Site response analysis of Christchurch soil sites using a non-linear model

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ABSTRACT: A simple model that can produce any level of damping for a given strain level or damping curve as observed in the laboratory tests is presented. This comprises of modified Masing unloading-reloading rule and a modified Kondner and Zelasko (MKZ) hyperbolic as the backbone curve. The model is used to scrutinize the effects of damping on the one-dimensional seismic site response analysis which involves the computation of the response of a semi-infinite horizontally layered deposit overlying a uniform half-space subjected to vertically propagating shear waves. The model's predictive capacity through simulations of observed ground motion records during the 2010 Darfield earthquake is examined. In addition, results are compared with the equivalent linear site response procedures conducted at the same strong motion station and the similarities and differences of the two approaches are discussed. The paper discusses the advantages and limitations of the equivalent linear and nonlinear models in terms of simulating soil nonlinearity and associated material damping. Cyclic triaxial test data on sand samples sourced from Christchurch have been used for the required shear modulus reduction and damping curves. The equivalent linear ground response analyses are carried out using Strata and the nonlinear total stress analyses are performed using OpenSees incorporating the originally developed stress-strain model.

1 INTRODUCTION

The dynamic response of soil deposits beneath a site has a significant influence on the ground motion hazard of engineered structures. The properties that typically need to be determined in order to characterize a particular soil site include shear modulus, G, and material damping ratio, h. Shear modulus represents the shear stiffness of the soil and can be approximated as degree of inclination of a loop in the case of dynamic loadings. Damping ratio, h, is a measure of the dissipated energy during a single cycle of shear deformation or simply a measure of breadth of the loop. The relationship between secant shear modulus, G_s, and shear strain amplitude is commonly characterised by shear modulus reduction curves. Furthermore, the nonlinearity in the stress-strain relationship, which leads to energy dissipation per loading cycle, results in the material damping ratio, h which increases with increasing shear strain.

Mathematical models, which are capable of predicting soil response, are required in order to theoretically understand local site effects. The complexity of the plastic behaviour of soils is the reason for the existence of a large body of various classes to model soil response depending to the desired level of accuracy and simplicity. Three general broad classes of soil models have been proposed, namely equivalent linear models (Schnabel et al. 1972), cyclic stress nonlinear models (Ramberg and Osgood 1943; Matasovic and Vucetic 1993; Hashash and Park 2001), and advanced constitutive models (Mroz 1967; Momen and Ghaboussi 1982; Dafalias 1986; Kabilamany and Ishihara 1990; Gutierrez et al. 1993; Cubrinovski and Ishihara 1998). On the one hand, the equivalent linear analysis is the simplest and most widely employed scheme for 1D analysis but has several important limitations. On the other hand, advanced constitutive models which establish a relationship between the rates of the strain and stress tensor, are in principle applicable to any arbitrary strain or loading path for 2D or 3D problems. However, numerous parameters which must be determined through laboratory and field tests limit its use for many practical problems which may lie far from those used for the calibration of the model (Kramer 1996).

In many practical cases such as level ground response analyses, the boundary value problem under study is restricted to one spatial dimension, and the soil is deformed in the simple-shear conditions. In such cases if the excessive pore pressure due to strong ground motion is negligible, it may be more reliable to construct a constitutive model which can approximately simulate the relationship between the stress and strain components involved in the problem, rather than using the general tensorial representation or simple equivalent linear scheme. The models of this type called nonlinear cyclic models can adequately represent the shear strength of the soil during cyclic loading. A conventional way of constructing a cyclic shear model is based on the concept of a backbone curve which relates the shear stress amplitude, τ, to the shear strain amplitude, γ. With the backbone curve being defined, the next step consists in constructing a hysteresis loop under general cyclic loading conditions (Kramer, 1996).

