STRAIN THRESHOLDS IN SOIL DYNAMICS

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ABSTRACT:
Based on conceptual frameworks of soil behavior and published experimental data related to stress and strain thresholds, alternative cyclic strain regime divisions and the cyclic strain thresholds that represent the change in soil behavior are proposed. The criteria taken into account for the development of this approach includes stress-strain behavior, stiffness degradation, pore pressure generation, post-cyclic strength and microscale processes. Fundamentally for clayey soils, four types of cyclic strain thresholds are identified: linear γ_{tl}, volumetric γ_{tv}, degradation γ_{td}, and flow γ_{tf}, with five strain regimes delimited by these thresholds.

KEYWORDS: Clayey soils, strain threshold, stress threshold, stiffness degradation, cyclic loading

1. INTRODUCTION

To evaluate the response of soil to different dynamic loadings, considering not only earthquake loading but also to the stability evaluation of marine structures subjected to the cyclic action of waves or pavements under traffic loading conditions, it is necessary to determine the modulus and damping properties of the various soil layers.
The deformation characteristics of soil are highly nonlinear and this is manifested in the shear modulus and damping ratio which vary significantly with the amplitude of shear strain under cyclic loading. With increasing strain amplitude, soil stiffness decays non-linearly, giving a typical S-shaped stiffness reduction curve in a semi-logarithmic space.
The amplitude of cyclic shear strain at which a marked reduction in stiffness begins to occur has been known to depend upon several factors such as soil composition, initial volumetric relations between soil phases, stress history, initial stress state, time, rate of loading and number of cycles. From these facts, it has been shown that in clayey soils, a wide range for the shear strain exists to initiate the modulus decrease.
It is clear that clayey soils with higher plasticity tend to have a more linear behavior at small strains and their normalized modulus reduction curve gradually moves to the right, showing a slower rate of reduction with increasing shear strain.
There have been slight differences in the implications and interpretations of the cyclic threshold shear strain among different researchers. Several strain regimes can be identified from the experimental data obtained from the literature. However, it is the purpose of this paper to summarize available data on the threshold strain for clayey soils under cyclic loading conditions and from this information, to propose alternative strain regime divisions with their corresponding thresholds that define a change in the cyclic soil behavior.

2. PREVIOUS STUDIES

Since the first comprehensive reports on dynamic soil properties (Seed and Idriss, 1970; Hardin and Drnevich, 1972a and 1972b), much progress has been made in improving dynamic testing apparatus so that the dynamic properties of a specific soil can be measured over a wide strain range.
Previous studies can be divided into two groups according to strain level measured and the testing apparatus used. The studies of the first group are related to very small strain properties, tested with the resonant column apparatus, where major concerns are centered on the determination of G_{max} and very small shear modulus. On the other hand, the studies of the second group are mainly concerned with the effects of degradation on the shear modulus and damping ratio measured with various types of cyclic loading test apparatus and cover a larger strain level.
A compilation of published stress and strain thresholds for soils is presented in Table 1, divided in agreement

with the strain regimes proposed in this revision. General trends common to clayey soils and other important investigations are summarized next:

Larew and Leonards (1962) from the study of subgrade soils under repeated loading suggested the existence of a critical level of repeated deviator stress, defined as a threshold stress state at which the slope of the deformation versus load cycles curve remains constant after the first few load applications.

Seed and Chan (1966) described a procedure to determine the combinations of sustained and pulsating stresses that cause failure. From the evaluation of parameters like nature of loading conditions (one-directional or two-directional), soil type, frequency, number of stress cycles and form of stress pulse, they presented their results in the form of iso-N_f curves in a pulsating versus sustained shear stress space. In subsequent years many researchers used the same analysis criteria, just varying the number of cycles of loading, and failure criterion (Houston and Herrmann 1980, Malek et al. 1989, McCarron 1995).

Hardin and Black (1968) concluded on the basis of several previous studies that vibrations shear modulus is independent of shear-strain amplitude smaller than about 1 x 10^{-4}.

Sangrey (1969) from an undisturbed New York clay subjected to repeated cycles of loading and unloading at low frequencies in triaxial tests, shows that for any particular consolidation history and in terms of effective stresses, a critical level of repeated stress exists. Below this, the final soil behavior is essentially elastic and a state of nonfailure equilibrium was reached; above this critical level, non-recoverable deformation and effective stress failure occurred.

