Soil Dynamics and Earthquake Engineering 53 (2013) 210–216

Contents lists available at ScienceDirect

Soil Dynamics and Earthquake Engineering

journal homepage: www.elsevier.com/locate/soildyn

Shear modulus degradation model for cohesive soils

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ARTICLE INFO

Article history:
Received 15 February 2013
Received in revised form
5 July 2013
Accepted 7 July 2013
Available online 9 August 2013

Keywords:
Stiffness degradation
Modulus reduction
Masing rule
Damping correction

ABSTRACT

One of the key issues in cyclic behaviour of soft clays is gradual degradation of shear modulus. In most of the cyclic soil models such degrdation of shear modulus of soil with the progression of loading cycle was incorporated, addition to the standard non-linear backbone curves. Such cyclic degradation was usually represented by a parameter, degradation index, which is a function of loading cycles and cyclic shear strain amplitude. However it is well understood from the past experimental studies that the degradation index depends on various other factors as well. The present paper aims to develop a simple empirical model involving degradation index as a function of number of loading cycles, plasticity index, cyclic shear strain, overconsolidation ratio, loading frequency based on the experimental results. It is then fitted with the hyperbolic hysteretic model to estimate the modulus degradation for different cycles. Finally the damping ratio is calculated based on Masing rule with correction factors and validated through experimental results.

1. Introduction

Degradation of soft clays during cyclic loading has been well reported in past studies involving element tests [1,2]. This is particularly significant with respect to the stability of onshore or offshore structures subjected to repeated loadings such as, earthquakes or ocean waves. As a result, in most of the cyclic soil models, substantial decrease of shear modulus of soil with the progression of loading cycle was incorporated, addition to the standard non-linear backbone curves [1,3]. Idriss et al. [1] proposed a simple degradation model for soft marine clay under cyclic and transient loading based on their experimental results. Cyclic degradation, as described by Idriss et al. [1], depends on a parameter termed as degradation index which is expressed as a function of cyclic strain amplitudes and number of cycles. Matasovic and Vucetic [4] further extended Idriss's concept of degradation index to incorporate effect of excess pore pressure. In addition to the cyclic strain amplitudes and number of cycles, it was also noted that the overconsolidation ratio and plasticity index also affect degradation [3,5].

Researchers in past proposed various formulations of degradation index. However most of such models considered degradation index as a function of two parameters such as, number of cycles and cyclic strain amplitude (e.g., Idriss et al. [1]; Pyke and Beikae [6]). Zhou and Gong [7] represented the degradation index based on number of loading cycles, loading frequency, overconsolidation ratio and cyclic stress ratio (*CSR*). However the model did not account for plasticity index and cyclic shear strain. Furthermore Maugeri et al. [8] discussed that the testing methods (such as cyclic loading torsional shear test, double specimen direct simple shear test, cyclic loading triaxial test etc) also influence the shear modulus reduction curve. The present paper aims to develop a simple empirical model involving degradation index as a function number of loading cycles, plasticity index, cyclic shear strain, overconsolidation ratio and loading frequency based on the experimental results reported in the past studies. Secondly, the proposed degradation index formulation in concurrence with the hyperbolic backbone curve is extended to study the influence of various parameters on modulus reduction curves and validated through the experimental results available in the literature. Finally the damping ratios for various types of soils with varying plasticity index will be compared with published experimental results.

2. Formulation of proposed model

The proposed model encompasses hyperbolic backbone curve with hysteretic stress–strain behaviour. The hysteretic stress–strain behaviour for unloading and reloading is modelled using the Masing rule [9]. The progressive degradation of the backbone curve under repeated loading is modelled using a modified form of Idriss's concept [1] of degradation index.

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http://dx.doi.org/10.1016/j.soildyn.2013.07.003

2.1. Hyperbolic–hysteretic stress–strain behaviour

The basic backbone curve for non-linear cyclic response of marine clay can be represented using hyperbolic relationship proposed by Nasim [10],

$$q = q_f - \frac{G_{\max}}{R + R^2 \varepsilon_s} \tag{1}$$

where, q is the deviator stress, q_f is the deviator stress at failure, ε_s is the generalised shear strain and assuming R as the modulus ratio,

$$R = \frac{G_{\max}}{q_f} \tag{2}$$

where, $G_{\max}$ is the small strain shear modulus.

2.1.1. Formulation of G_{max}

$G_{\max}$ can be represented as a function of mean effective confining stress and overconsolidation ratio (OCR) as proposed by Viggiani and Atkinson [11]

$$G_{\max} = A(P')^n (OCR)^m \tag{3}$$

where,

Mean effective confining stress (P'),

$$P' = \frac{1+2k_0}{3} \sigma_v{}'$$

where, k_0 is the coefficient of earth pressure at rest condition and $\sigma_v{}'$ is the effective vertical stress. Parameters A, m and n depend on plasticity index (PI) of soils [11] and can be expressed from the regression analysis of the experimental results as,

$$A = 5246 \exp(-0.041 PI) \tag{4}$$

$$n = 0.1259 \ln(PI) + 0.3746 \tag{5}$$

$$m = 0.0019 PI + 0.1788 \tag{6}$$

Fig. 1a and b shows the variations of the parameters A, m and n with plasticity index (PI) plotted along with the experimental results reported by Viggiani and Atkinson [11].