Generally, the shape of the backbone curve is determined by the maximum shear modulus, G_{max}, shear strength, τ_{max}, and several curve-fitting constants. A set of unloading-reloading rules are assumed to describe the hysteresis loop in conjunction with the backbone curve. This is commonly done with the help of Masing rules. The first Masing criterion postulates that the tangent shear moduli at the reversal point of the unloading or reloading branches of the loop are identical to the initial shear modulus. Furthermore, according to the second criterion, if a stress reversal occurs at a point defined by (γ_a, τ_a) the hysteresis curves are obtained as the corresponding parts of the backbone curve enlarged by a factor of $n = 2$, this is expressed mathematically by the following equation and shown in Figure 1a:

$$\frac{\tau - \tau_a}{n} = F_{bb}\left(\frac{\gamma - \gamma_a}{n}\right) \tag{1}$$

where F_{bb} = backbone function; τ, τ_a = shear stress; γ, γ_a = shear strain; n = Masing coefficient.

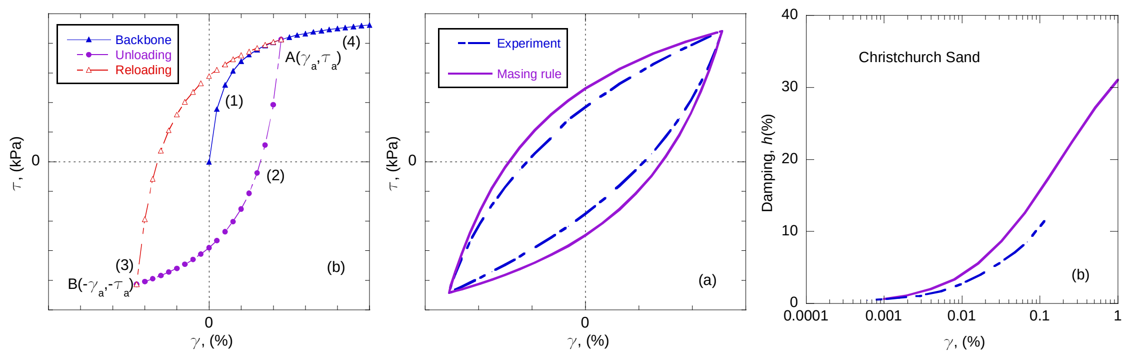

Figure 1 (a) Hyperbolic backbone curve and Masing unloading-reloading branches, (b) Comparison of experimental and Masing-based calculated hysteresis loops, (c) Experimental and Masing-type calculated damping ratio curves

By applying Masing rule to a hyperbolic model, Ishihara (1996) showed that the damping ratio converges to a limiting value of $2/\pi = 0.637$ when shear strain amplitude becomes infinitely large. Experiments give the maximum values of damping ratio for sands lying in a vicinity of 0.3 which is considerably smaller. Therefore, the magnitude of damping predicted by Masing rule is not supported by experimental test results (Hardin and Drnevich, 1972, Ishihara, 1996). Figures 1b-c illustrate that using Masing criteria, the area of the hysteresis loop is greater than that measured by experimental test data resulting in overestimation of damping ratio. The overestimation of hysteretic damping induced by employing Masing criteria can unconservatively lead to underestimation of some of the seismic response parameters such as peak ground acceleration. Therefore, the application of Masing's rules does not provide an adequate approximation simultaneously for shear modulus and damping ratio.

The issue of how to approximate simultaneously both the shear modulus and the damping ratio has been addressed by Muravskii and Frydman (1998) but the proposed model was rate dependent. Later Osinov (2003) followed similar approach to develop unloading-reloading curves which were independent from the backbone curve. Another solution of the aforementioned damping problem with

Masing criteria has been proposed by Phillips and Hashash (2009). While functional form provided consistency between the experimental and mathematical damping curves, the tangent modulus at the point of reversal reduces due to reduction factor and therefore is not equal to G_{max}. This is in contrary to the first Masing rule.

Given a soil model for symmetrical loadings, Pyke (1979) proposed an alternative unloading-reloading rule in which the Masing coefficient n can deviate from two, in order to extend the Masing model for use with irregular loadings. A factor n greater than two allowed simulation of cyclic hardening, while cyclic softening could be modelled by assuming a value of n less than two (Lo Presti et al. 2006). Likewise, it can be illustrated that the same idea can be employed in order to simulate any target damping ratio curve by modifying the Masing criterion.

