Site-Specific Dynamic Ground Response Analysis

1.0 Introduction

Caltrans Seismic Design Criteria, Version 2.0, with October 2019 Interim Revisions or SDC v2.0 (Caltrans, 2019a) provides the definitions of and the requirements for the development of the design response spectrum for Ordinary and Recovery bridges. The *Design Acceleration Response Spectrum* (*Design ARS)* module provides procedures for developing the seismic design response spectrum as per SDC v2.0 using the current version of the *ARS Onlin*e (Caltrans 2020a) web tool. The design response spectrum is developed in the form of the Design Acceleration Response Spectrum (Design ARS) for 5% damping and included, with other relevant design ground motion parameters, in the bridge foundation reports.

The *Design ARS* module does not address the development of the Design ARS for projects sites underlain by liquefiable, weak or soft soil sites for which a site-specific dynamic ground response analysis (DGRA) is required per Appendix B of the SDC v2.0

The purpose of this module is to provide information, guidance and requirements on the general methodology, procedures, practices, and guidelines for performing site-specific DGRA for Caltrans' bridge project sites when such an analysis is required per SDC v2.0.

Familiarity with the development of the design ARS for Caltrans bridge project sites per SDC v2.0, and in accordance with the procedure in the *Design ARS* module is assumed. Working level understanding on the fundamental concepts and principles of geotechnical earthquake engineering is necessary to perform a dynamic site response analysis.

The technology for site-specific DGRA is relatively new and rapidly evolving. This module will be updated as necessary to reflect new developments, and related Caltrans policies and practices.

2.0 Purposes of a Site Specific DGRA

The characteristics of earthquake-induced ground motions generated at or near the ground surface of a project site depend on a number of factors, including seismological, geophysical, geologic, topographic conditions and geotechnical parameters. The most important factors are the earthquake magnitude, the site-to-source distance, and the local soil conditions.

Other significant factors include the earthquake source type and its characteristics (e.g., fault type and its geometry, seismogenic and focal depths, etc.), and the geology of the travel paths of the seismic waves from the source to the top of the basement rock at the site. For bridge design, ground motions that reach at or near to the ground surface within are important.

For engineering analysis and design of structures, it is generally considered that effects of the earthquake magnitude, the site-to-source distance, the characteristics of the seismic sources (e.g., fault type), and the travel path through the rock are relatively well defined or constrained, and easily incorporated in the semi-empirical Ground Motion Models (GMMs), also known as the Ground Motion Prediction Equations. Based on recent advances, it is now considered that the potential effects of the local subsurface profiles consisting of non-liquefiable and firm soil layers are also well constrained by the GMMs. This means, for these soil sites, future ground motions at or near the ground surface may be predicated with acceptable reliability by direct use of the available empirical GMMs. For these site, the design ARS for the Caltrans bridge design are developed using the *ARS Online* web tool per procedures presented in the *Design ARS* module.

However, for project sites underlain by soft, weak or liquefied soil layers the potential effects of the portion of the seismic wave travel path from the top of the basement rock to the ground surface through the soil layers are not well constrained due to numerous complexities. Such complexities include highly non-linear shear stress-strain characteristics of such soils, and substantial progressive degradations in the strengths and stiffnesses of saturated, loose to medium dense granular or cohesionless soils ground shaking due to significant positive excess porewater pressure generation, including complete liquefaction. Complexities also arise due to natural variations in the types, conditions and layering of soils, and in the peak magnitude and temporal characteristics of the input ground motions at the site.

For the latter categories of sites, a dynamic ground response analysis which includes geotechnical engineering evaluation of the responses of the local soil layers to the seismic shear waves (SH) propagating from the top of the basement rock to the ground surface and vice versa (reflecting waves), is necessary to reliably predict the future ground motions at the ground surface or any other elevations within the soil profile. For a site-specific DGRA, the design ground motion at the top of the basement rock may be reliably evaluated based on the applicable empirical GMM and assuming “rock outcropping” (i.e., if there were no soils overlying bedrock) conditions at the site. The predicted design ground motions at the top of the bedrock, termed as the target design motions in a DGRA, is then applied as the input motions to the bottom of the soil profile. A numerical dynamic ground response analysis is then performed incorporating relevant soils characteristics by means of soil model parameters, as discussed later in this module, to predict the corresponding design motions at the ground surface or any depths required within the soil profile.

Figure 1 presents a schematic representation of the travel paths for seismic waves and the definitions of several terms used in a DGRA.

In the context of this module, a site-specific DGRA is a numerical dynamic ground response analysis used to evaluate the dynamic responses of the soil profile at a project site using site-specific soil properties/parameters. A computer code is required to conduct such analysis. This analysis includes determination of the target design ground motion at the top of basement rock. In some specific cases, the target design motion may be evaluated at some elevation within the soil profile below which the site-specific ground conditions can be characterized as “firm-ground”, as defined later in this module.

A site-specific DGRA analysis may be conducted for other purposes, including the evaluation of soil liquefaction hazards, and the generation of design ground motion time-histories at different depths within the soil profile at sites not included in the scope of this module. Such an analysis is outside the scope of this module.

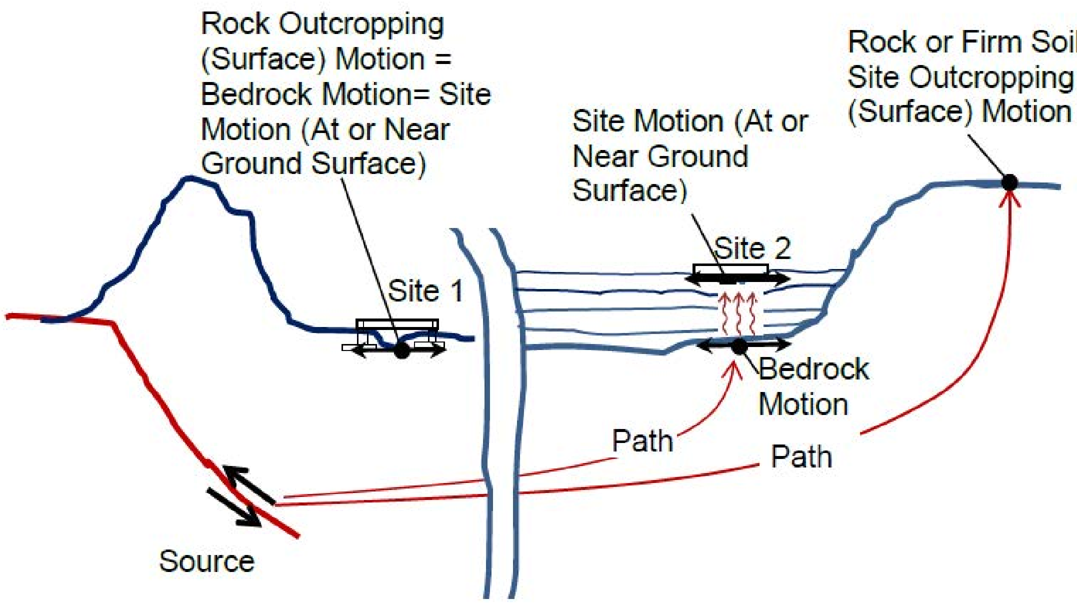

Figure 1. Schematic Representation of DGRA with Various Terms Used: (a) Site 1- No Soils Overlying Bedrock, and (b) Site 2 – Soils Overlying Bedrock

The scope of the site-specific DGRA covered in this module is limited to the development of the design ARS at or near the ground surface given a target design ARS, developed per SDC v2.0 and using *ARS Online,* at the top of the basement rock or "firm-soil" base of the soil profile, as defined later in this module.

Depending on the types and conditions of the site soils, and the characteristics (e.g., Peak Ground Acceleration) of the target design ground motion, a site-specific DGRA may be performed using either an Equivalent-Linear (EL) or a Non-Linear (NL) analysis method. The EL is a simplified DGRA method in which the non-linear soil behaviors are only approximately modelled. It is based on the principles of total stress, and appropriate for use at project sites where seismically induced soil shear-strains are small. A NL analysis can model non-linear soil behaviors more accurately.

Unless specified otherwise, a NL analysis may be used at all project sites. A NL analysis may be performed in terms of the total stress or effective stress method, depending on the soil and groundwater conditions. Both DGRA methods are discussed below. The EL analysis method provides useful background for the non-linear analysis methods.

3.0 Steps in a Site Specific DGRA

A site-specific DGRA consists of the following main steps. Additional information on each of these steps are presented in the following sections.

1. Determine if a site-specific DGRA is or likely to be required for the site.

2. If the answer to the above question is yes, determine the appropriate method(s) of dynamic ground analysis (e.g., EL, NL total stress-based, or NL effective stress-based) for the site and a computer software to perform the analysis.

3. Identify the required soil information and parameters that will be required for the analysis.

4. Plan and conduct a site investigation required to obtain the required soil information and parameters identified in Step 2.

5. Perform site characterization to evaluate the required subsurface information, including the soil/rock properties and parameters. Develop idealized design soil profile(s) with basic soil parameters and groundwater conditions.

6. Confirm the need for a site-specific DGRA based on the results of the subsurface investigation, site characterization and other analysis, as appropriate. If an analysis is needed, confirm/select the appropriate analysis method and the computer code.

7. Develop a detailed 1-D soil-column model by dividing the idealized soil profile layers into a number of sub-layers appropriate for a DGRA analysis. Include the elevation and depth of the soil-column model base.

8. Identify the soil model(s) and, when required, the excess porewater generation model(s), to be used in the analysis for each soil layer or sublayer, as appropriate. Determine the appropriate values of the model and other parameters or options necessary to complete the input data file for the computer code.

9. Develop design input time-histories (target motions) at the base of the soil-column.

10. Perform DGRA analysis using the selected software, and review and verify the reasonableness of results.

11. Develop the Design ARS at the ground surface.

4.0 When to Perform a Site-Specific DGRA

A site-specific DGRA is performed only when required per Appendix B of the SDC v2.0 or if specified in a Project-Specific Seismic Design Criteria.

Per Appendix B of the SDC v2.0, a site-specific DGRA is required for the development of the final design ARS for a bridge site underlain by soil profile(s) meeting the following conditions:

1. The Soil Profile Type is E or F as defined in Table B.10 of the SDC v2.0, or

2. The soil profile contains one or more five (5) feet thick soil layers with a shear wave velocity (average) less than 120 m/sec.

Based on SDC v2.0, a site-specific DGRA is also preferred for the development of the preliminary design ARS for project sites meeting the above criteria.

For Soil Profile Type E, SDC v2.0 permits the use of the ARS curves presented in SDC v2.0 Figures B.11- B.13 for preliminary design, where applicable.

The Soil Profile Type F includes sites where soil liquefaction is predicted to occur based on the initial design ground motions or ARS at the ground surface determined assuming non-liquefaction conditions. This ARS is determined using the measured V_{S30} and the *ARS Online* tool as discussed in the *Design ARS* module.

A soil profile with >120 feet of soft/medium stiff clay layers is also included in the Soil Profile Type F. In this context, soft-to-medium stiff clay is defined as the clay soils with undrained shear strength, S_u<1,000 psf (NEHRP, 2020).

A site-specific DGRA due to soil liquefaction is not required for a site if any one of the following conditions apply:

1. The bridge is a single span structure.

2. The bridge has a fundamental period of vibration ≤0.5 second.

3. Only the surface soil layer is liquefiable.

4. The total thickness of all liquefied soil layers, excluding the surface layer, is ≤5 feet.

For a bridge site meeting the above exception requirement(s), and not requiring a site-specific DGRA for reasons other than soil liquefaction, the design ARS may be developed directly by using the most recent version of the *ARS Online* webtool with site or support-specific V_{S30} corresponding to non-liquefied soil conditions. Here, V_{S30} is the time-averaged shear wave velocity for the upper 30 m (100 ft) of the bridge site or support-specific subsurface profile.

For the procedure to determine V_{S30} as well as the development of the design ARS for firm and non-liquefied soil conditions, see the *Design ARS* module in the Geotechnical Manual. For how to evaluate liquefaction hazards, see the *Liquefaction Evaluation* (Caltrans, 2020b) module.

When a site-specific DGRA is required for a liquefiable soil site, develop and present the design ARS for both “non-liquefied” and “liquefied” soil site or ground configurations. Two different sets of seismic analysis and design, one for each of these two design ARS and the corresponding site or ground configurations must performed as per Section 11.5.4.2 of the AASHTO-CA BDS-8 (Caltrans, 2019b).

If the surface soil layer is also liquefiable, include it in the soil profile model for the site-specific DGRA. However, for bridge design, develop and recommend the site-specific DGRA based design ARS at the top of the non-liquefied soil layer underlying the liquefied surface soil layer. The potential effects of the liquefied surface soil layer on all other aspects of the bridge seismic design are required to be considered as usual.

The need for a site-specific DGRA should be evaluated and ascertained in consultation with the Structure Designer (SD) as early as possible during the project development phase.

A site-specific DGRA is not required to develop the design ARS for Caltrans project sites for which such an analysis is not specifically required per Appendix B of the SDC v2.0. ARS Online v3.0 is used to develop the design ARS for these sites. No reduction in the SDC design ARS are permitted. This also includes the design ARS developed using ARS Online for “non-liquefied” conditions of a site that is predicted to experience liquefaction during the design seismic events.

5.0 Site Investigation

A site-specific geotechnical investigation and subsurface characterization is required to conduct a DGRA. The need for a site investigation for DGRA and the detailed requirements for such an investigation should be evaluated as early as possible during the project development phase. Whenever possible, additional needs for a site-specific DGRA should be incorporated into the site investigation conducted for the preparation of the foundation reports. The *Geotechnical Investigations* module provides general requirements and guidelines for conducting a site investigation.

It is essential to thoroughly review this module, including the methods of DGRA analysis and the computer software used to identify the subsurface information required for an appropriate DGRA.

Plan and conduct the site investigation accordingly. Include both field and laboratory testing necessary to determine the required soil properties and parameters.

A site investigation for a DGRA should include borings with sampling, Standard Penetration Tests (SPT), Cone Penetration Testing (CPT), or P-S Suspension Logging Testing and other field investigation techniques, and laboratory tests. The purpose of the investigation is to characterize the site including soil layering, depth to groundwater table and depth to bedrock or bedrock-like “firm-ground”. Information for each soil layer, should include, but not limited to:

- Soil type or classification and description
- Total unit weight
- Gradation and relative density for coarse-grained soils
- Consistency and Atterberg Limits for fine-grained soils
- Shear strength parameters
- Permeability/coefficient of consolidation, and
- Dynamic soil properties for shear stress-strains curves

For additional information on the site characterization refer to the Geotechnical Manual, and, as necessary, to the FHWA GEC 5, Geotechnical Site Characterization (FHWA, 2017), the Manual on Subsurface Investigation (National Academic of Sciences, 2019), and the reference manual "LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations (FHWA, 2012).

Soil shear strength parameter should be determined using appropriate field and laboratory tests based on the soil types, and the dynamic nature of seismic loading and drainage conditions. See the Stewart el al. (2014) report for guidance on how to determine soil shear strength for a DGRA.

Soil permeability or, the coefficient of consolidation is necessary only when conducting an effective stress-based NL analysis.

Dynamic properties of site-specific soils are required to represent the soil responses to ground shaking in a site-specific DGRA. Soil responses to seismic loading in the field are represented by cyclic shear-stress strain curves or constitutive relationships, which may include the static backbone curve, a set of loading and unloading rules for modeling hysteretic soil behavior, and for the generation and dissipation of excess pore pressure during ground shaking.

Due to extensive work required and difficulties involved in directly establishing dynamic properties by field and laboratory testing of representative soil samples, widely published generalized empirical models, after calibrating to the site-specific soils types and conditions, are often utilized in a site-specific DGRA.

The model and curve-fitting parameters, necessary to develop the site-specific soil constitutive relationship from the generalized empirical material models, are often obtained from correlations with site-specific soil index parameters and other commonly used soil properties.

Dynamic shear stress-strain curves or constitutive models are often provided in the form of the variations of the normalized soil shear modulus (G/G_{max}) and the damping ratio (D) as a function of shear strain (γ), where G is the shear modulus at shear strain (γ) and G_{max}

is the initial or maximum shear modulus at γ=0.0. Both G/G_{max} and D, as a function of γ, may be determined by conducting dynamic laboratory tests such as the resonant column/cyclic torsional shear tests. High quality samples of the representative soils are required for such dynamic testing.

The most common dynamic soil properties required for a DGRA are the initial or small strain shear modulus (G_{max}) and the soil damping ratio (D_{min}). These two properties are used with curve fitting parameters to establish and specify the soil shear stress-strain curves for site-specific soils.