Silver and Seed (1971) investigated in the laboratory the cyclic settlement of dry Crystal Silica sand in cyclic direct simple shear device. They show that a significant cyclic settlement occurs only if γ_c is larger than 0.05%. The authors consider that cyclic shear strain may be a fundamental parameter in determining the volume change behavior of cohesionless soil under dynamic loading conditions.

Anderson and Richart (1976) from a series of tests on clays using a torsional device, defined a threshold level for strain effect between 0.001% and 0.01%. If this level is exceeded, a progressive decrease in G_{max} is produced.

Stoll and Kald (1977) studied the behavior of granular soils and suggested that there is a cyclic shear strain level below which permanent volume changes become negligibly small. From stress controlled conditions they derived this strain level between $5x10^{-3}$% and $6x10^{-3}$%.

Koutsoftas (1978) measured the excess pore water pressure induced by cyclic loading using the dynamic triaxial test device and showed that the excess pore water pressure increased with increase of the axial strain amplitude. The undrained shear strength after cyclic loading decreases as the double amplitude strain increases. There is a consistent decrease in undrained secant and tangent modulus as the cyclic strain increases, Failure strains also increase with increasing cyclic strain level. The excess pore water pressure induced by cyclic loading cause a reduction in effective stress.

Idriss et al. (1978) developed a stress-strain model for soft clays subjected to one-dimensional undrained shear cyclic loading using triaxial data on San Francisco Bay mud. The stress-strain model uses a degradation index, δ, which can be related to the number of cycles by $\delta = N^{-t}$, in which t is a degradation parameter. The ordinates of the backbone curve decrease during cyclic loading. The degradation index, δ, is a measure of an irreversible degradation process occurring in the structure of the soil; therefore, δ stays constant or decreases but does not increase during cyclic loading. The backbone curve and the modulus reduction curve provide the same information, and once either is specified, the other curve is readily derived. The proposed degradation model is based on total stresses and the changes in pore-water pressure were not measured during the undrained cyclic tests.

Stokoe and Lodde (1978) investigated in the laboratory with the resonant column method the shear modulus and material damping of undisturbed samples of San Francisco Bay mud. They found that below a threshold shearing strain of about 0.001%, shear modulus and material damping of Bay mud were essentially independent of shearing strain amplitude. Above the threshold shearing strain, modulus decreased and damping increased as the strain increased. The threshold degradation strain amplitude was on the order of 0.01% for 1000 cycles of straining.

Matsui, Ohara and Ito (1980) reported the results of undrained triaxial cyclic loading tests on saturated clays, both normally and overconsolidated. Compression and extension stresses were applied alternately to a soil specimen under constant mean total principal stress. The effect of cyclic stress-strain history on shear characteristics of saturated clays has been considered with particular reference to excess pore pressures that were developed. For a given number of cycles, higher excess pore pressure and axial strains are generated at lower frequencies. The effect of cyclic shear stress level on the excess pore pressure corresponded to the development of deformations. A lower boundary value (τ_d/τ_f = 0.2) exists on the cyclic shear stress level. Saturated clays of greater OCR have more resistance against subsequent excess pore pressure generation.

Kim and Novak (1981) reported the dynamic behavior of soil under laboratory conditions using a resonant column apparatus and provide information on dynamic soil properties such as shear modulus and damping ratio that can be used in analysis and design. They found that the normalized modulus reduction curve of cohesive soils decreases as shear strain amplitude increases, once strain amplitudes exceed a certain threshold level. The G/G_{max} curve begins to decrease significantly once the shearing strains exceed about 0.01%.

Kokusho et al. (1982) carried out a systematic study conducted for dynamic properties of natural soft clay employing an improved cyclic triaxial apparatus. The shear modulus at 1 x 10^{-5} was chosen as the small strain modulus or elastic shear modulus. The results of the tests showed an insignificant effect of the confining stress and the consolidation histories. G_{max} increases more with time in high-plasticity clays than in low-plasticity clays or cohesionless soils with zero plasticity.

Ohara and Matsuda (1988) carried out two-way strain controlled cyclic simple shear tests under undrained conditions for normally consolidated and overconsolidated Kaolinite clay. The excess pore pressure induced by the cyclic shear and the settlement by dissipation of it were investigated. It was concluded that the excess pore water pressure reduces with increasing OCR and that the minimum shear strain amplitude $(\gamma_{dyn})_{min}$, which produces the excess pore water pressure, increases with OCR. The values of $(\gamma_{dyn})_{min}$ are 0.05%, 0.12%, 0.22% and 0.29% for OCR = 1, 2, 4 and 6 respectively.