2 STRESS-STRAIN MODEL FOR CYCLIC CHARACTERIZATION OF SANDS

The hysteresis curve for the stress-strain relationship is constructed in such a way that it produces both the required backbone curve and the required damping ratio as functions of the strain amplitude for soil under cyclic loading. The backbone describing the monotonic stress-strain curve is the modified hyperbolic model developed by Matasovic and Vucetic (1993). The cyclic behaviour, or unloading-reloading branches, has been modelled using a modified version of Masing criterion. A parameter, ϕ, is introduced for the unload-reload curves and, the parameter, n, is allowed to vary depending on the desired level of hysteretic damping.

As was explained previously, the Masing rule allows us to construct hysteresis loops from a given backbone curve without resort to any other data. On the other hand, however, there exists a quantity, namely the damping ratio, which plays as important a role in the modelling of the dynamic phenomena as the backbone curve. With the use of the Masing rules, the damping ratio is obtained from the constructed hysteresis loops and cannot be prescribed in advance. Even if a backbone curve approximates the cyclic stiffness of the soil satisfactorily, it may turn out that the obtained damping ratio fails to be in good agreement with the experimental data. Therefore, it is desirable to try to describe the hysteresis loops which produce a prescribed damping ratio as a function of the strain amplitude.

To preserve the simplicity of the solution proposed by Masing (1926), as well as achieving a better agreement between the experimental and modelled hysteretic damping, two conditions need to be satisfied. First, for symmetrical periodic and cyclic loadings, after an unloading with a following reloading a point comes back into the reversal point where the unloading began (similar to the conventional Masing's model) forming a closed loop for any level of shear strain. In addition, the curves should be similar in shape to that of the initial loading curve. Second, the tangent shear modulus on each reversal point should assume a value equal to the initial tangent modulus for the initial loading curve, G_{max}.

To meet the first condition, antisymmetry should be satisfied namely two points A(γ_a , τ_a) and B($-\gamma_a$, $-\tau_a$) in Figure 1a**Error! Reference source not found.** should fall on the unloading and reloading branches. The unloading-reloading equation should contain as a parameter the absolute value of strain, γ_a, at which the point (γ, τ) leaves the backbone curve because of either unloading or reloading. This parameter remains without changes until the point (γ, τ) intersects the backbone curve and afterwards abandons it at a new value $-\gamma_a$. Introducing the power coefficient ϕ, this can be confirmed by expanding equation 1 and using modified hyperbolic equation to obtain:

$$\frac{\tau-\tau_a}{n}=\frac{G_{\max}\dfrac{\gamma-\gamma_a}{n}}{\left(1+\beta\left|\dfrac{\gamma-\gamma_a}{n\gamma_r}\right|^{\alpha}\right)^{\phi}} \tag{2}$$

It is obvious that points A and B both fall on the above curve considering a Masing coefficient n, to be equal to two and ϕ = 1. However, this may not be true for an arbitrary value of n. Solving above equation for ϕ, and for a general n-value, and entering point B in the equation yields:

$$\phi = \frac{\ln\left(1+\left|\frac{\gamma_a}{\gamma_r}\right|^{\alpha}\right)}{\ln\left(1+\left|\frac{2\gamma_a}{n\gamma_r}\right|^{\alpha}\right)} \tag{3}$$

Therefore, any adopted combination of ϕ and n which satisfies equation 3 will result to a closed loop hysteresis. The next step would involve the area of the loop to be in agreement with material damping curve, the former representing a measure of the hysteretic damping. A best n value can readily be obtained by iteration, matching the damping ratio from experimental test results and the one calculated by unloading-reloading rule. Once n is adopted, curvature variable ϕ, can be obtained using equation 3. It can be shown that the derivative of the unload-reload equation at the reversal points is equal to the initial tangent modulus and hence the second condition remains valid. In Figures 2a-b hysteresis loops are shown for values of strain amplitude γ = 0.4 and 0.8%. The model is calibrated using measured modulus degradation and damping data from triaxial tests on Christchurch sand samples under confining pressure of 100 kPa and n-γ relationships are produced for each element test. Given the n-γ curves as the input experimental damping properties, the model is employed in a C++ computer program called OpenSees (McKenna and Fenves 2001) for 1D site response analyses.