The curve fitting parameters, as discussed later, are specific to the generalized empirical models and are determined other soil properties.

The initial or small strain soil shear modulus G_{max} is determined based on soil unit weight (γ_t) and seismic shear wave velocity (V_s), as follows:

$$G_{max} = \rho\, V_S^2 \qquad (1)$$

Here, ρ is the total density of the soil = ($\frac{\gamma_t}{g}$), and g is the acceleration of gravity.

To determine G_{max} using Equation (1) in-situ measurements of the soil/rock seismic shear wave velocities (V_s) per the *Design ARS* module are required whenever a DGRA is conducted.

To evaluate if a site-specific DGRA per SDC v2.0 is required or not, variations in the subsurface soil profiles between the bridge intermediate support locations should be considered when selecting the number of locations at which in-situ measured V_s-profiles should be obtained and used in the site-specific DGRA analysis

For a site-specific DGRA analysis, the measured V_s-profiles at two or more intermediate support locations may be considered relatively uniform when their calculated V_{S30} values fall within $\pm$20% of their average V_{S30}.

For support locations meeting this requirement, the soil profile with a calculated V_{S30} at or near their average V_{S30} may be used in the site-specific DGRA to develop a single representative design ARS. The average V_{S30} profile provides the median site response for the applicable support locations (Rodriguez-Marek and Bray, 2006). When only two measured Vs profile locations are involved, the soil profile with the lower V_{S30} should be used to develop the representative design ARS when the structure natural period is 0.6 seconds or higher. For structure natural period <0.6 seconds, the soil profile corresponding to the higher V_{S30} should be used. Enveloping response spectra developed based on different considerations should not be used to develop the design response spectrum for a bridge site or a specific-support or a group of supports.

If the above criterion is not meet, separate DGRA analyses should be performed for measured V_s-profiles not meeting the requirements to develop design spectrum for the associated support(s).

The small strain soil damping parameter (D_{min}) is determined by conducting laboratory tests such as the resonant column/torsional shear tests or estimated by using empirical correlations with other measured soil properties, as discussed later in this module.

Additional soil properties and parameters may be required for a site-specific DGRA, depending on the soil type(s), the method of analysis and the constitutive relationship(s) used. Review the information presented below on the different types and methods of DGRA and the specific computer code used to determine the actual need for additional soil properties and parameters.

Also see the *Soil Liquefaction* module for the soil parameters necessary to evaluate soil liquefaction hazards. Some, if not all, of these soil parameters may be required or useful for DGRA for liquefiable soil sites.

6.0 Soil Profile Modelling

6.1 One Dimensional (1-D) Soil-Column Model

For a site-specific DGRA performed to meet the requirements in Appendix B of the SDC v2.0, the local soil effects are due to the dynamic responses of the soil layers to vertically propagating seismic shear waves (SH waves). It is assumed the site topography is relatively flat with uniform and horizontal soil and rock layers extending infinitely in the lateral (horizontal) directions. Based on these conditions, the soil profile(s) at a bridge site or support location(s) may be modelled as a one-dimensional (1-D) vertical soil-column with horizontal soil layer boundaries.

If the subsurface conditions at a bridge site, particularly along a long bridge alignment, vary significantly between supports, more than one representative 1-D soil profiles should be developed. In this case, to incorporate the effects of the spatial variations in the subsurface conditions along the bridge alignment, a separate 1-D DGRA is performed for each of the representative soil profiles. One representative 1-D soil profile or column may be used for multiple bridge supports provided the soil underlying the supports are not significantly different.

The top of the 1-D soil-column is taken at the finish ground surface.

The base of the 1-D soil-column analyzed is taken at the top of bedrock underlying the soil deposit at the site. Bedrock in this context is defined as earth materials with $V_S \geq$ 2,500 feet/sec (760 m/sec) underlying the soil profile.

If the bedrock at site is very deep (e.g., deep basins) from the bottom of the bridge foundations, the base of the 1-D soil-column may be taken at a depth in ‘firm-ground” meeting the following requirements:

1. The V_{S30} of soil/rock underlying the “firm-ground base” elevation is $\geq$1450 feet/sec (or 440 m/sec). This V_{S30}, termed herein as the “Base V_{S30}”, is evaluated as per the *Design ARS* module based on the measured seismic shear wave velocities of

the upper 100 feet of soils/rock directly underneath the "firm-ground base" elevation.

2. The average small strain shear wave velocity (V_s) for any soil layer > 5 feet in thickness underlying the "firm-ground base" elevation is not less than 1200 feet/sec (360 m/sec), and

3. The "firm-ground base" shall not be taken at a depth less than the greater of:

    - 100 feet from the finish grade

    - 2B or 20 feet, whichever is deeper, from the bottom of the bridge foundation. Here, B is the width or diameter of the foundation in feet. For a pile-group, B is the width of the cap-footing in feet.

The 1-D soil-column model includes soil layering, groundwater conditions, depth to bedrock or "firm-ground' base. The measured seismic shear wave velocity profile(s) should extend sufficiently below the "firm-ground base" to evaluate "V_{30}" and verify the above conditions for a "firm-ground base".

Each soil layer includes, but not limited to, the thickness, soil type and descriptions including relative density with $(N_1)_{60}$ and/or (q_{c1N}) for coarse-grained soils and silts (ML) or consistency for fine-grained cohesive soils, unit weight and appropriate shear strength parameters.

A graphical representation of soil model should be developed. The graphical soil model should identify and list the above soil information for each soil layer. The graphical model should also list the specific shear-stress strain model(s) used with the associated soil input and the curve-fitting parameters. For the effective stress- based NL analysis, include the models used for the excess pore pressure generation and dissipation, and the associated soil input and curve fitting parameters.

6.2 Soil Layer Thickness (h)

The soil layer thickness determines the maximum or highest frequency (f_{max}) ground motion that is transmitted through that soil layer. The maximum frequency f_{max} for a soil layer may be determined as follows:

$$f_{max}(Hz) = \frac{V_s}{4h} \qquad (2)$$

Where, V_s = Average shear wave velocity (ft/sec) of the soil layer, and h = Layer thickness (feet).

For a given V_s, f_{max} decreases as the layer thickness increases. Therefore, the layer thicknesses should be selected as small as feasible. The maximum thickness of the soil layers within the 1-D soil-column model should not exceed the thickness required (h_{max}) to transmit maximum frequency not less than 30 Hz (Hashash et al., 2020). For example, to transmit maximum frequency up to 30 Hz, h_{max} (ft) based on Equation 2 should be $\leq(\frac{V_S}{4\,x30})\ or\ (\frac{V_S}{120})$, where Vs is in $\frac{\text{ft}}{\text{sec}}$.

Layer thickness should be selected such that all soil layers within the 1-D soil-column have the same transmittable maximum frequency.

6.3 Definition of the Half-Space

As part of the 1-D soil-column model development, it is necessary to define the properties of the earth material below the base, referred to as the "Half-space. In a DGRA analysis, the half-space is defined as an "Elastic Half-space" or a "Rigid Half-space".

Target design input ground motions at the base of the soil column (or at the top of the half-space) will be developed based on ground motions that are recorded at the ground surfaces of sites underlain by subsurface conditions similar to those at the project site underlying the base of the soil-column (i.e., with the soil-column removed). In the DGRA analysis, these input ground motions are identified or specified as the "Outcropping Motions" since these are recorded at the ground surface.

When the target design input motions are identified as the "Outcropping Motions", the half-space in a DGRA must be identified as an "Elastic Half-space". The "Rigid Half-space" is associated with the target design input motions identified as the "within motions" (i.e., recorded at depths).

Only outcropping motions shall be used in the site-specific DGRA included in the scope of this module, and thus the "Rigid Half-space" option will not be the used.

In a DGRA analysis, the following input parameters for the earth materials underlying the soil-column base may be required to define the "Elastic Half-space":

1. Seismic shear wave velocity (or initial shear modulus)
2. Unit weight, and
3. Damping ratio

7.0 Development of Input Time Histories

7.1 Target Design ARS

In a DGRA analysis, the design input motion or acceleration time-histories are applied at the base of the 1-D soil-column to be analyzed.

The input acceleration time-histories are developed based on the design ARS developed per SDC v2.0 by assuming base outcropping conditions at the site (i.e., without the 1-D soil column). This design ARS, termed as the target design ARS and defined in terms of a L number of the discrete data points (T_i, X_i) is evaluated using ARS Online v3 and using V_{S30} for the upper 100 feet of the subsurface conditions directly underneath the base of the soil-column model. T_i and X_i are the period (second) and pseudo-spectral acceleration (in unit of acceleration gravity, g) of the single degree of freedom oscillator *i* (i=1, L).

The target design ARS is determined based on the spectral accelerations obtained directly from the USGS' 2014 National Seismic Hazard Map or NSHM (USGS, 2021a) accessed at https://earthquake.usgs.gov/hazards/interactive/ for the specified return period. ***Amplification factors due to near-fault and/or basin effects are not applied when developing the target design ARS.***

Alternatively, the same spectral acceleration data points for the target design ARS at the base may be obtained from the *ARS Online* v3.0 output data table. Care must be exercised to select the spectral data values corresponding to the USGS' 2014 NSHM, and not include near-fault or basin effects related amplifications factors.

The target design ARS is defined in terms of the spectral acceleration $(S_a)_{ROTD50}$ as defined by Boore (2010) and represents the 50th percentile, randomly oriented, single component design horizontal ground motion at the site corresponding to the specified design return period. That is, $Xi = (S_a)_{ROTD50}$ for period T_i.

For subsequent use in the development of the input acceleration time-histories, it is necessary to perform hazard deaggregation analysis for the spectral accelerations (X_i) at selected periods (T_i) to determine the following information:

1. Deggregated mean earthquake moment magnitude (M), distance (R) and the ground motion parameter ε . The combination (M, R, ε) represents the "design earthquake event" for the spectral acceleration at the corresponding period.

2. The name and type of the predominant earthquake source associated with the mean or design earthquake event (M, R, ε)

Determine the above information for spectral accelerations for period T_i= 0.0 (i.e., PGA), $(T_n)_{soil}$ and 1.0 seconds. $(T_n)_{soil}$ is the natural period of the 1-D soil-column to be analyzed, which may be estimated by using Equation 3 (Sawada, 2004):

$$(T_n)_{Soil} = \frac{4H}{V_{s,avg}} = 4\left(\sum_{i=1}^{m} \frac{h_i}{V_{si}}\right) \text{ (seconds)} \quad (3)$$

Where,

H = Total thickness of the 1-D soil-column model $= \sum_{i=1}^{m} h_i$ (ft)

h_i = Thickness of the soil layer *i* (ft)

m = Total number of soil layers within the 1-D soil-column model.

$V_{s,avg}$ = Time-average shear wave velocity of the soil-column model

$$= \frac{\sum_{i=1}^{i=m} h_i}{\sum_{i=1}^{m} \frac{h_i}{V_{si}}}$$

V_{si} = Average seismic shear wave velocity of the layer *i* (ft/sec)

The three selected periods will be referred to hereafter as the periods of interest (T*).

The above source parameters are determined using the USGS Unified Hazard Tool or UHT (USGS, 2021a) and the "Interactive Fault Map" associated with the USGS 2014 NSHM (USGS, 2021b). UHT may be used to perform hazard deaggregation for the following specific structure periods, 0.0, 0.1, 0.2, 0.3, 0.5, 0.75, 1.0, 2.0, 3.0, 4.0 and 5.0 seconds. For other values of the period of interest (T*), use the nearest period from this list. See the Design ARS module for how to perform ground motion hazard deaggregation analysis using UHT to determine the mean earthquake parameters (M, R, ε) for a given period.

Results of the deaggregation analysis performed using the UHT tool includes a list of the seismic sources that contributes to the total hazard at the site. For a given period (T*), the seismic source contributing most to the hazard should be considered as the predominant seismic source. To determine the type of the predominant fault, refer to the USGS website at https://www.usgs.gov/natural-hazards/earthquake-hazards/faults (USGS, 2021b).

For the predominant background source, the focal mechanism or fault type may be assumed based on the predominant tectonic style of the region. The target design spectral acceleration for each of the above specified periods of interest (T*) may be associated with different predominant seismic sources.

7.2 Numbers of Input Motions

A total of at least eleven arbitrarily single-component horizontal acceleration time-histories selected from no less than seven different horizontal ground motion records and after modified by direct scaling to be compatible, on average, with the target design ARS should be used as the input motions to the base of the 1-D soil-column.

For each of the periods, T* = 0.0 and $(T_n)_{soil}$ seconds, at least two (2) single component horizontal motions, each from a different record should be used. These records should be selected from two different earthquake events. For T*=1.0 seconds, at least five single component motions, each from a different record, should be selected. These records should be selected from no less than three different earthquake events.

7.3 Selection of Initial Acceleration-Time Histories

For each period of interest (T*), enough horizontal ground motion records should be initially selected and examined for suitability based on the following guidelines and other requirements specified above and modified by scaling, so that the final evaluation results in the selection of at least the required minimum number of suitable modified single component horizontal input motions.

Records with no near-fault effects should be selected, unless adequate number of such records are not available. Initially, both horizontal components of a record should be examined. However, the one more suitable based on the characteristics discussed below should be selected for further consideration.

Recorded ground motion time-histories may be obtained from the Pacific Earthquake Engineering Center at https://ngawest2.berkeley.edu/ or other available ground motion record databases. Only properly processed ground motion records should be selected as initial or seed motions.

The peak ground acceleration, frequency content, and duration of the input base motions are key parameters. These parameters depend mainly on the earthquake magnitude, distance of the causative fault, and characteristics of the ground motion travel path to the top of basement rock and local soil conditions at the recording site. The initial acceleration-time histories can be selected from free-field records at the ground surface (i.e., outcropping motions) of the sites that are:

- Underlain by subsurface conditions that are like those underlying the base of the 1-D soil-column at the project site (i.e., without the 1-D soil column to be analyzed).
- Located within the same tectonic region as the project site (e.g., shallow crustal, subduction regimes). If adequate number of suitable records are not available, records from other similar tectonic regions may be used.
- Located at a distance equal or close to the deaggregated mean distance R, for the spectral acceleration at the specific period of interest (T^*), from the causative fault(s).

The selected initial time-histories for each specific period of interest (T^*) should be based on the following additional considerations and requirements:

- Generated by earthquakes of moment magnitudes same or close to the deaggregated mean moment magnitude (M).
- Generated by faults with similar source mechanism as the predominant fault.
- Durations and ground motion intensity representative of the design earthquake event (M, R, ε).
- Useable frequency that accommodate the range of frequencies important to non-linear response. The most critical is the lowest usable frequency. Only processed records with appropriate usable frequency range should be selected.

- Spectral shape similar to the target design spectrum near the period of interest (T*). This is an important requirement, based on which some of other requirements presented herein such as the distance may be relaxed. The spectral shape may be examined based on the ground motion parameter $\varepsilon(T^*)$ calculated for the selected horizontal ground motion component at period T*. See Stewart et al. (2014) for additional information.

- The scale factor limited to 0.25 to 4.0.

Appropriately selected recorded ground motions are used whenever available. For sites for which such records are not available in sufficient numbers, no more than two synthetic single horizontal component motions may be used for each period of interest.

7.4 Development of the Scaled Target Design Input Acceleration-Time Histories at the Base

Modifications to the initially selected acceleration time-histories are required to achieve compatibility with the ground motion intensities and the spectral shape represented by the target design ARS at each of the periods of interest (T*). Modifications should be performed by direct scaling only, which involves multiplying the acceleration values of a given record by a constant factor. This will result in an increase or decrease of its spectral accelerations by the same factor. Spectrum matching technique should not be used for a site-specific DGRA.

Any one of the two different direct scaling methods (Approach 1 and Approach 2) described in Stewart et al (2014) may be used to modify the initial time histories and obtain the required number of suitable target design input motions at the base. Approach 1 involves scaling a larger number of initially selected arbitrarily oriented, single components of horizontal ground motion or time-history records, and then identifying and selecting the required minimum number of motions (e.g., eleven) such that the average ARS of these scaled motions matches the target spectrum over the period range $0.2T_n$ to $2T_n$, where T_n is the structure natural period. The average ARS shall not be less than 90% of the target spectrum at any period with the above period range. As a default, the period range of 0.2 to 2.0 seconds may be used for SDC compliant bridges.

Approach 2 involves matching the target spectrum at a single period. In this approach, for each period of interest, the required number of ground motions are selected based on matching the spectral shape and then scaling so that their spectral acceleration at the selected period (T*) matches the target spectral acceleration at that period.