Diaz-Rodríguez, J.A. (1989) described a series of cyclic triaxial tests on undisturbed soil samples of Mexico City soil. The soil is a silty clay of a very high water content and soft consistency. The material exhibits elastic behavior in spite of its very high water content. Based on tests results, a procedure to determine a stress threshold is proposed from the reduction of post-cyclic strength after 100 cycles of loading. Below the stress threshold, repeated loading has a negligible effect on the post-cyclic undrained shear strength. Over this, a remarkable reduction of shear strength is observed.

Ansal and Erken (1989) conducting a series of cyclic simple shear under stress controlled conditions on kaolinite samples, defined a cyclic yield stress ratio which appears to vary linearly on a semi logarithmic scale with the number of cycles. Above this stress ratio, large deformation can take place. Additionally they indicated a critical stress ratio level below which no pore water pressure will develop.

Lefebvre, LeBoeuf and Demers (1989) performed an experimental investigation to study the stability threshold under cyclic loading and post-cyclic static strength of sensitive clay from the Hudson Bay region. They express the cyclic undrained resistance as a stability threshold, defined as the stress level below which the soil will not suffer failure regardless of the number of applied cycles. The value of the stability threshold expresses the degradation of the undrained strength due to the accumulation of pore pressure in the cycling. They point out the problems associated with the determination of the reference static strength and also the effect of the strain rate difference between static and cyclic tests.

Vucetic and Dobry (1991) studied the influence of the plasticity index (PI) on the cyclic stress-strain parameters of saturated soils for site-response evaluation and microzonation. It is shown than PI is the main factor controlling G/G_{max} and ζ for a wide variety of soils. It is concluded that soils with higher plasticity tend to have a more linear cyclic stress-strain response and to degrade less at larger strains than soils with a lower PI.

Jardine (1992) proposed a general framework of soil behavior consisting in three characteristic zones of stress-strain response that are surrounded by two kinematic Sub-Yield surfaces (Y_1 and Y_2) located within a Bounding Surface (Y_3). The first surface forms the boundary of linear elastic behavior (Zone I), the second surface is the limit of recoverable, but non-linear behavior (Zone II). Zone III is defined as the area of irrecoverable strains, which become more important approaching the Bounding Surface.

Vucetic (1994) based on a synthesis of published laboratory data, examined two types of cyclic thresholds shear strain and their approximate magnitudes identified for different types of soils. They are the linear cyclic threshold shear strain, γ_{tl}, and the volumetric cyclic threshold shear strain, γ_{tv}, with $\gamma_{tv} > \gamma_{tl}$. For cyclic strains below γ_{tl}, soil behaves essentially as a linear elastic material. Between γ_{tv} and γ_{tl} soil behaves markedly nonlinear but remains largely elastic because permanent changes of its microstructure still do not occur or are negligible. Above γ_{tv}, soil becomes increasingly nonlinear and inelastic, with significant permanent microstructural changes taking place under cyclic loading.