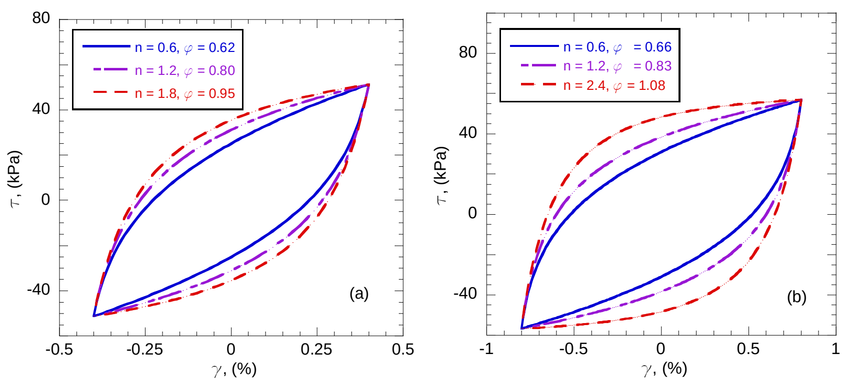

Figure 2 Varying Masing coefficient, n, and curvature parameter, ϕ for two strain amplitudes

In order to verify that the experimental damping is accurately simulated by the proposed model Figure 3 presents the comparison between the experimental and the modelled damping curves. It is clear that the simulated damping for medium to high strain levels can be accurately represented by the proposed formulation. Therefore, the suggested model allows us to regulate independently the behaviour of stiffness and damping and to reflect more precisely experimental results on cyclic loading of soils.

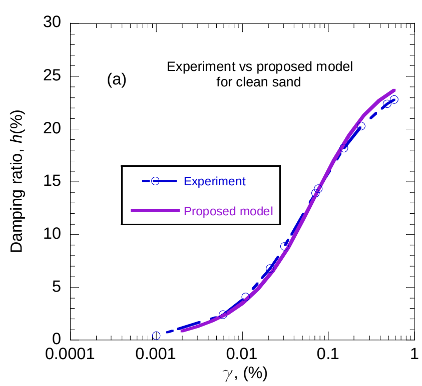

Figure 3 Comparison of the computed damping curves using the proposed model and the hyperbolic experimental input damping curve

3 STRONG MOTION STATIONS AND INPUT ACCELEROGRAMS

GNS Science, a New Zealand government-owned research institute developed GeoNet as a non-profit initiative to enhance the response to and preparation for natural hazards such as earthquakes in New Zealand. The New Zealand GeoNet is a strongly data collection and analysis system consisting of national and regional-scale sensor networks. In addition, GeoNet provides data and information for rapid event response (GeoNet ; Petersen et al. 2011). GeoNet strong motion records from ground motion stations which are located in Christchurch city were obtained for the purpose of this study. These stations recorded valuable time histories during the Canterbury earthquakes sequence, two of which was selected for the site response analyses in this paper, i.e. RHSC and CBGS stations. This station are placed on Springston formation which overlies on Riccarton gravel.

Based on in-situ geophysical test data measured by Wood et al. (2011) at these stations, the soil profiles were modelled for site response analyses (Figure 5). The required normalized modulus reduction and damping ratio curves for the analyses were calculated based on a series of cyclic drained triaxial tests conducted on sand mixtures obtained from a site in Christchurch, New Zealand (Arefi et al. 2012). All specimens were non-plastic and were tested triaxially under a confining pressure of 100 kPa; therefore the influence of the effective confining pressure and the plasticity index on the modulus reduction and the material damping curves could not be calculated. Darendeli (2001) developed an empirical framework in order to create a set of normalized modulus reduction and material damping curves incorporating the most important factors influencing the dynamic properties of soil. This model was calibrated utilizing the results of the tested soil in this study. The recommended values by Darendeli (2001) were assumed only for the corresponding parameters which could not be evaluated based on the limited number of tests available for Christchurch soil. Figure 4 illustrates the recommended curves computed based on Darendeli (2001) proposed framework employing cyclic triaxial test results of Christchurch sand.