8.0 Site-Specific DGRA Methods and Tools

8. 1 DGRA Methods

A dynamic ground response analysis requires numerically modelling the 1-D soil profile as a continuum or discretized system, the cyclic shear stress-strain behavior of soils (material modelling) and the vertical propagation of seismic shear waves from the soil profile base at rock to the ground surface as discussed in Appendix A.

Two broad categories of analysis methods are commonly used for conducting a site-specific DGRA:

- Equivalent-Linear (EL) Method
- Non-Linear (NL) Method.

The EL method is a total stress type analysis. A NL method may be either a total stress type or effective stress type analysis.

In an EL analysis method:

- The 1-D soil column is modelled as a horizontally layered continuum.
- Dynamic shear stress-strain behaviors of soils are characterized by using simplified assumptions and specially formulated “equivalent-linear” soil material models. The equation of motion is solved in the frequency domain by assuming constant (i.e., linear) soil secant shear modulus (G) and damping ratio (D).
- A uniquely defined iterative solution scheme is used to incorporate the actual non-linear and spatial or time variations in the soil properties.
- It can be conducted only in terms of total stresses.
- This method cannot properly incorporate the inherent limits imposed by the soil shear strength on their dynamic responses.
- Due the above simplifying assumptions and limitations, this analysis method cannot properly incorporate either the highly non-linear or the effective stress-based soil behaviors during strong seismic shaking when the subsurface material profile consists of soft/weak soft soils, including liquefiable soils.
- The accuracy of this method, even for firm soils decreases as the mobilized shear strain increases and becomes unacceptable for medium to large shear strain (>0.1 to 0.4%)
- Furthermore, being a total stress method, EL analysis cannot incorporate the important effects of the generation of positive excess pore pressure, including liquefaction in contractive coarse-grained and fine-grained soil with little or no plasticity as well as the degradation of the cohesive fine-grained soils that occurs during seismic shaking. Therefore, an EL analysis is not considered appropriate for sites predicted to experience soil liquefaction.
- The EL method lacks flexibility for incorporating more advanced and up to date soil constitutive models.

In a NL method of analysis:

- Truly non-linear and hysteretic shear stress-strain behaviors of soils can be modelled and incorporated in this analysis.
- The analysis is performed in the time-domain permitting to consider explicitly temporal or time-dependent variations in the soil non-linear shear stress-strain behaviors that occur during seismic shaking.
- Ground response analysis can be performed based on total stress and/or effective stress, as appropriate and for any maximum strain levels, including the shear strain at shear failure, and

In summary, the above cited limitations associated with the EL analysis method can be mostly eliminated by selecting the appropriate NL analysis method, soil parameters and the constitutive modelling. However, significant knowledge on advanced soil properties and constitutive modelling, and familiarity with more complex computer codes available for NL analysis, and the skills for making some important discretional decisions and judging the soundness of the results obtained from the analysis are necessary for conducting a sound NL analysis that will result in reliable results.

Knowledge on the design ground motion development methodologies for Caltrans project site, and skills on analyzing and interpreting the results are necessary irrespective of the site-specific DGRA method of analysis used.

In practice, the shear stress-strain behaviors of soils, especially those weak, soft and liquefied soils for which a site-specific DGRA are generally required by Caltrans' SDC v2, are highly non-linear. Furthermore, due to relatively high target design ground motions (PGA$\geq$0.4g) at the soil column base (bedrock or firm-ground), most of these soil sites are likely to experience relatively large shear strains during the design ground motion event which are beyond the reliable shear strain range for an EL analysis. For liquefiable sites, the EL method is not appropriate since the effects of the generation of and temporal variations in the excess porewater pressures must be considered.

As a result, the applicability of the EL method to Caltrans bridge sites, for which a site-specific DGRA is required per SDC v2.0, is expected to be limited. Yet, this method is covered in detail in this module since many of the associated concepts form the basis for the NL analysis. *Furthermore, an EL analysis may be performed in conjunction with a NL analysis to compare the results.*

Additional details on both the EL and NL method of analyses, including material modelling, are presented below and in Appendix A and B. For more details, See Schnabel et al. (1972), Kramer (1996), Stewart et al. (2008), NCHRP (2012), and Stewart et al (2014).

8.2 Applicability of the DGRA Methods

Do not use the EL analysis to determine the design ARS at or near the ground surface when either of the following conditions apply. In these case, a NL is required.

1. The target base design PGA or the spectral acceleration at 1.0 second period≥0.3g
2. The soil profile at the site is classified as Type F

Based on the above conditions, a NL analysis will usually be required at most Caltrans bridge sites. Unless specified otherwise, a total stress-based NL analysis may be used except when the excess pore water pressure during ground shaking results in a significant reduction in the soil shear strength and stiffness.

An effective stress-based NL analysis must be performed when a site-specific DGRA is required due to liquefaction of soils that are characterized by $(N_1)_{60cs}$-S_r ≤20 or $(q_{c1N})_{cs\text{-}Sr}$ ≤130 defined in the Equations 4 and 5, respectively.

$$(N_1)_{60cs-Sr} = (N_1)_{60} + \Delta(N_1)_{60-Sr} \quad (4)$$

$$(q_{c1N)cs-Sr} = q_{c1N} + \Delta q_{c1N-Sr} \quad (5)$$

Here, $(N_1)_{60}$ or (q_{c1N}) are the corrected SPT blow count and CPT cone tip resistances, respectively, each normalized to 1.0 tsf effective overburden pressure. See the *Liquefaction Evaluation* module for how to calculate $(N_1)_{60}$ and q_{c1N} from field measured SPT blow count and CPT resistances, respectively. $\Delta(N_1)_{60-Sr}$ and Δq_{c1N-Sr} are the fines correction factors recommended by Seed (1987) and Idriss and Boulanger (2008), respectively, as presented in Table 1.

The non-linear effective stress analysis must incorporate the build-up of excess porewater pressure and its effects on the soil shear strength and stiffness

A total stress-based NL analysis can be used to conduct the required site-specific DGRA when all the soil layers predicted to experience initial liquefaction are characterized by $(N_1)_{60cs}$-S_r >20 or $(q_{c1N})_{cs\text{-}Sr}$ >130. In such cases, an EL analysis may be performed to determine the design ARS provided the target design PGA at the soil-column rock or firm-ground base is <0.3g.

Table 1. Recommended Fines Correction Factors

Fines Content (% Passing US Sieve No. 200)	$\Delta(N_1)_{60-Sr}$	Δq_{c1N-Sr}
10	1	10
25	2	25
50	4	45
75	5	55

8.3 Input Soil Properties and Material Models

Representative 1-D soil profile(s) for the bridge site or support(s), including the depth to the bedrock or the “firm-ground” soil-column base and the groundwater table, are required. The 1-D soil-column is then divided into N number of layers as shown in Figure A-1 in Appendix A.

The following soil properties and material models are generally required to define a soil layer for both EL and NL site-specific DGRA:

- Layer thickness (h)
- Total unit weight of soils (γ_t)
- The nominal shear resistance or shear strength (τ_{max}), which is equal to the stress on the failure plane at the failure shear strain (γ_f), and the
- Material constitutive relationships or models defining the cyclic shear stress-strain behaviors of the soils

The following presents a summary on the additional basic soil parameters and other information related to the most recently developed and recommended material models for defining the cyclic shear stress-strain behaviors of soils. Details on these material models and the associated model parameters are presented in Appendix A. Additional information, including how to determine the model parameters, is presented in the Appendix B and in the subsequent sections of this module.

In the EL analysis, the cyclic shear stress-strain behaviors for each soil layer are represented by: (a) a backbone curve as shown in Figure A-2(a) and, (b) damping characteristics.

1) The backbone curve is defined using the followings:
    - Soil small strain shear modulus (G_{max}) or the seismic shear wave velocity V_s (See Equation 1)
    - Variations of the normalized equivalent or secant shear modulus $with\ (\frac{G}{G_{max}})$ with the amplitude of the cyclic shear strain (γ) cycles as shown in the upper panel in Figure A-2(b) and the
    - Shear strength (τ_{max})
2) The damping characteristics are defined in terms of the:

- Variations of the equivalent-linear viscous damping ratio (D), defined in Figure A-2(a), as a function of the amplitude of the cyclic shear strain (γ) as shown in the lower panel in Figure A-2(b), and
- Small (or zero) strain damping ratio D_{min}.

In the NL analyses, cyclic non-linear behaviors of soils are represented by using a variety of models and techniques by different researchers. A cyclic non-linear model follows the actual shear stress-strain curve during cyclic loading.

The availability of the specific material models and techniques depends on the computer code used to perform the NL DGRA. Stewart et al. (2008) provides a summary of the different soil material models used in some of the commonly used NL computer codes. For additional details refers to Appendix A and the computer code selected for the NL analysis. In summary, they generally include:

1. A non-linear (τ-γ) or backbone curve as shown In Figure A-2(a). The following information are generally used or required to define a backbone curve:
    a. A hyperbolic model with specific curve fitting parameters. A hyperbolic model is generally used. See Appendix A for details on the various hyperbolic models, specifically those used in DEEPOIL v.7.
    b. Zero or small strain tangent shear modulus (G_{max}) or the seismic shear wave velocity V_s (See Eq. 1), and
    c. Shear strength (τ_{max})
2. A set of “Rules” that describes, in conjunction with the backbone curve, the cyclic unloading and reloading, and degradation behaviors of soils, e.g., the Masing and Extended Massing Rules (See Kramer 1996, Steward et al. 2008), or the Non-Masing Rules (see Hashash et al. 2020).
3. Zero (or small strain) viscous damping ratio or D_{min}.
4. Models and parameters for material (stiffness and shear strength) degradations and/or for excess pore pressure generation and dissipations (including the coefficient of consolidation, C_v).

8.4 Equivalent-Linear Analysis

The Equivalent-Linear (EL) analysis is a specific type of DGRA method the theory of which was originally proposed by Idriss and Seed (1968). This analysis method was first coded into the original SHAKE software by Schnabel et al. (1972).

8.4.1 Computer Codes

A computer code is necessary to perform a site-specific 1-D dynamic ground response analysis. SHAKE91 (Idriss and Sun, 1991), SHAKE2000 (GeoMotions, 2012), DEEPSOIL, v7.0 (Hashash et al., 2020), FLAC (Itasca, 2011) and PLAXIS (Bentley, 2021) are some of the commonly used computer software for EL analysis.

DEEPSOIL v7.0 has no limitations on the number of soil layers, material properties, or the length of input motions. It offers an option for automatic soil profile generation. DEEPSOIL also has the capability to performs both total and effective stress-based NL analyses. It has a robust convergence algorithm and includes some of the most recent material models and other ground response analysis related improvements for conducting a site-specific DGRA. When conducting an NL analysis, DEEPSOIL also offers an option to select for the software to run a complimentary EL analysis without any additional input.

8.4.2 Soil Material Model Parameters

The EL method defines two “Equivalent-Linear” soil parameters, the shear modulus (G) and the damping ratio (D), as discussed in Appendix A, to model the dynamic shear stress-strain behavior of soils during seismic shaking.

In the EL analysis method, the hysteretic damping shown in Figure A-2(a) in Appendix A is incorporated in the viscous damping ratio (D) in Equation A-1 and soil shear stiffness G is taken as equal to the secant shear modulus. Both G and D are defined in terms of the peak shear-stress at the end of the loading cycles,

In the EL analysis method, the equation of motion for the vertically propagating seismic shear waves is solved in the frequency domain. The soil material model parameters needed for each soil layer include:

1. Soil total unit weight (γ_t)
2. Soil shear modulus $G(\gamma)$
3. Soil damping ratio $D(\gamma)$, and
4. Soil complex shear modulus $G^*[\gamma, D(\gamma)]$

As discussed in Appendix A, both $G(\gamma)$ and $D(\gamma)$ are taken as loading frequency-independent parameters.

For a site-specific DGRA analysis, the parameters G and D may be determined by laboratory testing of the representative samples of the site soil or evaluated based on empirical correlations with other measured soil parameters such as those presented in Appendix A. These empirical correlations have been developed based on extensive laboratory testing.

For convenience, input information on the G and D parameters are specified in terms of normalized shear modulus ratio (G/G_{max}) and D versus shear strain curves. When developed based on an empirical correlation, these curves are often referred to as the "Reference" MR and DR curves, respectively.

8.4.3 EL Analysis Scheme

During an EL analysis run, which includes the development of the entire acceleration time history at the ground surface given an input acceleration time-history at the base of the 1-D soil column, the soil dynamic parameters G and D are assumed to be constants (i.e., time-invariants). That is, the values of these parameters do not change from those initially specified as the ground shaking progressed. This also means that the soil shear stress-strain curve is linear at the onset of motions and remains linear during the entire shaking period. No degradation in the soil stiffness or changes in the soil damping occurs during such an analysis run.

The above assumptions permit the analysis to be conducted in the frequency-domain by using easy to construct transfer functions. However, these are limiting assumptions for real soil in that the shear stress-strain behavior is non-linear and changes with time as the ground shaking progresses during an earthquake event.

A unique analysis scheme is employed in the EL method to overcome the above limitations in an attempt to incorporate approximately the soil non-linearity. This analysis uses an iterative procedure in which the soil acceleration- and shear strain-time histories are first evaluated by assigning certain estimated values for both G/G_{max} and D for the soil layer. It solves the equation of motion by representing the effects of the vertically propagating seismic shear waves from the base of the 1-D soil-column to the top for the these assigned constant G and D values. From the results, the peak shear strain of the soils within each layer is then determined. Based on this peak shear strain an "effective average shear strain" parameter (γ_{eff}) for the entire shaking period is calculated for each soil layer. Based on this calculated γ_{eff} shear-strain, compatible normalized G/G_{max} and D values are revaluated for each soil layer based on the respective specified MR and DR such as those shown in Figure A-2(b).

The analysis is then repeated utilizing the updated G and D values, and a new set of shear-strain compatible G and D are determined for each soil layer. This process is repeated until, for all the soil layer, the shear strain compatible G and D values at end of the analysis run converges to those specified at the beginning of the run within an acceptable small limit.

8.4.4 Modulus Reduction (MR) Curves

The normalized modulus reduction or MR curves such as that shown in Figure A-2(b), may be developed by laboratory testing of the representative site soil samples or determined based on empirical correlations with other site-specific soil parameters as discussed in Appendices A and B.

The recommended empirical models for the MR curves or the “Reference MR Curves” for soils are included in Appendix B. These empirical MR models may also be used for intermediate geomaterials or soft rock (V_s<760 m/sec.) as applicable based on their relevant engineering characteristics and stress conditions (e.g., cohesionless or cohesive, PI, effective overburden stress, etc.).

The recommended empirical MR curves are the most recent and were developed assuming a modified hyperbolic function (Eq. A-17 in Appendix A) for the shear stress-strain backbone curve shown in Figure A-2(a). The hyperbolic model used in the development of these reference MR curves is not dependable when the shear strains of $\gamma \geq$ 0.1 to 0.4%, depending on the soil types and conditions, are mobilized. The higher the mobilized maximum shear strain as a percentage of the shear failure strain, the lower is the reliability.

See Appendix A for additional information on the MR curves. Uncertainties in the MR curves should be considered in the site-specific DGRA. See Stewart (2014) for details.

8.4.5 Damping Ratio (DR) Curves

The damping ratio curves may be determined from the same laboratory tests performed on site soil samples to determine the MR curves or by using empirical correlations with other measured properties of the site soils, as discussed in Appendices A and B.

The recommended empirical models for the “reference’ soil damping ratio (DR) curves are presented in Appendix B. The following model is used for these “Reference DR” curves:

$$D(\gamma) = D_{min} + f\left(\frac{G(\gamma)}{G_{max}}\right) \qquad (6)$$

Where, D_{min} is the zero-shear strain damping ratio and the 2nd term is a function of the modulus reduction factor (G/G_{max}) at shear strain γ. Note that the 2nd term in Equation 6 represents the hysteretic damping ratio $D(\gamma)$ defined in Figure A-2(a). The 1st term or D_{min} represents soil damping which is always found to be present at small strains even when the soil exhibits very small or essentially zero hysteretic damping as defined in Figure A-2(a).

Uncertainties in the damping ratios should be considered in the site-specific DGRA. See Stewart et al (2014) for details.

8.4.6 Complex Shear Modulus (G*)

For mathematical simplicity, a complex shear modulus (G*) is defined (see Eq. A-6 in Appendix A) and used in the EL analysis. No additional input parameter specific to the complex shear modulus G*) is required since it is defined in terms of G and D. However, when performing EL analysis using DEEPSOIL v7.0, it offers options for three different complex shear modulus formulations. Of these options, the “Frequency Independent

Complex Shear Modulus" should be used. This is the same complex shear modulus formulation as used in SHAKE (See Schnabel et al., 1972).

