12WCEE 2000

SEISMIC DESIGN OF BRIDGES: THE INFLUENCE OF TWO DIMENSIONAL SITE RESPONSE

Geoffrey R MARTIN[1] And Ping QIU[2]

SUMMARY

Many bridge sites, are characterised by two dimensional surface geometry. Subsurface stratigraphy may also be characterised by a two dimensional (2-D) rather than a one dimensional (1-D) geometry. In this paper the magnitude of earthquake ground motion variability that potentially could occur at bridge site through the application of 2-D non-linear site response analysis is illustrated. Potential errors if 1-D soil column analyses are used to evaluate earthquake site response, are also discussed. The site considered is a deep valley viaduct site with fill and rock at opposite abutments and variable alluvial deposits in the valley. The non-linear 2-D dynamic analysis computer code used in analyses incorporates an initial static stress state prior to earthquake loading and a constitutive soil model providing for soil yield and ultimate failure at high earthquake induced stress levels. A non-linear code is also used for 1-D analyses.

Horizontal earthquake ground motions (representative rock records) were input at depth in analyses, through a transmitting boundary. Uncoupled SV (in plane) and SH (out of plane) motions were independently input into the site profiles. Output surface ground motion characteristics were displayed as acceleration time histories and spectra at abutment and pier locations, and snapshot acceleration profiles across the site. Significant variations in amplitude and frequencies and incoherence of motions across the sites were noted. Whereas SH motion input showed similarities to 1-D analyses, SV motions showed large departures from 1-D solutions.

INTRODUCTION

Many bridge sites, particularly those which entail valley crossings, are clearly characterized by two-dimensional surface geometry. In addition, the subsurface stratigraphy generated for example by meandering stream deposition may also be characterized by a two-dimensional (2-D) rather than a one-dimensional (1-D) geometry involving soil lenses and sloping stratigraphy. Where a bridge site has a well-defined 2-D character, the applicability of 1-D site response analyses such as the well-known program SHAKE to evaluate earthquake site response is questionable. The role of 2-D response in generating incoherent ground motions across a bridge site, that is, motion having different amplitude and frequency characteristics at abutment and pier foundation locations, could have significant design implications.

The purpose of this paper is to: 1) illustrate the magnitude of earthquake ground motion incoherence that potentially could occur at a valley crossing bridge site through the application of 2-D non-linear site response analyses and 2) illustrate potential errors if 1-D soil column analyses are used to evaluate earthquake response at such a site.

NON-LINEAR EARTHQUAKE SITE RESPONSE PROGRAMS UTILIZED FOR ANALYSES

[1] *University of Southern California, Fax: 1-213-744-1426*
[2] *URS Greiner-Woodand Clyde, Santa Ana, California, Fax: 1-714-667-7147*

1-D non-linear soil column site response analyses for the case study described below, were conducted using the computer program DESRA-MUSC (Qiu, 1998). This program is a modified version of the program DESRA-2 (Lee and Finn, 1978) and may be used in a total stress or effective stress mode. To characterise non-linear shear stress-shear strain soil behaviour, the program uses the Iwan (1967) kinematic strain hardening model, which in a 1-D form, can be simulated numerically by a mechanistic array of elastic and coulomb sliding elements as shown in Figure 1. Using this model, on initial loading non-linear backbone curve can be constructed in a piecewise linear approximation using established relationships between shear modulus and shear strain amplitude as shown in Figure 2. Such curves form the basis for equivalent linear 1-D response codes such as SHAKE 91 (Idriss and Sun, 1992). The advantage of true non-linear codes, is that they can correctly simulate soil failure under high earthquake shaking levels. The DESRA-MUSC program includes a transmitting base boundary as shown in Figure 3, enabling the use of designated "outcrop" accelerograms as input motions at a defined base boundary of the soil column.