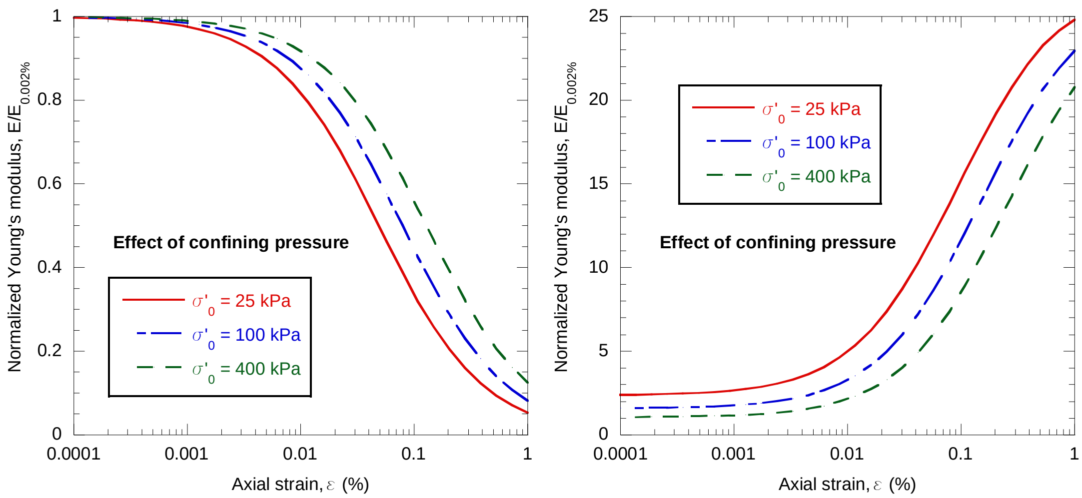

Figure 4 Recommended modulus reduction and damping curves based on Darendeli (2001) empirical framework and the cyclic triaxial test results of Christchurch sand

The method of analysis employed in the time-stepping procedure could in some respects be compared to the analysis of a structural response to input ground motion. Like a structure, the layered soil column was idealized as a multiple-degree-of-freedom lumped-mass system as shown in Figure 5. The stiffness and hysteretic damping of soil was represented with nonlinear hysteretic springs. The nonlinear springs can follow either Masing-type behaviour or the proposed hysteresis explained in this paper. Additional Rayleigh damping was considered for low-strain levels when the hysteretic damping is close to zero. The dashpot at the base of the model could be used with outcropping input motion otherwise a rigid base using within input motion recommended by Stewart et al. (2008) should be used. The control motion was specified at the bottom of the system of the lumped masses. Although, OpenSees allows analysis of multi-directional shaking, only one horizontal direction of shaking using lumped mass system is employed in this study.