8.4.7 Soil Shear Strength (τ_{max})

Soil shear strength will limit the magnitude of the high frequency ground motions, such as the peak ground acceleration, that can be transferred to the overlying soil layer(s). Low shear strength of soft soils and reduction in the shear strength and stiffness of soils due to liquefaction or cyclic degradation during a seismic event will result in an increase in the energy content of the low frequency ground motions. Non-linear effects are more pronounced in these cases. In terms of the ARS, such reduction will result in the spectral de-amplifications at short periods and higher amplifications at longer periods. The resulting higher spectral-amplifications at longer periods can have significant impacts on the seismic performances of bridges. It is thus important that these impacts on the design ARS be accurately incorporated into the site-specific DGRA.

Use of the V_s, soil density (ρ_t) and the MR curve imply a stress-strain curve of the soil with a maximum shear stress. The maximum shear stress at about at γ=10% is termed as the implied shear strength, $(\tau_{max})_{implied}$. The implied shear strength $(\tau_{max})_{implied}$ corresponding to the empirical MR curves can be significantly different than the actual shear strength (τ_{max}) of the soils operating during earthquakes, especially for soft and liquefied soils.

The EL analysis method is not capable of incorporating the above large strain or shear strength related impacts on the design ARS.

Due to generally high level of the design base motions at Caltrans project sites and weak/soft or liquefiable subsurface conditions for which a site-specific DGRA is required, soils in one or more layers are likely to experience large shear strains necessary to mobilize their nominal shear resistance or shear strength. The applicability of the EL analysis method to these project sites is thus very limited based on the criteria presented earlier.

The soil shear strength (τ_{max}) used is an important soil material model input parameter in the NL based DGRA. Soil shear strengths should be evaluated and specified, where necessary, by considering the nature of seismic loading, soil types and their behavior during seismic shaking. See Stewart et al. (2014) for additional guidance. Uncertainties in the soil shear strengths should be reduced by appropriate measures during field exploration and laboratory testing, evaluated per Stewart et al. (2014) and considered in the site-specific DGRA.

8.4.8 Effective Average Shear Strain (γ_{eff})

The effective average shear strain is used in the EL method to estimate the strain compatible soil shear modulus (G) and the damping factor (D) during each analysis run. The ratio (R_f) defined, as follows, is used to calculate the effective average shear strain (γ_{eff}) from calculated maximum shear strain (γ_{max}) for each soil layer:

$$R_f = \frac{\gamma_{eff}}{\gamma_{max}} \tag{7}$$

The recommended value for R_f in Equation 7 is 0.65.

8.5 Non-Linear Analysis

In a non-linear (NL) analysis, the shear stress-strain relationships of soils are modelled and incorporated in the ground response analysis in the manners that more closely represent the true non-linear cyclic behaviors of soil in the field when subjected vertically propagating seismic shear waves during an earthquake event.

A NL DGRA may be based on either total stress or effective stress. In such an analysis, the equation of motion for the vertically propagation seismic shear wave is solved in the time-domain. It can explicitly incorporate the following important aspects of soil behaviors:

- Non-linear inelastic shear-stress-strain behavior of soils
- Cyclic or hysteretic shear loading-unloading-reloading behavior,
- The generation and dissipation of excess pore pressure in soils, including soil liquefaction, and their effects (e.g., degradation) on the soil shear stress-strain behaviors as function time during shaking, and
- The soil shear strength (τ_{max}), which imposes an upper limit on the shear stress (demand) than a given soil layer can sustain when subjected to strong shaking and, thus, limits the ground motions transmitted to the adjacent soil layers.

The NL analysis methods can incorporate more advanced constitutive models, including complex soil behaviors such as yielding, flow rules and hardening/softening rules.

Thus, a properly conducted NL analysis method can eliminate many of the significant approximations or limitations involved with the EL method. This extends the applicability to and use of site-specific DGRA for the more representative and reliable ground motion evaluation for project sites underlain by soil profiles that include soft/weak and/or liquefiable soil layers.

However, a NL analysis is more complex requiring significant time and expertise. Until recently, methods and tools (e.g., computer software) for conducting a NL analysis, particularly the effective-stress method, were limited. Technology and computer software (e.g., DEEPSOIL) have advanced significantly during the recent years such that when the parameters are carefully selected, consistent and reliable results can be obtained from NL based site-specific DGRA, including the effective stress method.

8.5.1 Computer Codes

DEEPSOIL (Hashash et al., 2020), D-MOD2000 (GeoMotions, 2017), FLAC (Itasca, 2011) and PLAXIS (Bentley, 2021) are some of computer software that may be used for conducting a site-specific non-linear dynamic ground response analysis.

Stewart et al. (2008) evaluated several commonly used computer codes for 1-D NL analysis methods. These includes MOD_2 (Matasovic, 2006), DEEPSOIL (Hashash and Park, 2001; 2002), OpenSees (McKenna and Fenves, 2001), TESS (Pike, 2000) SUMDES (Li et al., 1992) and DESRA-2 (Lee and Finn, 1978). It was found that when the key controlling parameters were properly identified and the input was appropriately controlled, most of these computer codes provided similar results. However, use of these software to conduct a NL DGRA requires significant expertise and knowledge.

The most recent version of the computer codes should be used for Caltrans bridge project sites. Also, if necessary, consult with a specialist on the subject matter.

8.5.2 Soil Profile Domain Discretization

In a NL analysis, the equation of motion for vertically propagating seismic shear waves is solved in the time-domain. Dynamic ground or soil responses are obtained by formulating and solving a system of coupled equations of ground motions. The equation of motion is discretized temporally (i.e., in the time domain). A time-stepping integration scheme such as the Newmark's β implicit method (Newmark, 1959) or Heun's explicit method is used to obtain the solution.

For conducting a NL analysis, some computer software (e.g., DEEPSOIL, D-MOD2000, TESS, SUMDES and DESRA-2) model and analyze the 1-D soil-column domain as a multiple-degrees-of-freedom (MDOF) system using discretized lumped-mass-stiffness method. To evaluate dynamic ground responses in the time-domain, the dynamic equation of motion representing vertically propagating seismic shear wave is written in the following form:

$$[M][\ddot{u}] + [C][\dot{u}] + [K][u] = -[M][\ddot{a}_g(\mathrm{t})] \qquad (8)$$

In Equation 8, [M], [C] and [K] are the soil lumped mass matrix, viscous damping matrix, and the non-linear shear stiffness matrix, respectively; [u], [$\dot{u}$] and [$\ddot{u}$] are the soil displacements, velocities and accelerations, respectively, of the soil mass [M] at depth (x) and time (t), relative to the base (i.e., top of the rock or firm-ground half-space), and [$\ddot{a}_g$(t)] is the input or target acceleration at the base. Other computer codes model the soil profile as a continuum discretized into finite elements with nodes connecting the elements. FLAC and TESS use an explicit finite-difference scheme while OpenSees and PLAXIS use finite element method of analysis.

8.5.3 Soil Material Models

A NL based site-specific DGRA includes the following main soil material model input parameters.

- The non-linear shear stress-strain (τ-γ) or backbone curve.
- Rules for constructing unloading-reloading cycles or the hysteretic loops.
- Zero-strain soil damping, and
- For effective stress-based NL analysis, models for the generation, redistribution, and dissipation of excess porewater pressure during ground shaking.

A mathematical form of the backbone curve is required for NL analysis since the equation of motion is solved in the time domain by stepwise integration method which uses the instantaneous tangent modulus (G) of the soil as shown in Figure A-2.

Soil hysteretic damping at strain levels greater than $10^{-4 \text{ to }} 10^{-2}$ % is implicitly incorporated by following the loading-unloading-reloading paths (hysteretic cycles/loops) when solving the equation of motion in small time-step increments. That is for such strain levels, the term [C] is not necessary in Equation 8. However, due to the manner the backbone curve is specified, hysteretic damping that can be incorporated implicitly is effectively zero for strain level levels <($10^{-4 \text{ to }} 10^{-2}$ %). Real soil, however, exhibit some damping event at such very small strain level. It is thus necessary to include additional damping into the NL analysis, which is done by keeping the viscous damping matrix [C] in Equation 8 but forming it with zero-strain damping.

Various models, ranging from simple to advance in scope and complexities, are available for the NL backbone curve. The availability of the specific models for analysis are dependent on the computer software selected for the site-specific DGRA. The following is a summary of the main features of the some of the soil material models commonly used in the NL analysis software, e.g., DEEPSOIL and D-MOD2000.

8.5.4 Modelling Non-Linear Backbone (τ-γ) Curves

The backbone curve for NL analysis may range from very simple to complex hyperbolic models. The simple Konder and Zelasko (1963) or KZ model and the Modified KZ model discussed under EL analysis in Appendix A may be also used in an NL analysis. A few additional more advanced models used in the NL analysis are briefly discussed in Appendix A.

8.5.5 Modelling Soil Damping

Soil damping in a NL analysis often consists of two types, namely (1) Hysteretic (or Material) Damping and (2) Small Strain Viscous Damping

8.5.5.1 Hysteretic (Material) Damping Ratio

In most NL analysis, soil hysteretic damping is captured by incorporating the hysteretic loops into the time-stepping integration scheme used to solve the equation of motion. In that case, the viscous damping matrix [C] term in Equation 8 is not necessary, except for very small strain as discussed below. The amount of hysteretic damping incorporated into the time-domain solution depends on the backbone curve as well as the unloading-reloading rules used to model the hysteretic stress-strain loops.

8.5.5.2 Small Strain/Viscous Damping

At small shear strain levels ($<10^{-4}$ to 10^{-2} %), the hyperbolic backbone curve is linear, which results in the zero hysteretic damping (i.e., D=0.0). Real soil, however, exhibits some amount of damping even at very small shear strain. Therefore, some small strain viscous damping must be used in a NL analysis in addition to the implicitly used hysteretic damping. The is achieved by specifying a minimum damping (D_{min}). When using Equation 8, this is done by retaining the viscous damping matrix [C] (Hashash and Park, 2001). The matrix [C] is formulated using the small strain damping factor (D_{min}). See Appendix A for additional discussion on small strain viscous damping

In the NL analysis for Caltrans bridge sites, the small strain or zero-strain viscous damping should be considered as loading frequency independent.

In DEEPSOIL, the small strain or zero-strain viscous damping (ξ_0) should be taken as equal to D_{min} in Equation 6.

D-MOD2000 also uses a constant (and independent of the confining pressure) small strain viscous damping factor with a recommended upper value of 1.5% to 4%.

8.5.6 Modelling Hysteretic/Unloading-Reloading Behavior

The hysteretic or unloading-reloading behavior of soil during ground shaking is incorporated in the NL analysis by using Masing Rules (Masing, 1926) and the Extended Masing Rules (Vucetic, 1990). These are rules which, in connection with the specified backbone curve, describe the soil shear-strain unloading-reloading cycles. See Appendix A for additional discussion.

8.5.7 Effective Stress- Based NL Analysis

Models for the generation and dissipation of the excess pore pressures generated in the soils are required to conduct a site-specific DGRA using NL effective stress-based method. Such an analysis may be conducted for any site. It is required for Caltrans bridge project sites where soil liquefaction is predicted to occur, unless exempted based on conditions specified earlier in this module.

8.5.8 Models for Excess Porewater Pressure Generation

Models for the generation and dissipation of the excess pore pressures generated in the soils are required to conduct a site-specific DGRA using NL effective stress-based method. Such an analysis may be conducted for any site. It is required for Caltrans bridge project sites where soil liquefaction is predicted to occur, unless exempted per requirements documented earlier in this module.

See Appendix A for additional information on the models for excess porewater pressure generation.

8.5.9 Shear Modulus and Resistance Degradations

Matasovic and Vucetic (1993) recommended two degradation index parameters (δ_G and δ_τ) to model excess porewater induced degradation of the strength and stiffness of liquefiable soils for use in conjunction with the Matasovic-KZ (MKZ) model for the backbone curve discussed earlier. In this approach, the initial shear modulus (G^*_{mo}) and the shear resistance (τ^*_{mo}) are updated by multiplying by the degradation indices δ_G and δ_τ , respectively, which are evaluated based on the generated excess porewater pressure ratio. The MKZ backbone curve given by Equation A-18 is continuously updated with time. The continuously updated backbone curve is used to develop the hysteretic loop based on the selected un/reloading rules. This results in the continuous updating of the hysteretic damping ratio.

These degradation parameters at any time t are determined using Equations 9 and 10 as follows:

$$\delta_G = \sqrt{1 - u^*} \tag{9}$$

$$\delta_\tau = \sqrt{1 - (u^*)^\nu} \tag{10}$$

Where, δ_G = Shear modulus degradation index

δ_τ = Shear resistance degradation index τ

u^* = Normalized residual excess porewater pressure = ($\Delta u/\sigma'_{vo}$)

Δu = Residual excess porewater pressure at any time (t)

ν = Curve fitting parameter to better model the degradation of the normalized shear resistance (τ^*_{mo}).

The excess porewater pressure generation model is used to predict excess porewater pressure Δu at any time t.

Based on Matasovic and Vucetic (1993), for the three different types of sands tested to verify this concept, the values of the curve fitting parameter ranged from 3.5 to 5.0. These degradation parameters are also implemented within the Generalized Quadratic/Hyperbolic (GQ/H) model included in DEEPSOIL.

DEEPSOIL offer the option of using the above formulations for the degradation parameter with any of the available excess porewater pressure generation models, except the Matasovic and Vucetic (1995) model for which the degradation parameters are determined as follows:

$$\delta_G=\delta_\tau=N^{-1} \quad (11)$$

In Equation 11, N is the number of equivalent shear loading cycles after which the degradation parameters are evaluated.

8.5.10 Excess Porewater Pressure Dissipation

The excess porewater dissipation may be modelled based on the Terzaghi's classic 1-D consolidation theory. In a I-D soil profile used for the site-specific DGRA, dissipation of the excess porewater pressure can occur only in the vertical direction. It may be assumed that that the generation and dissipation of the excess porewater pressure occur simultaneously.

For each soil layer, the coefficient of consolidation (C_V) or permeability (k_v) is required as one of the input parameters for modelling excess pore pressure dissipation. Dissipation is generally assumed to occur in the vertical direction.

9.0 Results of a Site-Specific DGRA

Results obtained from a site-specific DGRA should include, for each input time-history, plots of: maximum or peak; (1) horizontal ground accelerations (PHGA), (2) Shear stresses, (3) Shear-strains, and (4) maximum mobilize shear strength or friction angle as a function of depth (and/or elevation) from the ground surface.

For each input-time history, results at the ground surface (or at the depth or elevation at which the design ARS is required) should include plots of total acceleration, shear stress ratio (τ/σ'_{vo}) and shear strain (γ) time-histories, and the cyclic stress-strain curve (τ-γ).

In case of soil liquefaction, these plots plus the plot of the excess porewater pressure ratio ($\Delta u/\sigma'_{vo}$) time-history should be obtained for the liquefied soil layer(s). In addition, the shear stress-strain curves should be plotted

This above information should be examined carefully to verify the reasonableness of the analysis performed and the resulting modifications in ground motions (e.g., spectral amplifications.

The presence of soft soil at site generally results in the amplification of the long period ground motions as shown in Figure B-2. Low intensity short period motions at relatively thick soft soil sites generally experience amplification, as shown Figure B-3 for the PGA. Refer to Seed et al. (1974) and Idriss (1990) for additional information.

Long period ground motions at soil sites where liquefaction resulting in reduction in the soil shear stiffness and strength occurs generally experience significant amplifications. Short period (e.g., less than 0.5 second) ground motions may de-amplify at some soil sites that experiences soil liquefaction during the seismic event. Refer to Youd and Carter (2005) and Gingerly et al (2015) for additional information.

Figure B-2 also shows that long period ground motions may also experience significant amplifications at deep (>230 feet) cohesionless soil sites, which is not incorporated in the current GMMs. Thus, for such deep soils sites, a site-specific DGRA may be required depending on the bridge seismic response characteristics and as determined by the Structure Designer.

The above summary on the types of ground modifications that have been identified in the literature based on limited actual records and analytical studies. The analytical studies are subjected to many limitations. However, soil profiles vary from site to site, and thus site-specific seismic soil responses may be significantly different from one site to another even when subjected to the same bedrock or base motions, and from those represented in the above summary. Furthermore, the definition for the soil liquefaction used in the literature also vary significantly (see Olsen et al., 2020). The analyst is responsible for verifying the reasonableness of the results of a site-specific DGRA considering information presented herein, the cited original references and other credible and up to date information that may be found in the publicly and readily available reputable professional and government publications, national or international.