Table 1. Stress and Strain Thresholds

Reference	Soil	IP	OCR	Proposed thresholds between strain regimes			
				Very Small Strains / Small Strains	Small Strains / Medium Strains	Medium Strains / Large Strains	Large Strains / Residual Strains
				γ_{tl}	γ_{tv}	γ_{td}	γ_{tf}
Larew and Leonard (1962)	Subgrade soils: sand/clay	--	--			Threshold stress state (60 000 to 80 000 cycles)	
Seed and Chan (1966)	Silty clay	--	--			Sustained plus pulsating stresses that cause failure (1, 10 and 100 cycles)	
Hardin and Black (1968)	Kaolinite	21	1	Vibration shear strain less than 10^{-4}			
Sangrey (1969)	Newfield clay	10	1-4			Critical level of repeated stress (effective stresses)	
Silver and Seed (1971)	Crystal silica	--	--		Volume change (γ=0.05%)		
Anderson and Richart (1976)	Clayey soils	19-64	--	Threshold level for strain effect (from 0.001% to 0.01%)			
Stoll and Kald (1977)	Nonplastic silty soils	0	--		Threshold of dilatation (from $5x10^{-3}$% to $6x10^{-3}$%)		
Koutsoftas (1978)	Inorganic marine clays	18-40	1-4			Cyclic strain to decrease the post-cyclic shear strength	
Idriss et al. (1978)	San Francisco Bay mud	49	NC-OC	Degradation index (δ = 1)	Degradation index (δ < 1)		
Stokoe and Lodde (1978)	San Francisco Bay mud	49	--	Threshold shearing strain (γ=0.001%)	Threshold degradation strain (γ=0.01% for 1000 cycles)		
Houston and Herrmann (1980)	Marine soils	0-70	--			Critical level of repeated loading. CLRL (300 000 cycles)	
Matsui et al. (1980)	Senry clay	55	1-4		Cyclic stress and maximum strain to generate positive pore pressure		
Kim and Novak (1981)	Clayey soils	12-30	1-6	Threshold shear strain (γ=0.01%)			
Kokusho et al. (1982)	Teganuma clay	40-100	1-10	Small strain modulus (γ=0.001%)			
Ohara and Matsuda (1988)	Kaolinite	25	1-6		Minimum shear strain to produce pore pressure/settlement		
Malek et al. (1989)	Boston Blue clay	21	1			Threshold shear stress level (effective stresses)	
Díaz-Rodríguez (1989)	Mexico City Soil	73-342	OC			Cyclic stress critical value (100 cycles)	
Ansal and Erken (1989)	Kaolinite	40	1		Critical stress ratio		
Lefebvre et al. (1989)	Hudson Bay clay	12	NC-OC			Stability threshold	
Jardine (1992)	Magnus till, London clay	18, 45	1-17	Y1 surface	Y2 surface	Y3 surface	
Vucetic (1994)	Different types of soils	0-55	1-10	Threshold shear strain	Volumetric cyclic threshold		
McCarron (1995)	Beafourt Sea clay	24	3, 10			CLRL ($3x10^4$ to $1x10^6$ cycles)	
Ishihara (1996)	Different types of soils	--	--		Degradation threshold strain		
Santamarina (2001)	Different types of soils	--	--	Linear threshold strain	Degradation threshold strain		Threshold strain for residual strength
Hsu and Vucetic (2004)	Different types of soils	0-33	--		Volumetric threshold shear strain for cyclic settlement		
Hsu and Vucetic (2006)	Clayey soils	14-30	--		Threshold shear strain for cyclic pore-water pressure		
Okur and Ansal (2007)	Turkey soils	9-40	NC-OC	Elastic threshold			Flow threshold

Ishihara (1996) derived from previous studies (Kokusho et al. 1982 and Vucetic 1994) proposed an approach to the determination of the threshold strain for cyclic degradation that is equivalent to modulus reduction in the range between 0.6 and 0.85.

Hsu and Vucetic (2004, 2006) review the threshold shear strain concepts, define the threshold shear strain for cyclic pore-water pressure, γ_t, as a fundamental property of fully saturated soils and the volumetric cyclic threshold shear strain for cyclic settlement, γ_{tv}, for dry or partially saturated sands and partially saturated clays.

Okur and Ansal (2007) based on stress controlled cyclic triaxial tests on soil samples from various sites in Turkey, proposed a methodology to estimate the modulus reduction and the thresholds between non-linear elastic, elasto-plastic and viscoplastic behavior. The elastic threshold (γ^E) defines the critical cyclic shear strain level where the reduction of stiffness begins. A second critical strain level, where the sample behaves as a viscoplastic material and has reached the steady state condition is defined as the flow threshold (γ^P). These thresholds can be determinate from approximate correlations with the plastic index.

3. DISCUSSION

According to literature, the generation of excess pore pressure is fundamentally a strain dependent rather than stress dependent process. Therefore, since the cyclic strength of soil appears to depend on generation of excess pore pressure, it follows that the cyclic strength deterioration is also fundamentally a strain dependent process.

With sands, when the excess pore pressure builds up to such a value, an effective stress failure is imminent. Apparently clay soils do not have a metastable interparticle structure. Then, there is no evidence of a sudden structural collapse and accompanying transient increase in pore pressure followed by large strains on succeeding cycles.

On the basis of proposed conceptual frameworks of soil behavior based on strain regimes (Vucetic 1994, Santamarina 2001) and stress-strain response (Jardine, 1992), the following modified points are presented:

The **very small strain elastic regime** is a region characterized by approximately constant stiffness, bonds at molecular level remain unchanged, very small energy losses are presented and there is not generation of pore pressure. The **linear threshold shear strain (γ_{tl})** separates the very small from small strain regime and is represented by the strain level at a value of G/G_{max}=0.99. Experimental determination indicates that for most clayey soils this threshold strain is on the order of 0.001% to 0.005%.