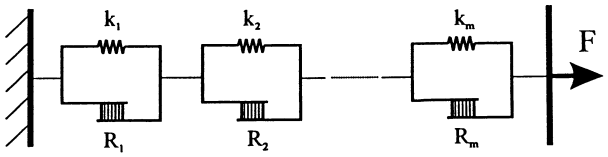

Figure 1. General Features of the Non-Linear Response Program DESRA-MUSC

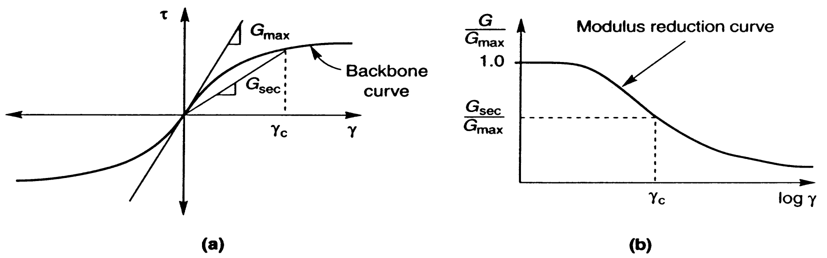

Figure 2. Backbone Curve Showing Variation of G_{sec} with Shear Strain

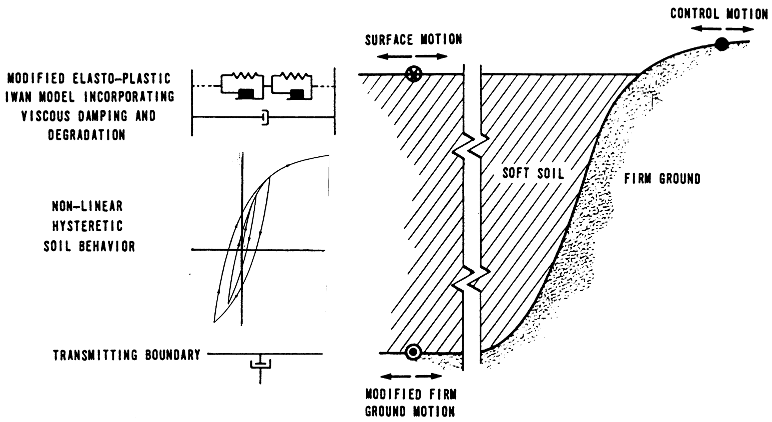

Figure 3. Application of "Outcrop" Motion to Soil Profile

Studies on the influence of two dimensional geometric irregularities of sub-surface soil layers on earthquake site response can be approached using equivalent linear codes such as QUAD4M (Hudson et al., 1994). In these codes, as for the 1-D computer code SHAKE, the non-linear and hysteretic energy dissipation characteristics of soils are simulated by an iterative approach using equivalent elastic and viscous damping models, as a function of shear strain amplitude. However, for large earthquake and softer soil combinations where potential soil failure could occur, and where earthquake induced permanent ground deformations could influence design criteria, true non-linear site response analyses in the time domain are preferable.

Non-linear 2-D site response studies have not been extensive. Joyner and Chen (1975) extended their one dimensional non-linear code based on the Iwan (1967) constitutive model, to study non-linear response of two-dimensional soil structures. This program was subsequently modified by Larkin et . al. (1991) leading to the computer program TENSI, a non-linear two dimensional total stress response analysis program using a multiple yield surface model to simulate non-linear soil behaviour.

A modified version of the program TENSI, developed at the University of Southern California (USC), (called TENSI-MUSC) was used for this research. Modifications included the development of improved constitutive models and the incorporation of initial static stress states to also allow permanent deformation computations. A multiple yield surface elasto-plastic constitutive model (essentially the Iwan model) is used together with a kinematic hardening rule. Von Mise's yield criterion is used for the undrained behaviour of saturated soils and the Drucker-Prager yield criteria is used for unsaturated soils, simulating a pressure sensitive soil. Lagrangian large strain formulation is used in order to analyse potential large cyclic ground motions and permanent deformations. The same shear modulus vs. shear strain amplitude curves used to characterise the non-linear cyclic properties for one-dimensional analysis are used to define non-linear behaviour in shear for the two-dimensional studies. For effective strength analysis, a modified version of the constitutive model used in the 1-D program DESRA-MUSC may be used to simulate progressive pore pressure increases during earthquake loading. In effect, the TENSI-MUSC program is a 2-D extension of the 1-D program DESRA-MUSC. Documentation of the TENSI-MUSC computer program and results of several studies to illustrate applications are described by Qiu (1998). An idealised case study using the program TENSI-MUSC is briefly described below to illustrate the significance of two-dimensional effects on site response.