9.1 Basic Design ARS at the Ground Surface

Results of a site-specific DGRA include an output acceleration time-history at the ground surface for each input horizontal ground motion component applied at the base.

For each selected target input base motion:

- Develop its ARS at the base level and the corresponding spectral accelerations $X(T_i)$, where i=1, N. N is the number of discrete single degree of freedom oscillators and T_i is the period for the oscillator i.
- Develop the ARS at the ground surface based on the corresponding output acceleration time-history at the ground surface by calculating spectral accelerations $Z(T_i)$, i=1, N.
- Next calculate the soil amplification factors $Y(T_i)$, i=1, N, at the ground surface using Equation 12 as follows:

$$Y(T_i) = \frac{Z(T_i)}{X(T_i)},\ \text{i =1, N} \qquad (12)$$

Once $Y(T_i)$ values for all of the selected target input base motions are known, calculate the mean site amplification factors $Y_m(T_i)$ for each period T_i (i=1, N) based on the amplifications factors $[Y(T_i)]_j$, where j=i, K, where K is the total number of input target base motions used in the analysis per Equation 13, as follows:

$$Y_m(T_i) = \frac{\sum_{j=1}^{K}[Y(T_i)]_j}{K},\ \text{i=1, N} \qquad (13)$$

The above calculated mean amplification factors $Y_m(T_i)$ for each period T_i (i=1, N) are then used to determine the spectral accelerations $Z_m(T_i)$ at the ground surface as follows:

$$Z_m(T_i) = X(T_i) \; x \; Y_m(T_i), \text{ i=1, N} \tag{14}$$

Develop a smoothed mean ARS at the ground surface based on the N data points $[\,Z_m(T_i), Ti]$, $i = 1, \mathrm{N}$ obtained using Equation 14.

This smoothed ARS developed ARS at the ground surface thus developed is defined in terms of the orientation-independent, rotated mean spectral intensity parameter $(S_a)_{ROTD50}$ as defined by Boore et al (2010) That is, $Z_m(T_i) = (S_a)_{ROTD50}$ at period (T_i). It is the *basic design ARS* for the site and is randomly-oriented respect to both the bridge structure and the contributing faults.

9.2 Design ARS at the Ground Surface from Site Specific DGRA

The *basic design ARS* developed above is also the "*Design ARS"* corresponding to the predicted design motions at the ground surface when the bridge site is not subjected to near-fault and/or basin-effects based on the provisions included in Appendix B of the SDC v.2. For sites subjected to one or both of these effects, additional amplification factors due to near-fault and/or basin-effects as specified in Appendix B of the SDC v2.0 must be applied to the above smoothed basic design ARS to determine the "Design ARS" corresponding to the design ground surface motions.

Refer to the procedures for the development of the design ARS for FEE in the *Design ARS* module for how to determine if a bridge site is subjected to these effects, and also, when required, for how to determine the applicable amplification factors per Appendix B of the SDC v2.0 using the Caltrans current *ARS Online* v3 tool.

Using the ARS Online v3 tool, the amplification factors due to near-fault effects is determined based on deaggregated mean site-to-source (R) for the 1.0-second period spectral acceleration for the target design ARS at the base. The basin-effects is determined based on the site coordinates.

For bridge sites subjected to near-fault and/or basin-effects, the "*Design ARS"* corresponding to the ground surface motion is obtained by multiplying the above developed *basic design ARS* by the applicable amplification factors.

10. Reporting

Prepare a Foundation Report and include the following information/discussion in the ground motion section.

- A discussion on the need for a site-specific DGRA at the site.
- A summary of the communications with the SD regarding the site-specific DGRA. Include all relevant structure analysis and design information or requirements provided by or obtained from the SD.
- Discussion on the site exploration, including field and laboratory testing, performed specifically for the site-specific DGRA and the results obtained

- A schematic of the representative 1-D soil-column model(s) used in the analysis. Include all relevant soil/rock information, e.g., layering, types, descriptions, groundwater conditions, model base elevation and material input properties pertinent to the site-specific DGRA.
- A brief discussion on the development of the 1-D soil-column(s), including the selection of its base.
- Plot(s) of the seismic shear wave velocity as a function of elevation and depth from the ground surface.
- A discussion on the basis for the selection of the site-specific DGRA method, the computer software, the constitutive models and the analysis or constitutive model-specific advanced material properties, including MR and DR data or curves, if used. Include plots of the shear stress-strain relationships and the various modelling or curve-fitting parameters used.
- Discussion on the development of the target design ARS at the base. Include the target design ARS at the base of the soil-column.
- Discussion on the selection and scaling of the outcropping seed motions to obtain the required number of target design input motions at the base. Present the main characteristics of the selected seed motions such as those included in Table C-1 in Appendix C, of the selected initial or seed motions corresponding to the target design input motions at the base.
- Include plots of the acceleration, velocity and displacement time-histories, and the ARS curve for each of these initial or seed motions.
- Include on a separate figure plots of the ARS for each of the scaled target design input motion at the base, the median ARS (per Approach 1 scaling option in Section 7.4) and the target design ARS. If the target design motions were scaled based on Approach 2 in Section 7.4, present this information for each matching period of interest (T*) on a separate figure.
- Plots of the peak ground acceleration, the mobilized peak shear stress, and the peak shear strain as a function of depth for each target input motion, and
- For the surface soil layer, and the liquefied soil layers, if any, plots of the times histories for acceleration, velocity, displacement, shear stress ratio (τ/σ'_{vo}), and shear strain (γ).
- For NL analysis, the hysteretic shear stress-strain plots for selected weak/soft soil layers (e.g., S_u <1000 psf) and liquefied soil layers.
- The excess pore pressure ratio ($\Delta u/\sigma'_{vo}$) time-histories for the liquefied soil layers.
- The required design ground motion parameters per the *Foundation Reports for Bridges* module, including the design ARS, developed based on the site-specific DGRA. Use the appropriate design ground motion presentation template from the Design ARS module.

- For liquefied soil sites, also include the design ground information, including the design ARS at the ground surface, developed by assuming non-liquefied site conditions.
- Provide any additional information considered relevant to the analysis conducted and the results obtained, or if specifically requested by SD.

REFERENCES (INCLUDING REFERENCES CITED IN THE APPENDICES)

1. Bentley (2021), PLAXIS 2D Ultimate with Geotechnical SELECT Entitlements, Bentley Systems Incorporated, Exton, PA.
2. Berrill, J.B. & Davis, R.O. (1985), Energy Dissipation and Seismic Liquefaction of Sands: Revised Model, Soils and Foundations 25(2), pp. 106-118.
3. Boore, D. M. (2010). Orientation-Independent Nongeometric-Mean Measure of Seismic Intensity from Two Horizontal Components Motion, Bulletin of the Seismological Society of America, Vol. 100, No. 4, pp, 1830-1835, August.
4. Caltrans (2019a), Seismic Design Criteria, Version 2.0, with October 2019 Interim Revisions (SDC, v2.0).
5. Caltrans (2019b). AASHTO LRFD Bridge Design Specifications, Eighth Edition with California Amendments (AASHTO-CA BDS-8).
6. Caltrans (2020a). *ARS Online*, Version 3, California Dept. of Transportation.
7. Caltrans (2020b). *Liquefaction Evaluation*, Geotechnical Manual, Geotechnical Services, California Dept. of Transportation.
8. Caltrans (2020c). *Geotechnical Investigations*, Geotechnical Manual, Geotechnical Services, California Dept. of Transportation.
9. Caltrans (2021a). *Design Acceleration Response Spectrum*, Geotechnical Manual, Geotechnical Services, California Dept. of Transportation
10. Caltrans (2021b). *Foundation Reports for Bridges*, Geotechnical Manual, Geotechnical Services, California Dept. of Transportation
11. Choi W.K. (2008). Dynamic Properties of Ash-Flow Tuffs, PhD Thesis, Department of Civil Engineering, University of Texas, Austin, TX.
12. Darendeli, M. B. (2001). Development of a New Family of Normalized Modulus Reduction and Material Damping Curves, Department of Civil, Architectural and Environmental Engineering, The University of Texas, Austin, TX.
13. FHWA (2012). LRFD Seismic Analysis and Design of Transportation Geotechnical Features and Structural Foundations, Report No. FHWA-NHI-11-032. February.
14. FHWA (2017), GEC No. 5, Geotechnical Site Characterizations, Publication No. FHWA-16-072, April.
15. GeoMotions (2012), SHAKE2000, A Computer Program for the 1-D Analysis of Geotechnical Earthquake Engineering Problems, Lacey, Washington.
16. GeoMotions (2017), “D-MOD2000: A Nonlinear Computer Program for Seismic Response Analysis of Horizontally Layered Soil Deposits, Earthfill Dams, and Solid Waste Landfills,” Lacey, Washington.
17. Gingery, J. R., Elgamal, A., and Bray, J. D. (2014), Response Spectra at Liquefaction Sites During Shallow Crustal Earthquakes, Earthquake Spectra 31(4.
18. Green, R.A., Mitchell, J.K. and Polito, C.P. (2000), "An Energy-Based Pore Pressure Generation Model for Cohesionless Soils", Proceedings: John Booker Memorial Symposium, Melbourne, Australia, November 16-17.
19. Groholski, D. R., Hashash, Y. M. A., Kim, B., Musgrove, M., Harmon, J. and Steward, J. P. (2016), A Simplified Model for Small-Strain Nonlinearity and Strength in 1D Seismic Site Response Analysis, J. Geotech. Geoenviron. Eng., 142(9): 04016042.

20. Hardin, B.O and Drnevich, J. P. (1972), Shear Modulus and Damping in Soils: Design Equations and Curves, ASCE Journal of Soil Mechanics and Foundation Division, 98(7), pp. 667-692.
21. Hashash Y.M.A., Park D. (2001), Non-linear One-dimensional Seismic Ground Motion Propagation in the Mississippi Embayment, Eng. Geol., 62: 185206.
22. Hashash Y.M.A., Park D. (2002), Viscous damping formulation and high frequency motion propagation in nonlinear site response analysis, Soil Dynamics and. Earthquake Engineering, 22(7), pp. 611-624.
23. Hashash, Y. M. A., Phillips, C., and Groholski, D. R. (2010). “Recent Advances in Non-linear Site Response Analysis.” 5th Int. Conf. in Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, Missouri Univ. of Science and Technology, Rolla, MO.
24. Hashash, Y.M.A., Musgrove, M.I., Harmon, J.A., Ilhan, O., Xing, G., Numanoglu, O., Groholski, D.R., Phillips, C.A., and Park, D. (2020) “DEEPSOIL 7.0, User Manual”, Board of Trustees of Univ. of Illinois at Urbana-Champaign, IL.
25. Idriss, I. M. and Seed, H.B. (1968), Seismic Response of Horizontal Layers, Jnl. of the Soil Mechanics and Foundation Division, ASCE, Vol. 94, No. SM4, July.
26. Idriss, I. M. and J. I. Sun (1992). “User’s Manual for SHAKE91, A Computer Program for Conducting Equivalent Linear Seismic Response Analyses of Horizontally Layered Soil Deposits,” Center for Geotechnical Modelling, Dept. of Civil & Environmental Engineering, Uni. of California Davis, CA.
27. Idriss, I. M. and Boulanger, R.W. (2008). Soil Liquefaction During Earthquakes, MNO-12, Earthquake Engineering Research institute, Berkeley, CA.
28. Idriss, I.M. (1990). Response of soft soil sites during earthquakes, Proc. H. Bolton Seed Memorial Symposium, J. M. Duncan (editor), Vol. 2, 273-290.
29. Itasca, Inc. (2011), FLAC, Fast Lagrangian Analysis of Continua, Minneapolis, MN.
30. Kempton, J. J. and Stewart, J. P. (2006), Prediction Equations for Significant Duration of Earthquakes Ground Motions Considering Site and Near-Source Effects, Earthquake Spectra, Volume 22, No. 4, pp. 985-1013.
31. Konder, R. L. and Zelasko, J. S. (1963). A hyperbolic stress-strain formulation of sands, Proceedings of the 2nd Pan American Conference on Soil Mechanics and Foundation Engineering, Sao Paulo, Brazil, 289-324.Kramer, S. L. (1996). Geotechnical Earthquake Engineering, Prentice-Hall, Inc.
32. Kramer, S. L. (1996), Geotechnical Earthquake Engineering, Prentice Hall, NJ.
33. Lee, M. K. W. and W. D. L. Finn. (1978). “DESRA-2: Dynamic Effective Stress Response Analysis of Soil Deposits with Energy Transmitting Boundary Including Assessment of Liquefaction Potential,” Soil Mechanics Series 36, Dept. of Civil Eng., Univ. of British Columbia, Vancouver, Canada, 60 p
34. Li X.-S., Wang Z.-L., Shen C-K. (1992). SUMDES: A Nonlinear Procedure for Response Analysis of Horizontally- Layered Sites Subjected to Multi-Directional Earthquake Loading, Dept. of Civil Engineering, Univ. of California, Davis, CA.
35. Masing, G. (1926). Eigenspannungen und Verfestigung beim Messing, Proc. of the 2nd International Congress on Applied Mechanics, Zürich, Switzerland.
36. Matasovic. N. (1993). "Seismic response of composite soil deposits." PhD dissertation. University of California. Los Angeles. Calif

37. Matasovic, N. and M. Vucetic, (1993). "Cyclic Characterization of Liquefiable Sands," Jnl. of Geotechnical Engineering, Vol. 119, No. 11, pp. 1805–1822
38. Matasovic, Neven, and M. Vucetic (1995) "Generalized Cyclic Degradation Pore Pressure Generation Model for Clays," ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 121, No. 1, pp 33-42.
39. Matasovic N. (2006). D-MOD_2: A Computer Program for Seismic Response Analysis of Horizontally Layered Soil Deposits, Earthfill Dams, and Solid Waste Landfills, User's Manual, GeoMotions, LLC, Lacey, Washington.
40. McKenna F., Fenves G. L. (2001). The OpenSees Command Language Manual, Version 1.2., Pacific Earthquake Engineering Research Center, Berkeley, CA.
41. Menq, Farn-Yuh (2003). Dynamic Properties of Sandy and Gravelly Soils, Department of Civil, Architectural and Environmental Engineering, The University of Texas, Austin, Texas.
42. National Academies of Sciences, Engineering, and Medicine 2019. Manual on Subsurface Investigations. Washington, DC: The National Academies Press. https://doi.org/10.17226/25379.
43. NEHRP (2020), NEHRP Recommended Seismic Provisions for New Buildings and Other Structures, Volume 1: Part 1 Provisions, Part 2 Commentary, FEMA P-2082-1/September.
44. Newmark, N. M., "A Method of Computation for Structural Dynamics," Journal of Engineering Mechanics Division, Vol. 85, 1959, pp. 67–94.
45. NCHRP (2012), Practices and Procedure for Site-Specific Evaluations of Earthquake Ground Motions, NCHRP Synthesis 428, Transportation Research Board, Washington, D.C
46. Olson, S. M., Mei, X., and Hashash Y. M. A. (2020), Nonlinear Site Response Analysis with Pore-Water Pressure Generation for Liquefaction Triggering Evaluation, Journal of Geotechnical Engineering, 146(2), 04019128.
47. Park D., J.-K. Ahn. (2013), "Accumulated Stress Based Model for Prediction of Residual Pore Pressure," Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris, pp 1567-1570.
48. Phillips, Camilo, and Youssef M. A. Hashash (2008) "A simplified constitutive model to simultaneously match modulus reduction and damping soil curves for nonlinear site response analysis," Geotechnical Earthquake Engineering & Soil Dynamics IV (GEESD IV). Sacramento, California.
49. Pyke, R. M. (2000). TESS: A computer program for nonlinear ground response analyses. TAGA Engineering Systems & Software, Lafayette, Calif.
50. Rayleigh, J. and Lindsay, R. (1945) The Theory of Sound. Dover Publications, Inc.
51. Roblee, Cliff and Chiou, Brian. (2004). "A proposed geoindex model for design selection of non-linear properties for site response analyses." Caltrans Geo-Research Group. Sacramento, CA.
52. Rodriguez-Marek, A. and Bray, J.D. (2006). Seismic Site Response for Near-Fault Forward Directivity Ground Motions, Journal of Geotechnical and Geoenviornmental Engineering, Vol. 132, No. 12, December, pp.1611-1620.
53. Sawada, S. (2004), A Simplified Equation to Approximate Natural Period of Layered Ground on the Elastic Bedrock for Seismic Design of Structures, 13th World Conference on Earthquake Engineering, Vancouver, B.C., Canada.