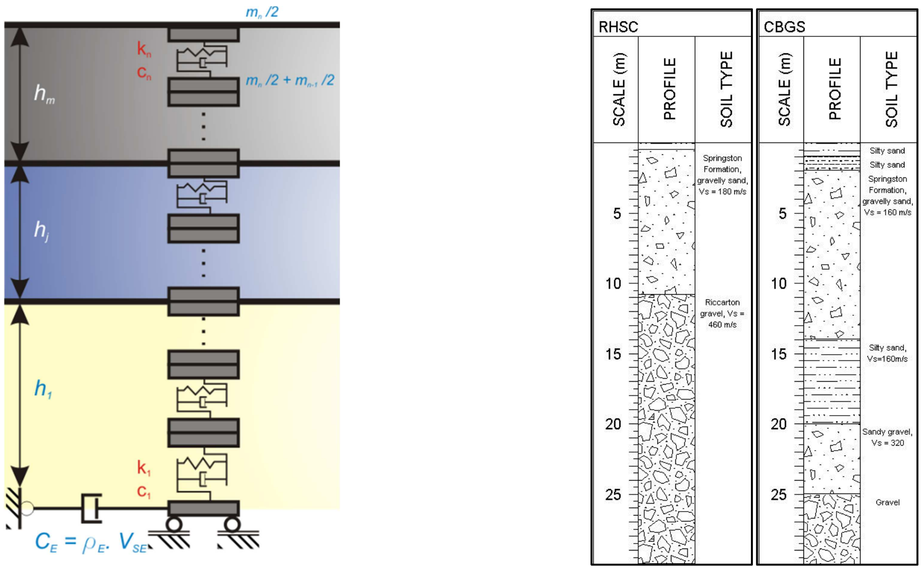

Figure 5. Schematic representation of the site response model (left), stratigraphy of the upper 30mat the RHSC and CBGS strong motion stations (after Wood et al. 2011, CERA 2012) (right)

The base rock incident motion was assessed using the given motion specified at the surface of the soil in RHSC station which overlies mainly on Riccarton Gravel and is expected to behave almost elastically during a moderate shaking. To determine the corresponding parameters at the surface of another station, the time history of ground surface motion recorded at point (RHSC) is used (Figure 6). This motion is then deconvolved through its soil profile as shown in Figure 3 in order to determine the time history of bedrock motion (at point B) that would produce the time history of motion at point A (Kramer 1996). The corresponding rock at outcropping motion produces the bedrock motion applied at the base (point D) of the soil profile at the site of interest. A conventional ground response analysis is then performed to predict the motion at the surface of the soil profile of interest (point E, CBGS in this paper). In the site response analysis of the other strong motion stations soil profile, the Masing-type behaviour can be compared with the proposed model which enables simultaneous simulation of the modulus and damping properties of the soil layers. However, further computational work is under investigation.

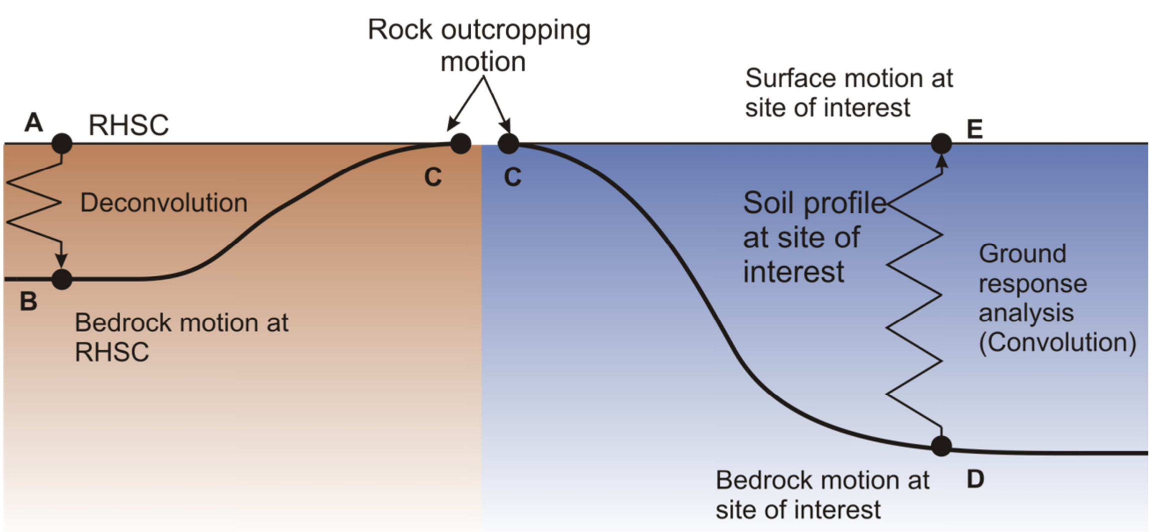

Figure 6. Procedure for modifying ground motion parameters using the deconvolution method in the RHSC site and conventional site response analysis in the site of interest (after Kramer 1996)