CASE STUDY: SITE CONDITIONS

In the following case study, an existing (but idealised) California bridge site is used for the analysis as shown in Figure 4. Typical California soft rock is encountered at the right hand side of the valley, while alluvial deposits and fills occur on the left hand side of the valley, overlying the soft rock. The objective of the study was to investigate the differential ground accelerations across the site caused by such an irregular soil configuration and to compare one-dimensional and two dimensional site response solutions. One-dimensional solutions were obtained using the program DESRA-MUSC previously described, at the two soil "column" locations shown on the 2-D profile.

Figure 4 also shows the soil properties used for the analyses. The shear modulus vs. shear strain amplitude curves used to characterise the non-linear soil properties are shown in Figure 5. The soft rock outcrop ground motion used for the analyses is shown in Figure 6. Vertically propagating shear waves are assumed incoming at the rock base line of the profile (a transmitting boundary). Two cases are analysed: (1) motion in the plane of the cross section (SV waves) and (2) motion normal to the cross section (SH waves). The water level is assumed below the dense sand. Hence total stress solutions are evaluated. Qiu (1998) describes solutions for high water levels, where effective stress response including pore pressure increases, are studied.

ANALYSIS RESULTS

Figure 7 show one-dimensional DESRA-MUSC results (ground surface acceleration time histories and response spectral for Columns 1 and 2. The site fill soils significantly attenuate the peak input rock accelerations and high frequency spectra ordinates. Results are somewhat similar for columns 1 and 2, although the loose and weaker sand layer of Column 1 (absent in column 2) attenuates spectral ordinates slightly more. Figure 8 shows representative shear stress-shear strain hysteresis loops for the loose and medium dense sands of column 1 and the medium-dense sand of column 2.

Figure 9 shows peak ground accelerations and ratios of ground accelerations normalised by the peak input motion, together with 5% damped spectra at selected node points, for the SV input wave motion. The distribution of the peak ground accelerations across the mesh are highly nonsymmetric because of the irregular soil deposit. Significant ground motion incoherence clearly exists across the valley. Ground motion has been attenuated by up to 40% when incident waves propagate through fill deposit. Ground motions are similar to 1-D solution at node point 504. However, ground surface accelerations increase significantly as the abutment slope is approached. Wide fluctuations in peak accelerations occur across the valley. The strong attenuation of peak acceleration and high frequency spectral ordinates shown by the one-dimensional solution for Node 390 (in column 2) does not occur in the two-dimensional solution. This is due to the influence of wave reflection and refraction at nearby sloping boundaries and increased strength of the medium dense sand due to transient increased confining pressures. On the rock abutment, the accelerations amplify due to wave reflections from the slope. When the nodes approach the vertical boundary, the peak ground acceleration ratio to the input motion approaches 1.0, because one-dimensional free-field boundary conditions are applied at the vertical boundaries. The snapshot of ground accelerations when accelerations peak on the rock abutment, is indicative of the ground motion incoherence. Figure 10 shows representative shear stress- shear strain hysteresis loops for the loose and medium dense sands of column 1 and the medium dense sands of column 2 for the SV wave solution. Note the permanent yield strains occurring on first loading due to the initial static shear stresses acting in the soil elements. Such strains lead to a post earthquake relative displacement between node points 387 and 518 of 0.8 ft. and between node points 517 and 518 of 0.4 ft. Figures 11 and 12 show similar data as in Figures 9 and 10, but for the SH input wave motion. Note for the SH solution, static shear stresses are absent on horizontal planes, and hence permanent deformations are minimal. Peak accelerations and spectral ordinates show more attenuation than those for SV input, particularly in the valley, and do not show the variability across the site due to the lack of reflection and refraction at sloping boundaries. The increase in acceleration level at node point 517 at the fill abutment locations characterising the SV solution, do not occur for the SH solution. Overall, the SH solutions are more similar to the ID solutions shown in Figures 7 and 8.

CONCUSIONS

For deep valley sites characterised by non uniform site stratigraphy, the 2-D response analyses conducted in this study, clearly indicate the significance of 2-D site response on the nature of ground motion input to seismic analyses of bridge structures with pier foundations at different locations across the valley. Significant differences between SV (in plane) and SH (out of plane) response could be expected, with the latter more closely resembling solutions which might be obtained from 1-D column site response analyses. However, the impact on bridge structure design from such input motion incoherence generated by 2-D conditions, can best be evaluated through dynamic sensitivity analyses using structural computer codes.