54. Seed, H. B., Ugas, C. and Lysmer, J. (1974). Site Dependent Spectra for Earthquake-Resistant Design, Report No. EERC 74 12, College of Engineering. University of California, Berkeley, California.
55. Seed, H. B. (1987), Design Problems in Soil Liquefaction, Journal of Geotechnical Engineering, ASCE, 113(8), 827-45.
56. Schnabel, P. B., Lysmer, J., and Seed, H. B. (1972), "SHAKE: A Computer Program for Earthquake Response Analysis of Horizontally Layered Sites", Report No. UCB/EERC-72/12, EERC, Univ. of California, Berkeley, California.
57. Stewart et al. (2008), Benchmarking of Nonlinear Geotechnical Ground Response Analysis Procedures, PEER Report 2008/04, Univ. of California, Berkeley.
58. Stewart et al. (2014), Guidelines for Performing Hazard-Consistent One-Dimensional Ground Response Analysis for Ground Motion Prediction, PEER Report 2014/16, Pacific Earthquake Engineering Research Center, CA.
59. Towhata, I. (2008), Geotechnical Earthquake Engineering, Springer-Verlag Berlin Heidelberg, Germany.
60. USGS (2021a). Unified Hazard Tool or UHT (Accessed at https://earthquake.usgs.gov/hazards/interactive/)
61. USGS (2021b), Earthquake Hazards: Faults - Interactive Fault Map, Accessed on June 2, 2021at https://www.usgs.gov/natural-hazards/earthquake-hazards/faults, United States Geological Survey.
62. Vucetic, M., and Dobry, R. (1986). “Pore pressure build-up and liquefaction at level sandy sites during earthquakes.” Research Rep. CE-86-3, Dept. of Civil Engineering, Rensselaer Polytechnic Institute, Troy, NY.
63. Vucetic, M. (1990), “Normalized Behavior of Clay under Irregular Cyclic Loading,” *Canadian Geotechnical Journal*, Vol. 27, pp. 29–46.
64. Youd, T. L. and Carter, B. L. (2005). Influence of Soil Softening and Liquefaction on Spectral Acceleration, ASCE Jnl. of Geotechnical and Geoenvironmental Engineering, 131(7), pp. 811-825.
65. Zhang J., Andrus R.D., Juang C.H. (2005). Normalized shear modulus and material damping ratio relationships, ASCE, *J. Geotech. Eng.*, 131(4), pp. 453-464.

APPENDIX A

1-D SEISMIC SHEAR WAVE PROPAGATION AND SOIL MATERIAL MODELS

A-1. ONE-DIMENSIONAL (1-D) SEISMIC SHEAR WAVE PROPAGATION

Dynamic ground responses during an earthquake-induced ground shaking event are evaluated by solving the dynamic equation of motions representing the propagation of seismic wave through the ground. For engineering applications, soil responses to vertically propagating seismic shear waves (SH) is of primary concern.

The scope of this module is limited to evaluating site- or support specific soil deposits consisting of homogenous soil layers with horizontal boundaries that extend to infinite. For mathematical modelling and numerical evaluation of the dynamic responses during earthquake shaking, such a soil deposit is represented as a horizontally layered 1-D soil-column as shown in Figure A-1. When the 1-D soil-column of height (H) in Figure A-1 is subjected to earthquake-induced base ground motions in one horizontal direction (as shown by the particle motion vector) due to vertically propagating seismic waves, soils at any depth (x) from the ground surface and time (t) will experience only horizontal displacement, u(x, t).

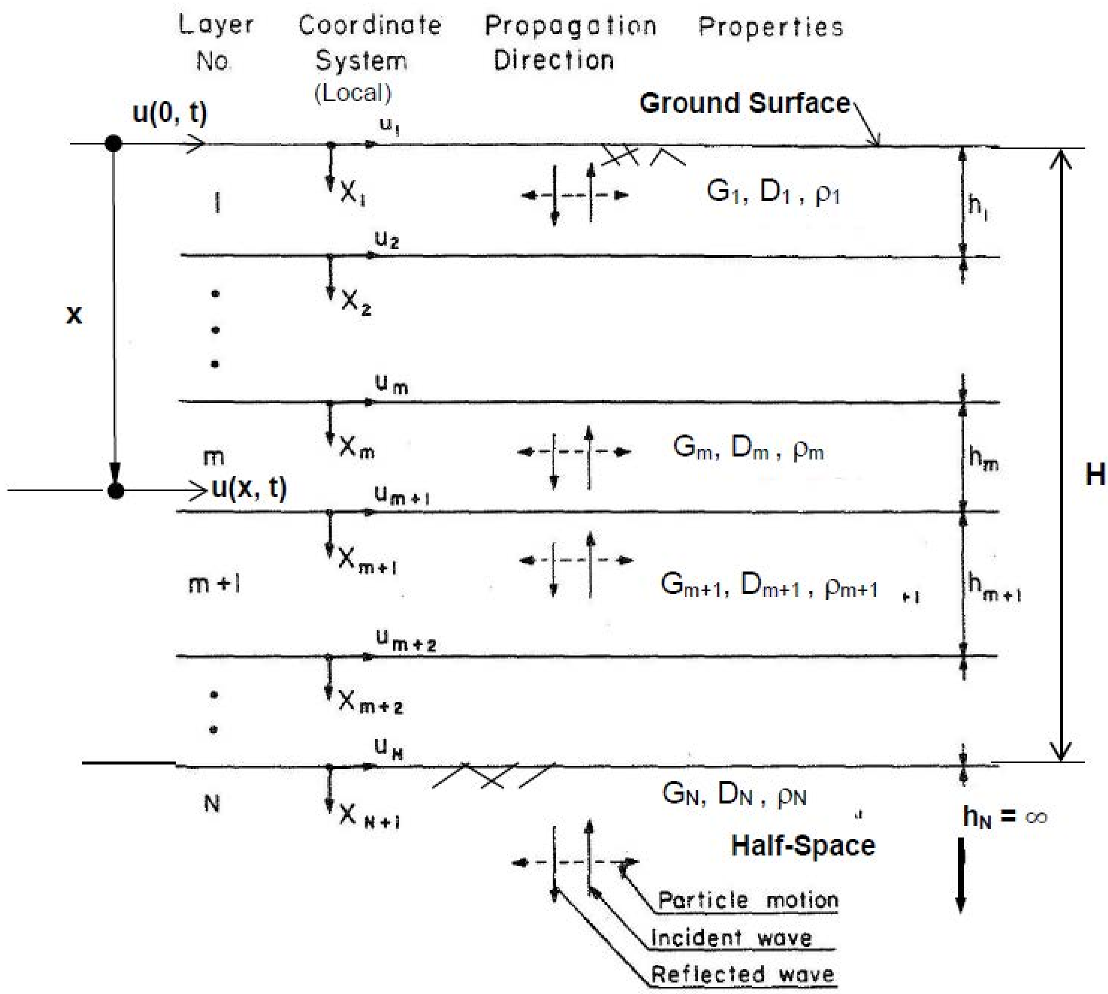

Figure A-1. 1-D Soil Column Model (Modified after Schnabel et al., 1972)

For mathematical formulation, the dynamic soil shear stress (τ) and strain (γ) relationships of the soil elements during earthquake-induced ground shaking is represented by the Kelvin-Voigt viscoelastic material model:

$$\tau = G(\gamma)\gamma + \eta \frac{\delta\gamma}{\delta t} \qquad \text{(A-1)}$$

Where, G is the shear modulus, η is the soil viscosity, and t is the time. Based on Equation A-1, the shear stress consists of an elastic part proportional to γ and a viscous part proportional to strain rate.

Based on the above shear stress-strain model, when modelled as a continuum, the one-dimensional (1D) dynamic equation of motion for seismic shear waves propagating vertically through a homogenous horizontal soil layer or profile is written as (Schnabel et al, 1972).

$$\rho \frac{\partial^2 u}{\partial t^2} = G \frac{\partial^2 u}{\partial x^2} + \eta \frac{\partial^3 u}{\partial x^2\, \partial t} \qquad \text{(A-2)}$$

Where, the ground or soil responses are represented by the following parameters:

u = u(x, t) is the horizontal ground displacement at time (t) at depth (x) from the top of the vertical soil column of height (H) as shown in Figure A-1 at time (t), and

$$\rho = \frac{\text{Soil Total Unit Weight } (\gamma_t)}{g}$$

g = Acceleration due to gravity

Once the above equation is solved for the displacement-time history u(x, t) at any selected depth x, e.g., u(0, t) at the ground surface where x=0.0, the other required seismic ground motion response parameters, including the velocity-time history $\dot{u}$(x, t), acceleration time-history $\ddot{u}$(x, t), the shear stress-time history τ(x, t) and the shear strain-history γ(x, t), and the ARS Curve at depth x, including the ground surface (i.e., x=0.0), can be easily obtained by additional analysis.

For harmonic waves, the displacement u(x, t) in Equation A-2 is written as:

$$u(x, t) = U(x)\, e^{i\omega t} \qquad \text{(A-3)}$$

where U(x) is a real number.

Substituting in Equation A-2, the equation of motion becomes the following ordinary differential equation:

$$(G + i\omega\eta) \frac{dU(x)}{dx} = -\rho\omega^2 \eta \qquad \text{(A-4)}$$

Or,

$$G^* \frac{dU(x)}{dx} = -\rho\omega^2 \eta \qquad \text{(A-5)}$$

Here, G* is the complex shear modulus, defined as:

$$G^* = G + i\omega\eta \tag{A-6}$$

For additional information in the concept of complex modulus for modelling dynamic response of soils see Towhata (2008).

The solution of the above equation of motion is written as:

$$u(x,t) = Ee^{\,i(k^*x + \omega t)} + Fe^{-i(k^*x - \omega t)} \tag{A-7}$$

Where, E and F depends in the boundary conditions and k* is complex wave number given by:

$$k^* = \omega\sqrt{\frac{\rho}{G^*}} \tag{A-8}$$

The first and second terms in Equation A-7 represent propagation of the harmonic incident wave propagation in the negative x-direction (upward) and reflected wave in the positive x-direction (downward), respectively.

A-2. EQUVALENT-LINEAR (NL) ANALYSIS

A-2.1. Soil Material Models

Based on the “Equivalent-Linear” concept, the hysteretic shear stress-strain behaviors of soils when subjected to a cycle of shear loading is represented by the ***secant*** shear modulus (G_p) and the hysteretic damping ratio (D) calculated based on the relationship shown in Figure A-2(a).

The plots of the peak points (τ_p, γ_p) obtained from cyclic laboratory tests on soil samples, result in the stress-strain (τ-γ) curve, identified as the “Backbone Curve’ in Figure A-2 (a). This backbone (τ-γ) curve is often taken the same as the monotonically loaded shear stress-strain curve.

In the Equivalent-Linear analysis, the symbol G is used to represent the secant shear modulus G_p, and the parameters G (secant modulus) and D (hysteretic damping) are termed as “equivalent-linear” material parameters.

The variation of the parameter G with γ is more conveniently presented in the normalized form (G/G_{max}) versus γ as shown in Figure A-2(b). Here G_{max} is the maximum or the zero-strain ($\approx$small strain) soil shear modulus as shown in Figure A-2(a). This curve is termed as the normalized modulus reduction (MR) curve.

The shear stress (τ) at any point on the backbone curve corresponding to the shear strain (γ) is determined as:

$$\tau = \rho V_S^2 \left(\frac{G}{G_{max}}\right)\gamma \tag{A-9}$$

Where, ρ is the mass density and V_S is the seismic shear wave velocity of the soil.

Alternatively, if the non-linear stress-strain backbone curve is known, Equation A-9 may be used to derive (G/G_{max}) versus γ (or the MR) curve.

The maximum value of the shear stress of the (τ,γ) curve that can be generated during a DGRA analysis for a soil layer based on the known ρ, V_s and a reference MR, as discussed later in this module, is often denoted as $(\tau_{mob})_{max}$. The corresponding mobilized friction angle $(\phi_{mob})_{max}$ is calculated as:

$$(\phi_{mob})_{max} = tan^{-1}\left[\frac{(\tau_{mob})max}{\sigma'_{vo}}\right] \quad \text{(A-10)}$$

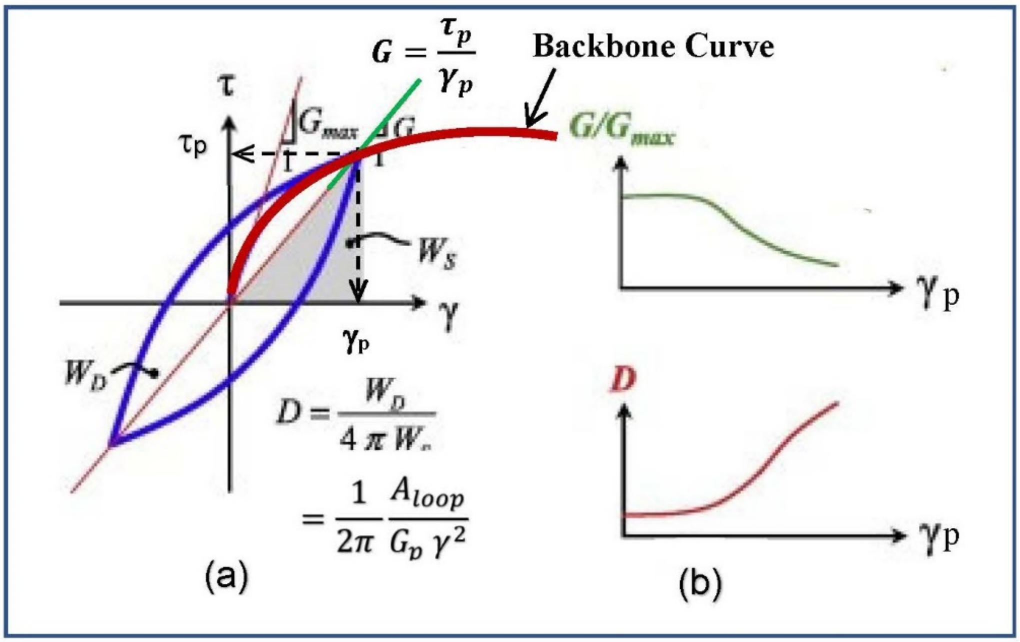

Figure A-2. Schematic Representation of Equivalent-Linear Modelling of Soil Stress-Strain Behavior: (a) Hysteresis Shear-Stress-Strain Loop Depicting the Initial Tangent Modulus (G_{max}), and Secant Modulus (G) and Damping Ratio (D) for a Full Cycle of Shear Loading with Peak Shear Strain Amplitude (γ_p); (b) Normalized (G/G_{max}) or Modulus Reduction (MR) Curve and Damping Ratio (DR) Plots as a Function of the Peak Shear Strain Amplitude (γ_p).

The normalized modulus reduction (MR) and damping ratio (DR) curves in Figure A-2(b) represent reduction in the secant shear modulus and increase in the damping ratio with the amplitude of the cyclic shear strain cycles. Together these two curves are used in the EL analysis to model the cyclic viscoelastic shear stress-strain behavior of soils during dynamic loading generated by the vertically propagating seismic shear waves during an earthquake event.

Both G and D depend on a number of soil, environmental and loading condition related factors, including the soil types, the mobilized shear strain (γ), density (or void ratio), initial effective vertical stress (σ'_{vo}), gradations, plasticity index (PI), over-consolidation ratio, geologic age, cementation, and strain rate (see, Kramer, 1996). In the ground response analysis, both parameters are specified as a function of the most significant factor, the mobilized shear strain (γ) at the end of each loading cycle.

The EL analysis is formulated to solve the equation of motion (Equation A-2) in the frequency domain. This equation of motion, which is time domain, is first transformed into the frequency domain by using transfer functions (See Schnabel et al, 1972) at the layer boundaries. This can only be done for linear materials. The EL analysis accomplishes this for soils, which are truly non-linear, by using the concept of “equivalent-linear” material properties.

In the material model shown in Figure A-2, the energy loss during a given shear loading cycle is represented by the hysteric damping ratio (D) calculated based on the ratio of the dissipated energy to the maximum elastic strain energy, both measured graphically based on results of cyclic laboratory testing. The hysteretic damping factor (D) has been found to be independent of the frequency of loading (Kramer, 1996; Towhata, 2008).

If the soil shear strain $\gamma\ (= \frac{\delta u}{\delta x})$ is harmonic with a frequency ω, the soil material or hysteretic damping ratio (D), when modelled as equivalent to the viscous damping ratio of a discrete Single Degree of Freedom (SDOF) Kelvin-Voigt model of mass m with natural frequency $\omega_n = \omega$, is related to the soil viscosity (η) as follows:

$$D = \frac{\omega\eta}{2G} \qquad \text{(A-11)}$$

Based on Equation (A-11), soil material damping is proportional to loading frequency ω. However, based on laboratory testing, the energy loss or the damping in real soil is insensitive to the loading frequency. To eliminate the frequency dependence of the damping ratio (D), Equation A-11 is rewritten as follows to produce an equivalent soil viscosity:

$$\eta = \left(\frac{2G}{\omega}\right)D \qquad \text{(A-12)}$$

The soil shear modulus G is also frequency independent.