In the **small strain regime**, soil presents a non-linear stress-strain behavior and minor fabric change; there is not accumulation of pore pressure during undrained cyclic loading or volume change for drained conditions. The **volumetric cyclic threshold strain (γ_{tv})** represents the limit between small strain regime, with fully recoverable behavior and medium strain regime, with minor strength degradation. This threshold has been derived from strain controlled conditions (Vucetic 1994) and stress controlled conditions (Matsui et al. 1980, Ansal and Erken 1989). Their value relates to a modulus reduction (G/G_{max}) between 0.6 and 0.85. For silts and clays having plastic index from 14 to 30, the γ_{tv} is between 0.024% to 0.06% (Hsu and Vucetic 2004, 2006).

At the boundary of the medium strain regime, exist a **degradation strain threshold (γ_{td})** which represents the higher strain value that reflect the decisive de-structuring of the specimen and separates the medium strain regime from the large strain regime. The **degradation strain threshold (γ_{td})** has been derived from stress controlled conditions (Díaz-Rodríguez 1989) and adapted to strain threshold by Díaz-Rodríguez and Santamarina (2001).

This threshold has been examined from stress controlled test and its existence has been confirmed by a great number of investigators mentioned previously. This degradation threshold has been represented as a critical level of repeated loading in which soil failure will never occur. Below this, a hysteretic equilibrium behavior and a nearly elastic pore pressure response was observed. Some clays tested to determinate this threshold present high residual pore pressure under the critical stress ratio, which may accelerate creep phenomena and eventual failure if pore pressure is not allowed to dissipate. To evaluate this variable, in some test a large number of cycles have been applied ($3x10^5$ cycles -Houston and Herrmann 1980, $1x10^5$ cycles- Malek et al. 1989-, $3x10^4$ cycles -McCarron et al. 1995-), however, even for offshore structures, undrained shearing is unlikely to persist for more than 1000 cycles (Karlsrud and Haugen 1983).

Below this critical cyclic stress, it has been determined that most of the deformations and the accumulation of pore pressure are generated in the first cycles (Lefebvre 1989, McCarron et al. 1995) and gradual stabilization or

a strain-hardened behavior is presented. During the test, these specimens exhibit a cumulative axial strain that can be evaluated in terms of a strain threshold. Some values of this strain that have been reported are: γ=2% (maximum) for seafloor soils (Houston and Herrmann 1980), γ= 0.5% to 0.9% for sensitive clay from the Hudson Bay (Lefevbre 1989), γ=3% for México City Soil (Díaz-Rodríguez and Santamarina 2001). Below this degradation threshold, the clay structure remains relatively unaltered in such form that the post-cyclic strength reaches the original peak failure envelope and approximately the initial elastic modulus.
For the stiffness reduction curve indicated in Figure 1, the only variable is the number of cycles, therefore the volumetric strain threshold is approximately unique for a given clayey soil and the degradation strain threshold depends on the applied approach: for a certain number of cycles (Díaz-Rodriguez, 1989), when the pore pressure and shear strain have been stabilized (Lefebvre et al. 1989) or for a number of cycles that pursue a practical purpose.

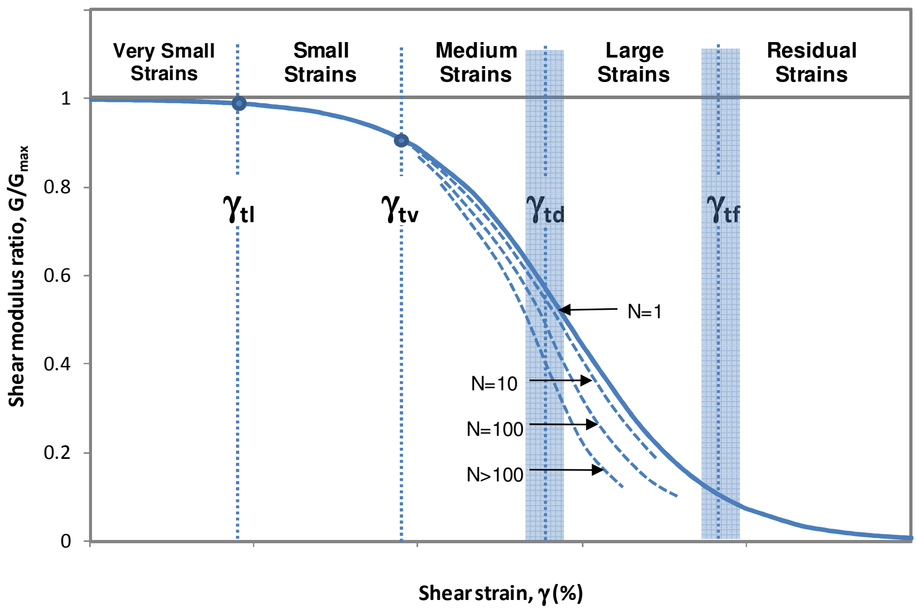

Fig. 1. Variation of dynamic shear modulus with shear strain amplitude. Proposed Strain regimes and strain thresholds.