ACKNOWLEDGEMENTS

The research described in this paper was funded by a Federal Highway Administration grant (Contract DTFH61-92-6-00112) to the National Center of Earthquake Engineering, Buffalo. This support is gratefully acknowledged.

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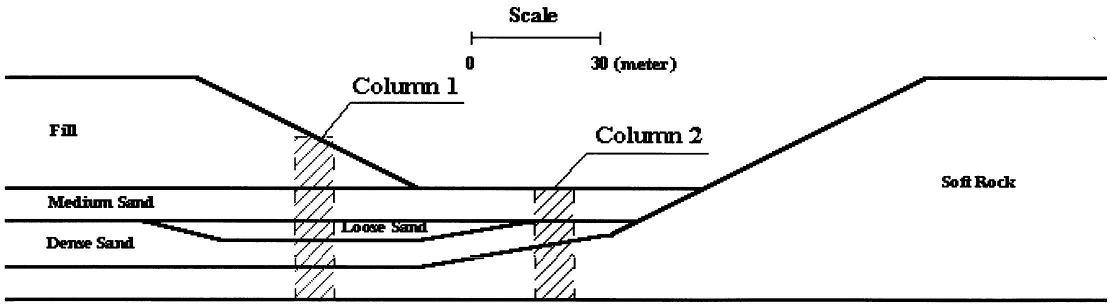


Layer	Unit Weight (kN/m^3)	Cohesion (kPa)	Friction Angle	Blow Count $(N_1)_{60}$ - blow/300 mm	Shear Wave Velocity (m/ft)	Poisson's Ratio
Fill	16.0	5	30	16	270	0.35
Loose Sand	15.2	N.A.	30	8	270	0.30
Medium Dense Sand	17.6	N.A.	35	16	300	0.33
Dense Sand	20.8	N.A.	40	30	340	0.35
Soft Rock	22.4	55000	N.A.	N.A.	760	0.35