Substituting for η from Equation (A-12) in Equation A-6,

$$G^* = G(1 + 2iD) \qquad \text{(A-13)}$$

The complex shear modulus G* is also loading frequency independent since both G and D parameters are frequency-independent.

A-2.2 Modelling Modulus Reduction and Damping Curves

A-2.2.1 Konder and Zelasko (KZ) Hyperbolic Model

Konder and Zelasko (1963) originally proposed the following hyperbolic curve to model the non-linear shear stress-strain (τ-γ) or the backbone curve for soils as shown in Figure 2(a).

$$\boldsymbol{\tau} = \frac{G_{max}\,\gamma}{1+\frac{G_{max}}{\tau_{max}}\gamma} = \boldsymbol{\tau_{max}}\left\{\frac{\frac{\gamma}{\gamma_r}}{\left(\left(1+\frac{\gamma}{\gamma_r}\right)\right)}\right\} \qquad \text{(A-14)}$$

Where, G_{max} = Maximum or initial tangent shear modulus

τ_{max} = Shear stress at failure (or shear strength),

and γ_r = Reference shear strain = $\left(\frac{\tau\ max}{G_{max}}\right)$

The above definition of the reference strain (γ_r) was originally proposed by Hardin and Drnevich (1972). The parameter γ_r = (τ_{max}/G_{max}) is a material constant.

The above KZ hyperbolic relationship for the backbone curve may be rewritten as

$$\frac{G}{G_{max}} = \frac{1}{1+\frac{\gamma}{\gamma_r}} \qquad \text{(A-15)}$$

$$\text{or,}\quad \frac{\tau}{\tau_{max}} = \frac{\frac{\gamma}{\gamma_r}}{1+\frac{\gamma}{\gamma_r}} \qquad \text{(A-16)}$$

Thus, for the KZ model, when γ=γ_r, τ/τ_{max} = G/G_{max} =0.5

The soil material models used currently are usually some modified versions of the above original KZ hyperbolic model.

In the modified hyperbolic models, the reference strain (γ_r) is variably defined. In that cases, the parameter (γ_r) is referred to as the "pseudo-reference strain" to distinguish from the above definition. The definition for the "pseudo-reference strain" is specific to the soil material model(s) and thus must be identified accordingly.

A-2.2.2 Modified Konder-Zelasko (Modified KZ) Hyperbolic Model

Currently used empirical modulus reduction (MR) correlations have been developed by fitting experimental data to the following modified KZ hyperbolic model of the backbone curve:

$$\frac{G(\gamma)}{G_{max}} = \frac{1}{1+\left(\frac{\gamma}{\gamma_r}\right)^{\alpha}} \qquad \text{(A-17)}$$

Where, γ_r and α are the model or regression parameters referred to as the pseudo-reference shear strain and the curvature coefficient, respectively. The parameter α defines the curvature of the modulus reduction curve. The parameters γ_r and α depend on the soil conditions and properties such as the mean effective stress (σ'_o), coefficient of uniformity (C_u) for coarse-grained soils and PI for fine-grained soils.

The model parameters (i.e., γ_r and α) necessary to define the various currently used recent empirical modulus reduction curves are provided in their respective reference documents. Stewart et al (2014) provides a summary of these models.

The model parameter G_{max} needed to determine the soil modulus G at any shear strain (γ) is calculated based on the following user input soil properties:

- Soil total unit weight (γ_t), and
- Seismic shear wave velocity (V_s)

Uncertainties associated with the modulus reduction curve as well as V_s should be considered in the site-specific DGRA. See Stewart et al (2014) for details.

A-2.3 Reference MR and DR Curves

In the EL model, the equivalent modulus reduction (MR) and damping ratio (DR) curves are used to define the soil constitutive model for each soil layer. These curves may be determined based on dynamic laboratory tests performed on carefully collected undisturbed samples of the representative site soils.

Researchers have developed reference model curves for different soil types by testing large numbers of samples. These reference model curves are developed based on the assumption that the soil backbone curves can represented by hyperbolic curves. The curve fitting parameters for the hyperbolic models are correlated with basic soil properties and conditions such the soil type, gradation, initial effective overburden or confining pressure, uniformity coefficient (C_u) for coarse-grained soils, and soil plasticity index (PI) for fine-grained soils, etc.

Therefore, alternative to laboratory testing of site-specific soil, the MR and DR curves for site-specific DGRA are most often estimated based on the following recently developed empirical reference models (Stewart et al, 2014):

- Darendeli (2001)
- Roblee and Chiu (2004)
- Meng (2003)
- Zhang et al (2005), and
- Choi (2008)

Other more recently published empirical models may be used with justifications. Older empirical relationships than those listed herein should not be used.

The MR and DR curves obtained based on these empirical models are often referred to as the “Reference Curves”. These curves were developed by fitting the above modified KZ hyperbolic model into experimental data. Each empirical model consists of a set of reference MR and DR curves.

For details on the above empirical reference models, see the corresponding reference study listed under the reference section of this module. Applicability of the models depends on the soil types, shear strain range and other factors.

For a summary of the various attributes of the above empirical or reference correlations, for both the MR and DR curves, including their applicability in terms of the soil types and the shear strain ranges, see Appendix B. For additional details, see the Stewart el al. (2014).

The modulus reduction and the damping ratio curves may be specified using discrete data points only for the EL analysis method. The modulus reduction curves for use in an EL analysis can also be developed from any hyperbolic backbone curve, including those discussed below.

A-3 NON-LINEAR (NL) ANALYSIS

A-3.1 Soil Material Models

The backbone curve for NL analysis may range from the simple models, such that the KZ and MKZ hyperbolic models discussed above to more complex models, including those discussed below. The material model in a NL analysis consists of the backbone curve, a set of rules that govern the unloading-reloading behaviors, zero (or strain small strain) damping ratio, stiffness and strength degradation or the model for excess pore pressure generation and dissipations, and other effects.

A-3.1.1 Matasovic-Konder-Zelasko (MKZ) Hyperbolic Model

Matasovic (1993) recommended a modified hyperbolic pressure dependent model based on the above KZ model. The normalized shear resistance versus shear strain backbone curve in the Matasovic (1993) modified hyperbolic model is defined by:

$$\frac{\tau}{\sigma'_{v0}} = \frac{G^*_{mo}\,\gamma}{1+\beta\left(\frac{\gamma}{\gamma^*_r}\right)^s} = \frac{G^*_{mo}\,\gamma}{1+\beta\left(\frac{G^*_{mo}}{\tau^*_{mo}}\gamma\right)^s} \qquad \text{(A-18)}$$

Where, σ'_{vo} = Initial vertical effective stress

G^*_{mo} = Initial tangent shear modulus

$\tau^*_{mo} = (\tau_{mo}/\sigma'_{vc})$ = Normalized shear resistance

τ_{mo} = Maximum shear resistance, corresponds approximately to the upper boundary of the shear strain range of 1-3%, which is usually much lower than the actual failure strain ($\gamma_f \approx 10\%$) of most soils.

And β, *s* and γ_r are the model parameters. The parameters β and *s* controls the position and adjusts the slope of the backbone curve, respectively.

The MKZ model assumed that the NL seismic response of soils do not exceed the shear strain range of 1-3%. As seen from Eq. A-18, the pseudo-reference strain γ_r is defined herein as, $\gamma_r = \tau^*_{mo}/G^*_{mo}$.

The MKZ model is used in the DMOD-2000. Based on Matasovic and Vucetic (1993), for sand β = 1.0 to 1.9 and *s = 0.67 to 0.98.* Generally, β*=1.0 and s* $\approx$0.92 per Groholski et at. (2016).

A-3.1.2 Pressure-Dependent MKZ Model

Hashash and Park (2001) extended MKZ hyperbolic model by incorporating the pressure dependency. In this extended model termed as the "Pressure-Dependent MKZ Model", the model parameter γ_r and hence the shear stress (τ) is defined as a function of the effective vertical stress (σ'_v) stress. This was accomplished by defining the model parameter γ_r as follows:

$$\gamma_r = \gamma_{ref} \left(\frac{\sigma'_v}{(\sigma'_v)_{ref}} \right)^b \quad \text{(A-19)}$$

Here, $(\sigma'_v)_{ref}$ is the reference effective vertical stress at which $\gamma_r = \gamma_{ref}$, *and b is a curve fitting parameter.*

The Pressure-Dependent MKZ Model is available for use in the DEEPSOIL.

A-3.1.3 General Quadratic/Hyperbolic (GQ/H) Model with Shear Strength Control

Recent empirical correlations for the backbone curves, such as Darendeli (2001), were developed by fitting to the hyperbolic model experimental data obtained at small to medium shear strain levels. Shear resistances at large strains are determined by extrapolation, which can either underestimate or overestimate the shear resistances at larger strains. Depending on the values of the curve fitting parameters, the other hyperbolic models described above may also underestimate or overestimate the shear resistances at larger strains (>1-3%). Some of these models also represent the shear-stress-curves at small strain levels less accurately (Groholski et al., 2016)

To address the above small and large-strain related limitations, Groholski et al. (2016) proposed the following new non-linear stress-strain (τ-γ) model referred to as the Generalized Quadratic/Hyperbolic (GQ/H) Model. It incorporates corrections to obtain a model curve that fits better with the laboratory measured initial or monotonic stress-strain (i.e., backbone) curve) at all strain levels, including the actual (measured or user estimated) shear strength or failure shear stress at large strains (e.g., at γ_f =10%). The GQ/H model is expressed as follows, for the case the parameter $\theta_\tau \neq 0$:

$$\frac{\tau}{\tau_{max}} = \frac{2\left(\frac{\gamma}{\gamma_r}\right)}{1+\left(\frac{\gamma}{\gamma_r}\right)+\sqrt{\left[1+\left(\frac{\gamma}{\gamma_r}\right)\right]^2-4\theta_\tau\left(\frac{\gamma}{\gamma_r}\right)}} \quad \text{(A-20)}$$

Where, γ_r = Reference strain = (τ_{max}/G_{max})

τ_{max} = Nominal shear resistance (or shear strength impli)

G_{max} = Initial shear modulus, and

θ_τ = Shear resistance correction factor at large strains ($0<\theta_\tau\leq 1$)

The shear resistance reduction factor is a function of the shear strain (γ) over a defined range of large strain and is given by the following hyperbolic function.

$$\theta_\tau = \left[\theta_1 + \theta_2 \frac{\theta_4 \left(\frac{\gamma}{\gamma_r}\right)^{\theta_5}}{\theta_3{}^{\theta_5} + \theta_4 * \left(\frac{\gamma}{\gamma_r}\right)^{\theta_5}}\right] \leq 1.0 \qquad \text{(A-21)}$$

Where, θ_1, θ_2, θ_3, θ_4 and θ_5 are curve-fitting parameters. These parameters are specific to this model.

These parameters may be determined by fitting laboratory-obtained MR curves or to the Reference MR curves developed based on the empirical models discussed earlier, or obtained from the reference study (i.e., Groholski et al., 2016).

DEEPSOIL has an option to select GQ/H model. It has a subroutine to determine the curve fitting parameters θ_1, θ_2, θ_3, θ_4 and θ_5 based on the user specified reference MR curve.

For a given soil layer, the user specifies the soil nominal shear resistance or shear strength (τ_{max}) at large strains, the initial shear modulus (G_{max}) or the soil unit weight and V_s. The curve fitting parameters (θ_1, θ_2, θ_3, θ_4 and θ_5) are used to construct the shear resistance corrected backbone curve.

Groholski et al. (2016) recommends to allow curve by this method to where the implied shear strength (i.e., the model predicted shear stress at γ_f =10%) reaches a desired fraction of the actual shear strength (τ_f) such as 95%. When using one of the “Reference” MR curves (i.e., see Appendix B), curve fitting for small strain should be performed for up to the shear strain range of the applicability of the data for the selected reference curve with a condition that the model predicts a maximum shear stress at shear strain (γ) =10% that is a close to (e.g., 95%) the user specified shear strength (τ_f).

The user specified the normalized modulus reduction curve or select one of the available “Reference” MR curves for a given soil layer. Upon completing the soil layer domain, the GQ/H curve-fitting routine calculates the corrected shear resistances at large strains, develops the corrected shear stress-strain backbone curve and provides the values of parameters θ_1, θ_2, θ_3, θ_4 and θ_5 used. These values are then directly used in subsequent GQ/H model components for the backbone curve degradation due to excess pore pressure generation when performing effective -stress bases NL analysis and in soil hysteretic behavior modelling.

A-3.2 Modelling Unloading-Unloading Behavior

In NL analysis initial loading follows the backbone curve. The cyclic hysteretic behavior is modelled by a set of rules so that the time-domain solution follows the actual unloading-reloading curve when reversals in the shear stress occur. These rules include the Masing Rules and the Extended Masing Rules. See Kramer (1996) and Stewart et al. (2014) for additional information.

Non-linear analysis procedures using Massing Rules in conjunction of most availabe non-linear stress-strain models are not able obtain good match with laboratory measured variations of the modulus and damping with cyclic shear strain. Phillips and Hashash (2008) provided a methodology for altering the Masing Rules to provide better better results. This methodology is referred to as the *Modulus Reduction and Damping Factor*

(MRDF) approach. In this approach, soil damping behavior is modified by applying a reduction factor $F(\gamma_m)$ to the hysteretic damping factor calculated by using Masing Rules (ξ_{Masing}) as follows:

$$\xi_{MasingHysteretic} = F(\gamma_m) * \xi_{Masing} \qquad \text{(A-22)}$$

Where, γ_m is the maximum shear strain experienced by the soil at any time and ξ_{Masing} is the hysteretic damping ratio calculated using Masing Rules, based on the MR curve.

The reduction factor $F(\gamma_m)$ is given by:

$$F(\gamma_m) = p_1 + p_2 \left(1 - \frac{G_{\gamma m}}{G_{max}}\right)^{p_3} \qquad \text{(A-23)}$$

Where, p_1, p_2 and p_3 are non-dimensional model parameters that results in the best fit with the target or specified damping curve or data. $G_{\gamma m}$ is the shear modulus at strain γ_m.

The MRDF approach is implemented in the DEEPSOIL by multiplying the unloading-reloading curve generated based on the Extended Masing Rules and the selected hyperbolic model (i.e., backbone curve) by the reduction factor $F(\gamma_m)$. DEEPSOIL offers the MRDF approach as one of the two default options for hysteretic/unloading-reloading formulation with both the Pressure-Dependent MZK model and the GQ/H model. The option to use the MRDF approach is referred to as the "Non-Masing Re/Unloading" option in the DEEPSOIL. The other hysteretic unloading-reloading formulation option in DEEPSOIL is "Masing Un/Reloading" which corresponds to the unaltered Extended Masing Rules.

When using the 'Non-Masing Re/Unloading" or the MRDF option, first determine the selected constitutive model parameters that provide best fit to the target or specified backbone curve or data.

Then determine the reduction factor parameters p_1, p_2 and p_3 that provide best fit of the target damping curve without modifications of the fit to the backbone curve.

A-3.2 Small Strain Viscous Damping for NL Analysis

Hysteretic damping is incorporated in the NL analysis directly by the unloading-reloading rules. Small or zero-strain damping must be used in a NL analysis in addition to the hysteretic damping. This is done by retaining the viscous damping matrix [C] in the Equation 8 (Hashash and Park, 2001). The matrix [C] is formulated using the small strain damping factor.

The matrix [C] in Equation 8 represents Rayleigh damping (Rayleigh and Lindsey, 1945) which is considered a function of the mass matrix and the stiffness matrix. A range of methodologies, termed as the simplified, full, and extended Rayleigh damping matrix formulations have been recommended for generating the viscous damping matrix [C]. For details, see Hashash et al. (2010) and Stewart et al (2014).

For Caltrans bridge projects, frequency-independent formulation should be use for the viscous damping matrix [C].

DEEPSOIL offers several options, including the simplified "Frequency-Independent" option for the small strain damping matrix formulation. This option should be used in the NL analyses conducted for Caltrans project sites. In this option, the small strain viscous matrix [C] in Equation 8 is formulated using zero shear strain equivalent damping ratio (ξ_0) as follows:

$$[C] = \frac{2\xi_0}{\omega_o}[K] \qquad \text{(A-24)}$$

Where, ω_o is the natural frequency $\left(=\sqrt{\frac{m}{k}}\right)$ and $[K]$ is the shear stiffness matrix for the soil.

