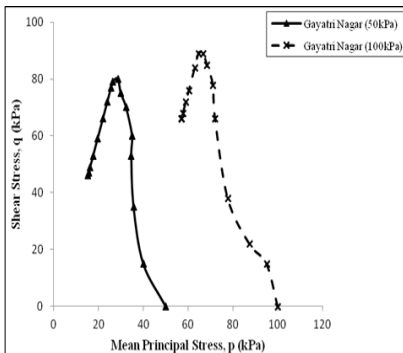


Figure 17. Effective stress paths of Gayatri Nagar soil

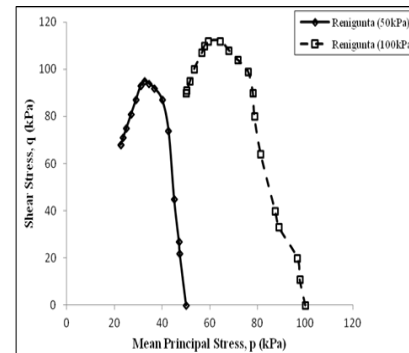


Figure 18. Effective stress paths of Renigunta soil

- An attempt has been made to the hardening law, proposed by Wheeler in order to incorporate different soil states in a unified manner.
- The model proposed with modified hardening law has wide applicability encompassing unsaturated cemented state to saturated uncemented state.
- The de-structuring of soil during shear is modeled based on phenomenological approach in order to describe the observed behavior mathematically so as to minimize empiricism associated with the model parameters.
- The evaluation of the stress ratio observed with the shear deformation has been mathematically

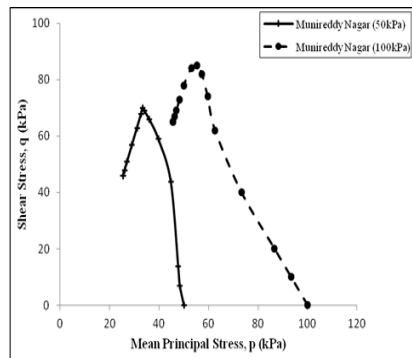


Figure 19. Effective stress paths of Munireddy Nagar soil

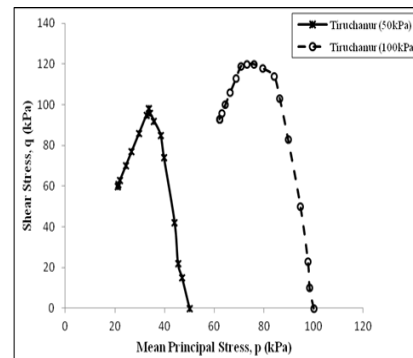


Figure 20. Effective stress paths of Tiruchanur soil

incorporated in the proposed modification of hardening rule.

- The hardening law proposed is based on phenomenological approach wherein strain softening behaviour can be modelled by hyperbolic approach involving stress-ratio and shear strain, which is the unique feature.
- The model proposed has two hyperbolic parameters which are found to be proportional to confining stress. These parameters can be obtained from the simple triaxial test results, which experimental results are used for computing other model parameters
- The applicability of the model to wide spectrum of soils characterized by strain softening to strain hardening and therefore testifies its ability to capture the salient features of observed response for different states of soils.
- The proposed work is an attempt to predict the behaviour based on observed response of soils for undrained condition. In other words, the phenomenological hardening law is specific to undrained condition which can be tested to other stress paths if more elaborate experimental data is available for soils at different states.