Figure 4. Soil Profile and Material Properties used for 2-D Analysis

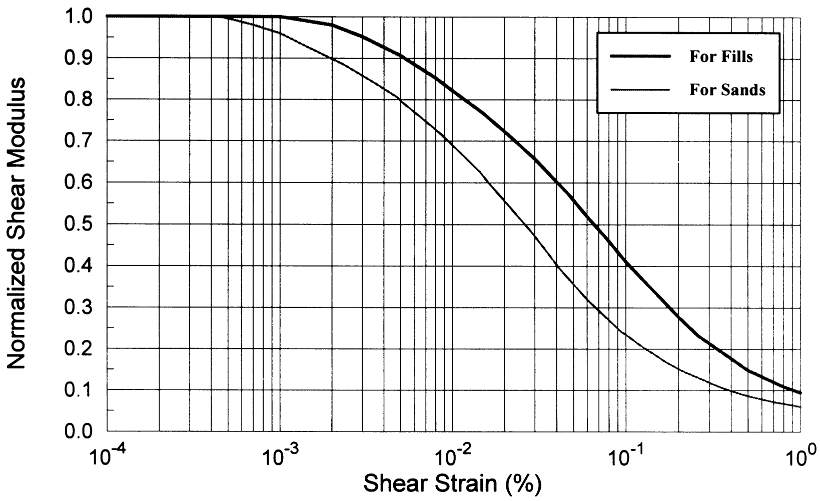


Figure 5. Shear Modulus vs. Shear Strain Amplitude Curves

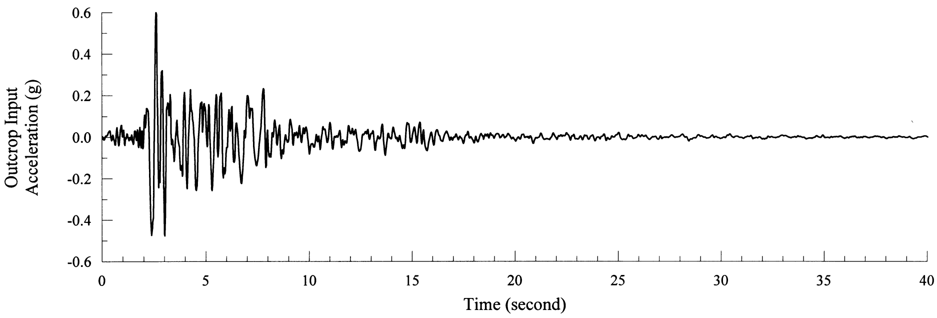


Figure 6. Input Accelerogram used for Analyses

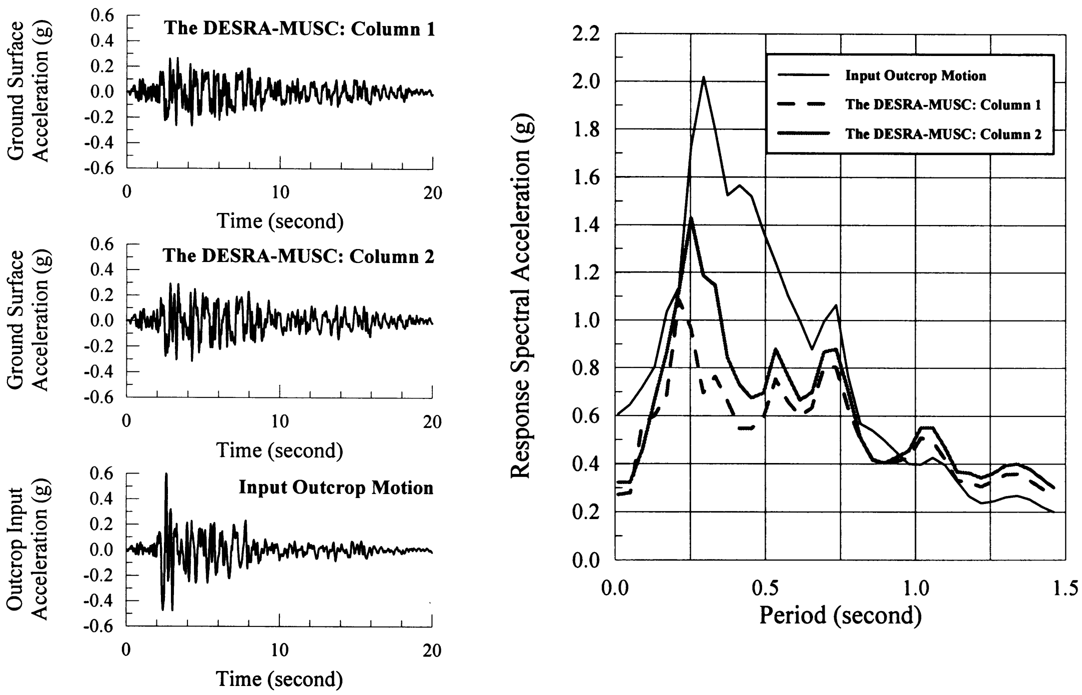
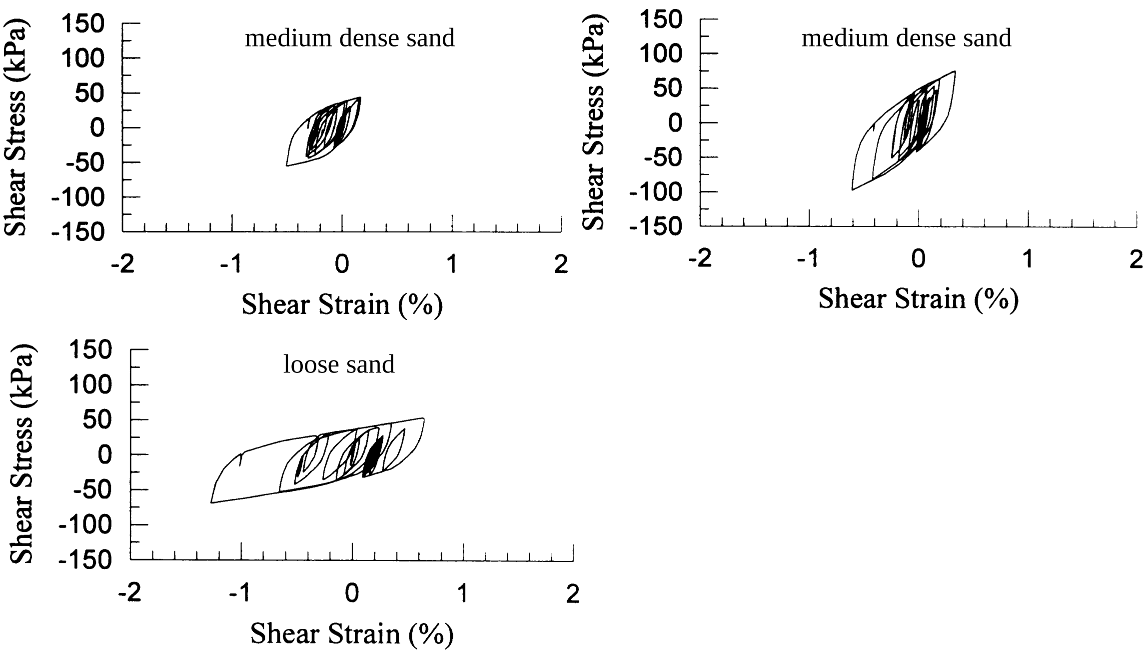