Hashash and Park (2001) recommends the following relationship for the pressure dependent zero strain small strain/viscous damping ratio is defined as follows:

$$\xi_o = Small\ Strain\ Damping\ Ratio\ (D_{min})\ x\ \left(\frac{1}{\sigma_v'}\right)^d \qquad \text{(A-25)}$$

Here, d = Model parameter for pressure dependent viscous/small strain damping, and

σ'_v = Effective vertical stress (in tsf)

D_{min} = Small strain damping ratio obtained from the DR curve.

The value of the parameter d may be taken as equal to 0.0 to model the small strain damping as independent of the effective vertical stress.

A-3.3 Excess Porewater Generation Models

A variety of models are available for the generation of excess porewater pressures in saturated soil during ground shaking. DEEPSOIL v7.0 includes number of such models. These include Dobry and Matasovic models for sand and clay (Vucetic and Dobry, 1986; Matasovic, 1993; Matasovic and Vucetic, 1993; Matasovic and Vucetic, 1995), Green, Mitchell and Polito (Green et al; 2000) or GMP model for cohesionless soils, and Park and Ahn (Park and Ahn, 2013) model for sand

In addition to the above, DEEPSOIL v7.0 also offers an additional model designated as the "Generalized" model which allows for user-defined excess porewater pressure generation model. This model is developed based on Berrill and Davis (1985) and Green et al. (2000) energy-based models.

For brief descriptions of the above model and recommendations for use, see the User Manual for DEEPSOIL v7.0 (Hashash et al, 2020). These models vary significantly in their basis, complexities and the types and number of input parameters needed. For additional information, including the meaning and how to determine or the model parameters for a specific model, refer to the original or source reference included under the Reference section of this module and the user manual of the specific computer software used.

APPENDIX B

Supplemental Information on the "Reference" Material Models (MR and DC Curves)

(After Stewart et al, 2014)

Table B-1. Summary of attributes of hyperbolic MR curve models from literature.

Reference	Model for γ_r	Model for α	Soil types considered	Applicable strain range
Darendeli [2001][1]	$\gamma_r(\%) = (\phi_1 + \phi_2 + PI \times OCR^{\phi_3}) \times (\sigma'_0/p_a)^{\phi_4}$	$\alpha = \phi_5$	Generic (110 samples from 20 sites)	0.0001–0.5%
Menq [2003][2]	$\gamma_r(\%) = 0.12 \times C_u^{\phi_1} \times (\sigma'_0/p_a)^{\phi_2}$	$\alpha = 0.86 + 0.1 \times \log(\sigma'_0/p_a)$	Granular with D_{50} > ~ 0.3 mm (59 reconstituted specimens)	0.0001–0.6%
Roblee and Chiou [2004]	From table GeoIndex model parameters	From table GeoIndex model parameters	Clay, sand and silt (154 samples from 28 sites)	0.0001–4.0%
Zhang et al. [2005][3]	$\gamma_r(\%) = \gamma_{r1}{}^k\ (\sigma'_0/p_a)^k$		Mineral soils in South Carolina (122 samples)	0.0001–0.3%
	Quaternary:	Quaternary:		
	$\gamma_{r1} = 0.011PI + 0.0749$	$\alpha = 0.0021PI + 0.834$		
	$k = 0.316e^{-0.0142PI}$	Residual/saprolite:		
	Residual/saprolite:	$\alpha = 0.0043PI + 0.794$		
	$\gamma_{r1} = 0.0009PI + 0.0385$	Tertiary and older:		
	$k = 0.42e^{-0.0456PI}$	$\alpha = 0.0043PI + 0.794$		
	Tertiary and older:	$\alpha = 0.0009PI + 1.026$		
	$\gamma_{r1} = 0.0004PI + 0.0311$			
	$k = 0.316e^{-0.011PI}$			
Choi [2008]	$\gamma_r(\%) = 0.046(\sigma'_0/p_a)^{0.46}$	$\alpha = -0.298\log_{10}(\gamma_r) + 0.656$	Bandelier Tuff, Pajarito Plateau, NM (38 samples)	0.0001–0.04 %

[1] See Table 2 for parameters.
[2] See Table 3 for parameters.
[3] γ_{r1} is γ_r at $\sigma'_0 = p_a$.

Table 2 Parameters for Darendeli [2001] model for modulus reduction.

Parameter	Value
ϕ_1	0.0352
ϕ_2	0.001
ϕ_3	0.3246
ϕ_4	0.3483
ϕ_5	0.92

Table 3 Parameters for Menq [2003] model for modulus reduction.

Parameter	Value
ϕ_1	-0.6
ϕ_2	0.382

Table 4 GeoIndex soil classes [Roblee and Chiou 2004].

GeoIndex Abbreviation	GeoIndex Soil Description	Passing #200	Plasticity Index
1–PCA	Primarily Coarse –All Plasticity Values	<=30%	All
2–FML	Fine-Grained Matrix–Lower Plasticity	>30%	<=15%
3–FMH	Fine-Grained Matrix–Higher Plasticity	>30%	>15%

Table 5 GeoIndex model parameters for modulus reduction [Roblee and Chiou 2004].

GeoIndex Class	1-PCA Soil		2-FML Soil		3-FMH Soil	
Depth (m)	γ_r	α	γ_r	α	γ_r	α
0–10	0.032	0.85	0.057	0.9	0.085	0.98
10–20	0.044	0.85	0.065	0.9	0.097	0.98
20–40	0.061	0.85	0.074	0.9	0.111	0.98
40–80	0.085	0.85	0.085	0.9	0.126	0.98
80–160	0.130	0.85	0.130	0.9	0.130	0.98
>160	0.200	0.85	0.200	0.9	0.200	0.98

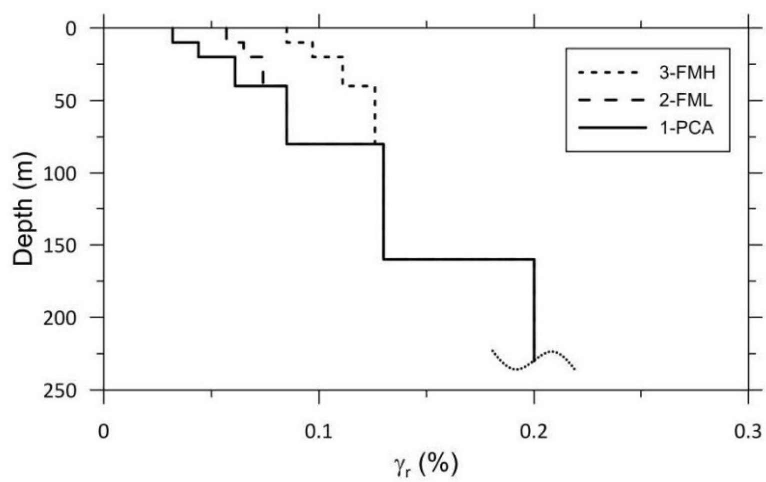

Figure A-1 Effect of depth on reference shear strain (γ_r) for different categories of soils [Roblee and Chiou 2004].

Table B-6. Summary of attributes of hyperbolic soil damping models from literature.

Reference	Model for D_{min}	Model for D-D_{min}	Soil types considered	Applicable strain range
Darendeli [2001][1]	$D_{min} = (\phi_6 + \phi_7 \times \mathrm{PI} \times \mathrm{OCR}^{\phi_8}) \times \sigma_0'^{\phi_9} \times [1 + \phi_{10} \ln(freq)]$	$D(\gamma) - D_{min} = b \times D_M(\gamma) \times (G(\gamma)/G_{max})^{0.1}$ $b = \phi_{11} + \phi_{12} \times \ln(N)$	Generic (110 samples from 20 sites)	0.0001–0.5%
Menq [2003][2]	$D_{min} = \phi_3 \times C_u^{\phi_4} \times D_{50}^{\phi_5} \times (\sigma_0'/p_a)^{\phi_6}$	Same as Darendeli [2001]	Granular (59 reconstituted specimens)	0.0001–0.6%
Roblee and Chiou [2004]	From Table 9	Same as Darendeli [2001]	Clay, sand and silt (154 samples from	0.0001–4.0%
Zhang et al. [2005]	$D_{min} = D_{min1}(\sigma_0'/p_a)^{-k/2}$ $D_{min1} = a(PI) + b$ a=0.008 b=0.82	$D(\gamma) - D_{min} = 10.6(G/G_{max})^2 - 31.6(G/G_{max}) + 21.0$	Mineral soils in South Carolina (122 samples)	0.0001–0.3%
Choi [2008]	$D_{min} = 119e^{-0.0026\gamma_t}$ γ_t = material density	$D(\gamma) - D_{min} = C(\gamma/\gamma_D)^{a_D}$ $\gamma_D = 0.038(\sigma_0/p_a)^{0.54}$ $a_D = -0.545 \log_{10} \gamma_D + 0.337$	Bandelier Tuff, Pajarito Plateau, NM (38 samples)	0.0001–0.04%

[1] See Table 7 for parameters.
[2] See Table 8 for parameters.

The Masing Damping (D_M) used in the Darendeli (2001) model for D-D_{min} can be estimated based on the following approximate method using the γ_r and α parameter describing the shape of the backbone curve (See Table B-1):

For α = 1.0,
$$D_{M,\alpha=1.0}(\gamma)[\%] = \frac{100}{\pi}\left[4\,\frac{\gamma-\gamma_r \quad ln\left(\frac{\gamma+\gamma_r}{\gamma_r}\right)}{\frac{\gamma^2}{\gamma+\gamma_r}} - 2\right] \qquad \text{Eq. B-1}$$

For, $\alpha \neq 1.0$,
$$D_M = c_1(D_{M,\alpha=1.0}) + c_2\,(D_{M,\alpha=1.0})^2 + c_3\,(D_{M,\alpha=1.0})^2 \qquad \text{Eq. B-2(a)}$$

Where,
$$c_1 = 0.2523 + 1.2861\alpha - 1.1143\alpha^2 \qquad \text{Eq. B-2(b)}$$

$$c_2 = -0.0095 - 0.071\alpha + 0.0805\alpha^2 \qquad \text{Eq. B-2(c)}$$

$$c_3 = 0.0003 + 0.0002\alpha - 0.0005\alpha^2 \qquad \text{Eq. B-2(d)}$$

The D_{min} and b parameters for use in Roblee and Chiou (2004) are provided in Table B-9 below as a function of depth.

Table B-7

Parameters for Darendeli [2001] model for soil damping.

Parameter	Value
ϕ_6	0.8005
ϕ_7	0.0129
ϕ_8	-0.1069
ϕ_9	-0.2889
ϕ_{10}	0.2919
ϕ_{11}	0.6329
ϕ_{12}	-0.0057

Table B-8

Parameters for Menq [2003] model for soil damping.

Parameter	Value
ϕ_3	0.55
ϕ_4	0.1
ϕ_5	-0.3
ϕ_6	-0.08
ϕ_{11}	0.6329
ϕ_{12}	-0.0057

Table B-9

GeoIndex model parameters for damping [Roblee and Chiou 2004].

Depth (m)	D_{min}	b
0-10	1.30	1.62
10-20	1.15	1.62
20-40	1.02	1.62
40-80	0.90	1.62
80-160	0.80	1.62
>160	0.70	1.62

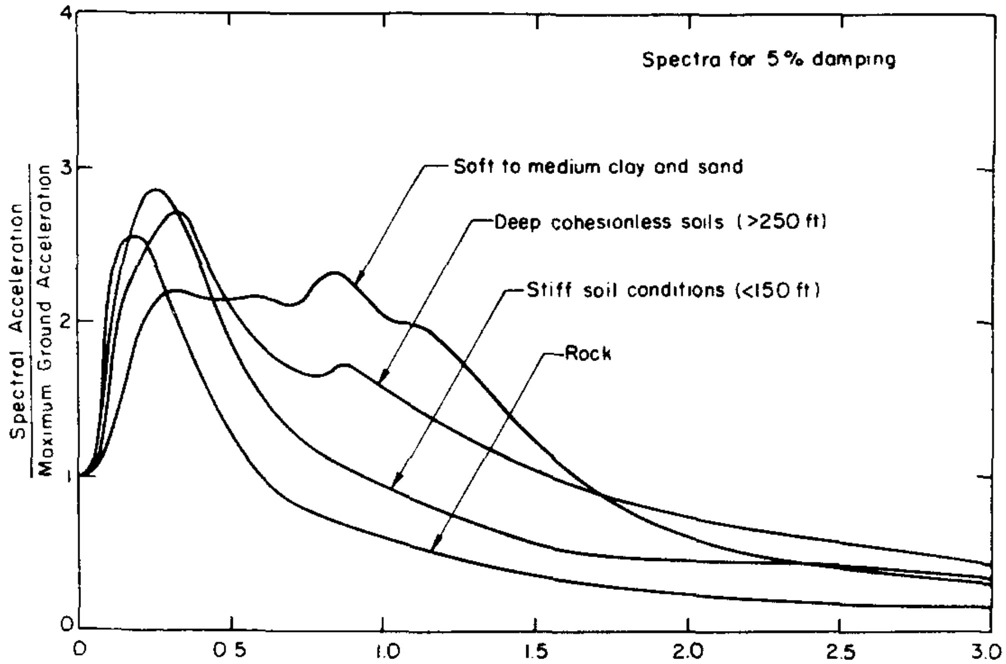

Figure B-2. Average Normalized Acceleration Response Spectra for Different Local Soil Profile Conditions (After Seed et al,., 1974)

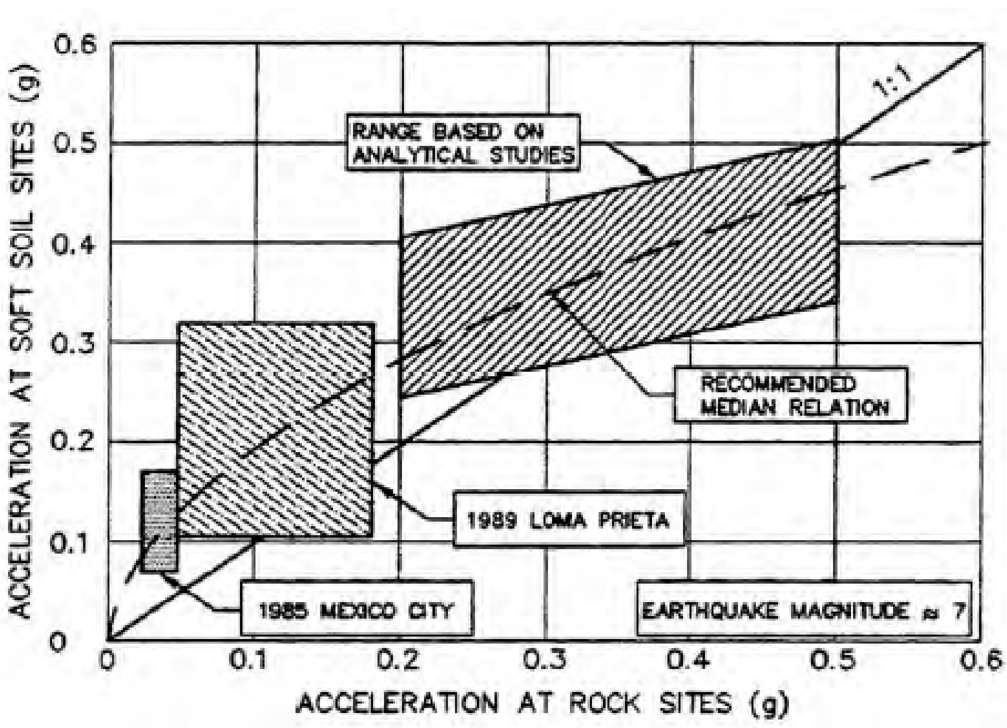

Figure B-3 Approximate Relationship Between Horizontal Peak Ground Accelerations on Rock and at the Surface of Soft Soil Sites (Idriss, 1990)

APPENDIX C

Table C-1. Characteristics of the Selected Seed Motions

Record ID No.	Original Database /ID No.	Station (Name/ Coordinates)	Site Condition (Class/V_{s30})	Earthquake	M_w	Distance (km)	PGA (g)	Earthquake Duration (sec)[1]		Scale Factor
								t_d Earthquake Duration (seconds)/td [Superscript: 1]	D_{a5-95} Earthquake Duration (seconds) /Da5-95	
1										
2										
3										
4										
5										
6										
7										
8										
9										
11										
11										

Notes: t_d = Total Duration, D_{a5-95}=Significant Duration as function of the acceleration time record (time period between 5% and 95% Arias Intensity in a Husid Plot (See, Kempton and Steward, 2006).