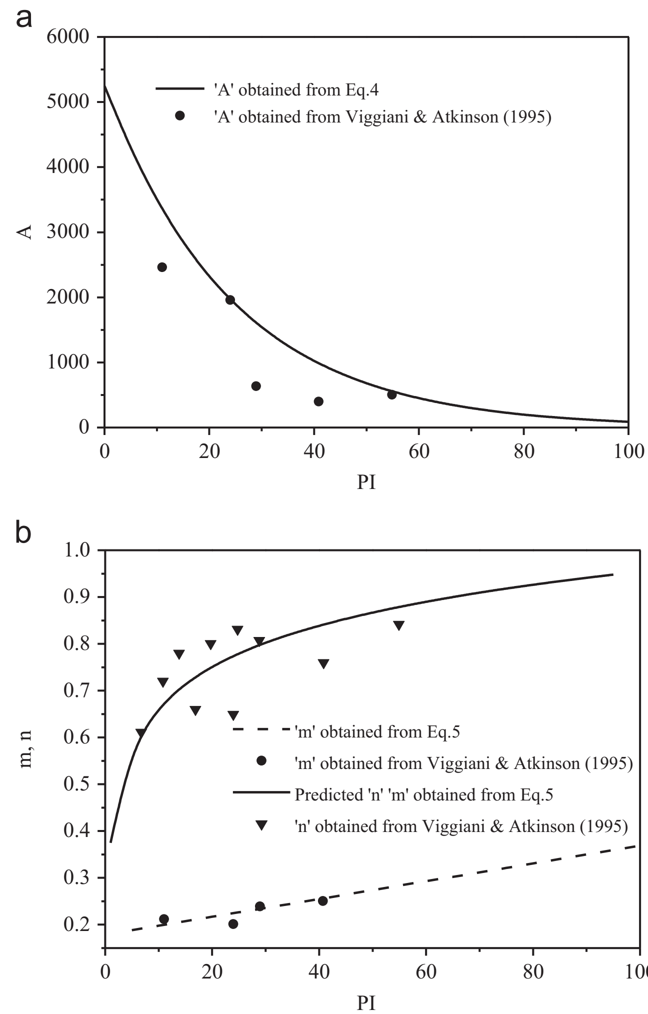

Fig. 1. (a). Comparison of computed and experimental stiffness parameter '*A*' Vs. plasticity index (*PI*). (b). Comparison of computed and experimental stiffness parameter '*n*' and '*m*' Vs. plasticity index (*PI*).

2.1.2. Formulation of q_f

The deviator stress at failure, q_f, is given by,

$$q_f = \frac{Mp'}{2^{\Delta}} \tag{7}$$

where, M is the slope of the critical state line in q'–p plane, p' is the mean effective confining stress and assuming,

$$\Delta = \frac{\lambda - \kappa}{\lambda} \tag{8}$$

λ and κ are the slopes of the normal compression line and recompression line, respectively and can be expressed in terms of plasticity index [12],

$$\lambda = 0.02 + 0.45 PI \tag{9}$$

$$\kappa = 0.0008(PI - 4.6) \tag{10}$$

where PI is the plasticity index.

From the Eqs. (8), (9) and (10) Δ can be represented as,

$$\Delta = \frac{0.023864 + 0.00366 PI}{0.02 + 0.0045 PI} \tag{11}$$

M can be expressed in terms angle of friction (φ) as [13],

$$M = \frac{6 \sin \phi}{3 - \sin \phi} \tag{12}$$

Angle of friction, in turn, can be related to plasticity index (PI) as [14],

$$\sin \phi = 0.814 - 0.234 \log PI \tag{13}$$

From Eqs. (12) and (13) M can be represented as a function of plasticity index (PI),

$$M = \frac{4.884 - 1.404 \log PI}{2.186 - 0.234 \log PI} \tag{14}$$

By substituting Eqs. (8)–(14) in Eq. (7), deviator stress at failure, q_f, can be expressed in terms of plasticity index (PI) as,

$$q_f = \frac{\left(\frac{4.884 - 1.404 \log PI}{2.186 - 0.234 \log PI}\right) P'}{2^{((0.023864 + 0.00366 PI)/(0.02 + 0.0045 PI))}} \tag{15}$$

2.1.3. Secant shear modulus

The secant shear modulus at the load reversal point can be represented as,

$$G_{\sec} = \frac{q_r}{\varepsilon_r} \tag{16}$$

where, q_r is the deviator stress at load reversal point and ε_r is the reference strain