4 SITE RESPONSE ANALYSIS

A set of equivalent linear and nonlinear site response analyses, using the RHSC and CBGS strong motion station soil profiles were carried out in order to evaluate the influence of the induced hysteretic damping. The commonly used two-mode Rayleigh damping was employed to separately simulate the low-strain damping. The analysis results are presented in Figure 5 and include results using equivalent linear analysis (EQL), nonlinear analysis using Masing-type hysteresis (Masing), and nonlinear analysis proposed in this paper. Figure 5a presents that the equivalent analysis represent correctly the recorded motion at the surface of RHSC station. This is due to the fact that the input motion employed for such analysis was the deconvolved time history of the surface motion. Furthermore, both Masing and proposed model give a PGA that is lower than the equivalent approach. However, in the mid-period range (0.04-1s) they provide responses that are significantly higher than the EQL spectrum. Interestingly, all the results are similar for long period ranges.

The performance of the nonlinear site response analysis as well as the proposed formulation can be assessed employing the deconvolved fault-parallel motion for site response analyses of other strong motion stations such as CBGS in this case. In terms of PGA, all methods resulted in similar results; however Masing-type analyses slightly underestimated the PGA at the CBGS stations. In general, the proposed model predicted higher response values than the Masing-type formulation - except for the period range 0.04-0.1s. It is seen in Figure 5b that all methods overestimated the spectral acceleration at periods higher 1s. The overestimation of damping using Masing rules is more pronounced at larger strain levels and because the maximum shear strains computed in this profile were less than 0.1%, it is not conclusive whether the proposed model can adequately capture the response.

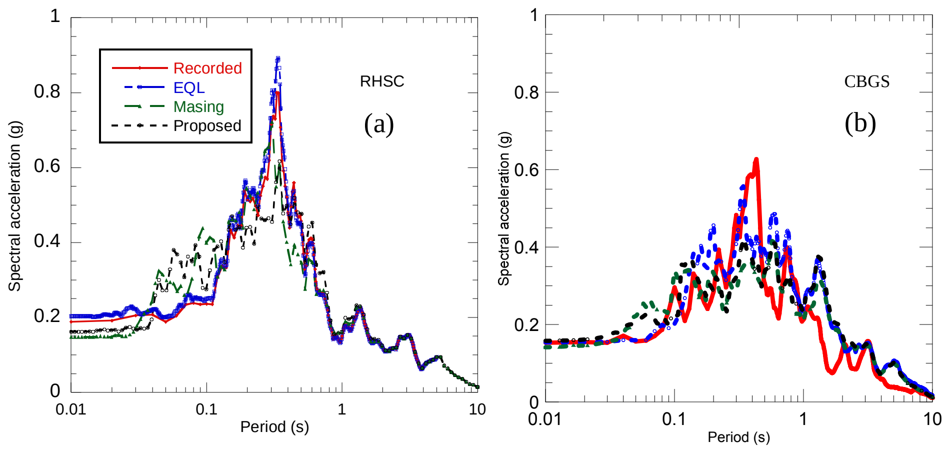

Figure 7 Surface response spectra comparison for RHSC (a), and for CBGS (b) strong motion stations

5 CONCLUSIONS

A new simple equation was proposed for modelling of unloading-reloading branches of cyclic stress-strain hysteresis loops for sandy soils. The proposed model uses the hyperbolic model as the backbone curve to represent the modulus reduction curve. It was shown that the model is capable of capturing any desired level of energy dissipation as a function of shear strain in contrast to conventional models which tend to overestimate damping. Therefore, both the modulus reduction and damping curves can be simulated simultaneously. In a further attempt, the proposed model was employed to simulate the nonlinear behaviour of two profiles which underlie two strong motion stations. The equivalent analysis, nonlinear analysis using Masing criteria, and nonlinear analysis using the proposed model were compared in order to study the effect of modelling damping in the surface ground motion.

6 ACKNOWLEDGEMENT

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