For the **large strain regime** the de-structuring of initial fabric is exhibited, the primary deformation mechanism is related to fabric changes and high energy losses are presented during cyclic loading. The **flow threshold (γ_{tf})** defines the transition point where the sample behaves in a viscoplastic mode and reaches a steady state phase. Okur and Ansal (2007), define this strain threshold as the point where the rigidity is approximately 10% of it is initial value.
Figure 2 shows the strain regimes and thresholds proposed for saturated clayey soils subjected to undrained cyclic loading, thereby summarizing the behavior that distinguishes each regime in relationship with stress-strain behavior, stiffness degradation and pore pressure generation characteristics.

4. CONCLUSIONS

A survey of data available from the literature leads to the following conclusions concerning the strain thresholds of fine grained or clayey soils.
An alternative strain regime division for cyclic behavior of clayey soils was proposed, the strain thresholds that delimit each regime were defined on the basis of changes in cyclic stress-strain behavior, stiffness degradation, pore pressure generation, post-cyclic strength and microscale processes. Each strain threshold value is influenced to a different degree by factors like type and physical state of the soil (fabric, composition, plasticity, void ratio), stress history (OCR, stress state, aging effects) and loading conditions (modality and rate of loading, number of cycles, drainage conditions).

This approach unifies diverse criteria related to the cyclic behavior of clayey soils for different strain regimes and in this form an assessment tool was proposed to diminish uncertainly in the selection of design parameters and estimation of the response of structures subject to cyclic loads.

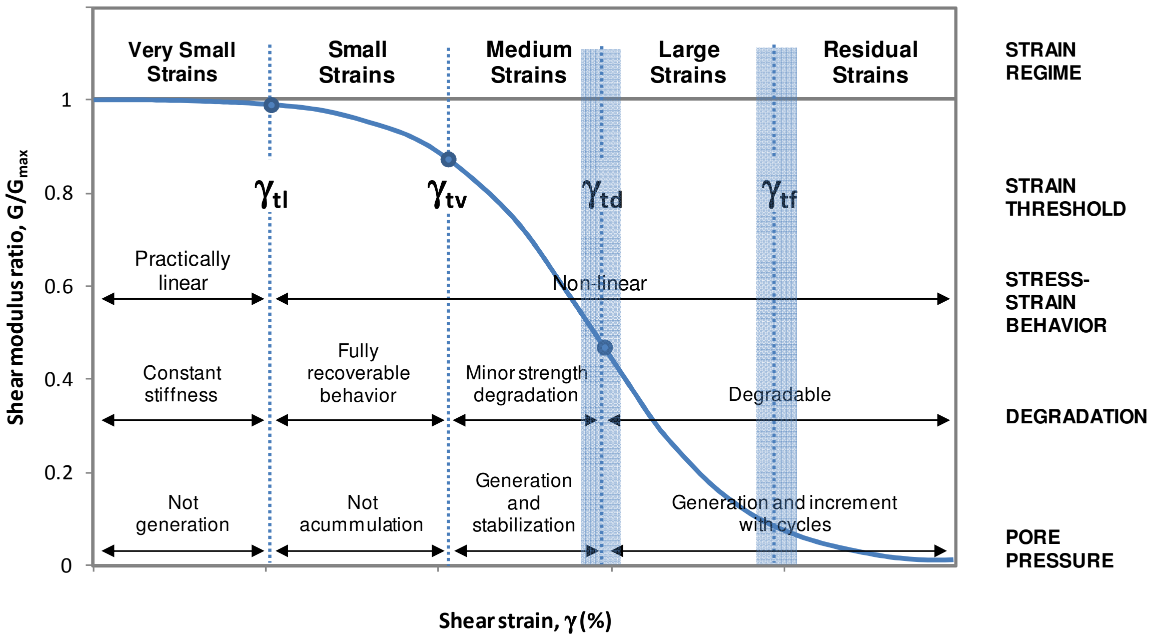

Fig. 2. Soil behavior between proposed strain thresholds for saturated clayey soils.

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