References

- Burland, J.B. (1990). On the compressibility and shear strength of natural clays. *Geotechnique*, 40(3):329-378. <https://doi.org/10.1680/geot.1990.40.3.329>
- Dafalias, Y.F. (1987). An isotropic critical state clay plasticity model. *Constitutive Laws For Engineering Materials: Theory and Applications*. Elsevier Science, p. 513-521.
- Duncan, J.M. and Chang, C.Y. (1970). Nonlinear analysis of stress and strain in soils. *Journal of the Soil Mechanics and Foundations Division, ASCE*, 96(5):1629-1653. <https://doi.org/10.1061/JSFEAQ.0001458>
- Futai, M.M., Almeida, M.S.S., and Lacerda, W.A. (2004). Yield, strength and critical state behaviour of a tropical saturated soil. *Journal of Geotechnical and Geoenvironmental Engineering, ASCE*, 130(11):1169-1179. [https://doi.org/10.1061/\(ASCE\)1090-0241\(2004\)130:11\(1169\)](https://doi.org/10.1061/(ASCE)1090-0241(2004)130:11(1169))
- Graham, J., Noonan, M.L., and Lew, K.V. (1983). Yield states and stress-strain relations in natural plastic clay. *Canadian Geotechnical Journal*, 20(3):502-516. <https://doi.org/10.1139/t83-058>
- Korhonen, K.H. and Lojander, M. (1987). Yielding of permo clay. *Proceeding of the 2nd International Conference on Constitutive Laws for Engineering Materials*, Tucson, Arizona, 2:1249-1255.
- Näätänen, A., Lojander, M., Wheeler, S.J., and Karstunen, M. (1999). Experimental investigation of an anisotropic hardening model for soft clays. In: Jamiolkowski, M., Lancellotta, R., and Lo Presti, D. (eds.), *Pre-Failure deformation characteristics of geomaterials*, 1:541-548.
- Nagendra Prasad, K. and Sulochana, N. (2013). Hyperbolic constitutive model for tropical residual soils. *International Journal of Civil Engineering and Technology (IJCIET)*, 4(3):121-133.
- Nagendra Prasad, K., Srinivasa Murthy, B.R., Sitharam, T.G., and Vatsala, A. (1999). Hyperbolic stress-strain and pore pressure response of sensitive clays. *Indian Geotechnical Journal*, 29(3):221-241.
- Rao, D.V. (2012). *A Generalized Constitutive Model for Prediction of Behaviour of Soils*, [Ph.D. Thesis]. Sri Venkateswara University, Tirupati.
- Roscoe, K.H. and Burland, J.B. (1968). On the generalized stress-strain behaviour of wet clay. *Engineering plasticity*, In: Heyman, J. and Keckie, F.A. (eds.), *Cambridge University Press*. New York, p. 535-609.
- Sandhya Rani, R., Nagendra Prasad, K., and Sai Krishna, T. (2014a). Applicability of mohr-coulomb & drucker-prager models for assessment of undrained shear behaviour of clayey soils. *International Journal of Civil Engineering and Technology (IJCIET)*, 5(10):104-123.
- Sandhya Rani, R., Nagendra Prasad, K., and Vidhushimani, M. (2014b). Relative influences of stress paths with different initial soil states on stress-strain response using model tests. *International Journal of Civil Engineering and Technology (IJCIET)*, 5(8):20-31.
- Schofield, A.N. and Wroth, C.D. (1968). *Critical state soil mechanics*. McGraw-Hill, New York, 229p.
- Sulochana, N. (2013). *Analysis, Assessment and Constitutive Modelling of Soils*, [Ph.D. Thesis]. Sri Venkateswara University, Tirupati.

- Sulochana, N., Nagendra Prasad, K., and Chandra, B. (2013). Phenomenological approach to analyse the behaviour of tropical residual clayey soils. *International Journal of Civil Engineering (IJCE)*, 5(2):81-96.
- Wheeler, S.J. (1997). A rotational hardening elasto-plastic model for clays. *Proceedings of the 14th International Conference on Soil Mechanics and Foundation Engineering*, Hamburg, A.A. Balkema, Rotterdam, 1:431-434.
- Wheeler, S.J., Nääätänen, A., and Karstunen, M. (1999). Anisotropic hardening model for normally consolidated soft clay. In: *Proceedings of the 7th International Symposium on Numerical Models in Geomechanics (NUMOG)*, Graz, Austria, A.A. Balkema, Rotterdam, p. 33-40. <https://doi.org/10.1201/9781003078548-7>
- Wood, D.M. (1991). *Soil behaviour and critical state soil mechanics*. Cambridge University Press, New York. <https://doi.org/10.1017/CBO9781139878272>