Figure 7. DESRA-MUSC Solutions: Ground Surface Response



Column 1 **Column 2**

Figure 8. DESRA-MUSC Solutions: Hysteresis Loops

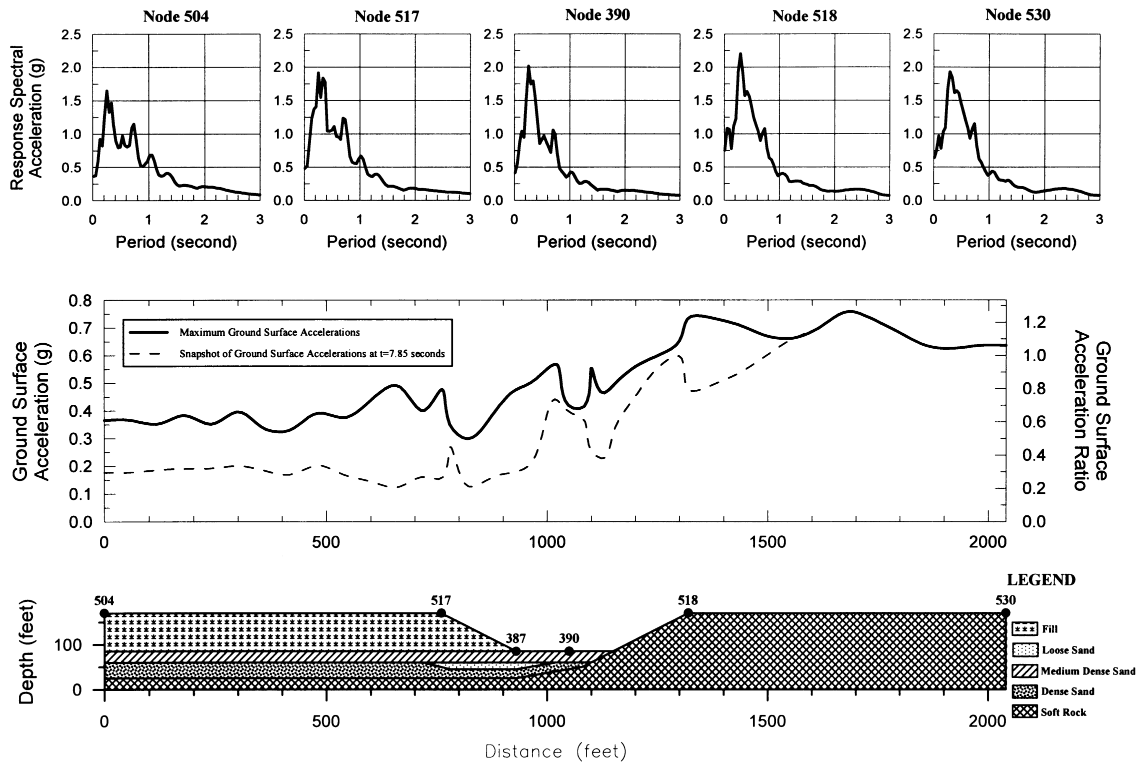


Figure 9. Ground Surface Accelerations from TENSI-MUSC Total Stress Analysis - SV Wave Propagation

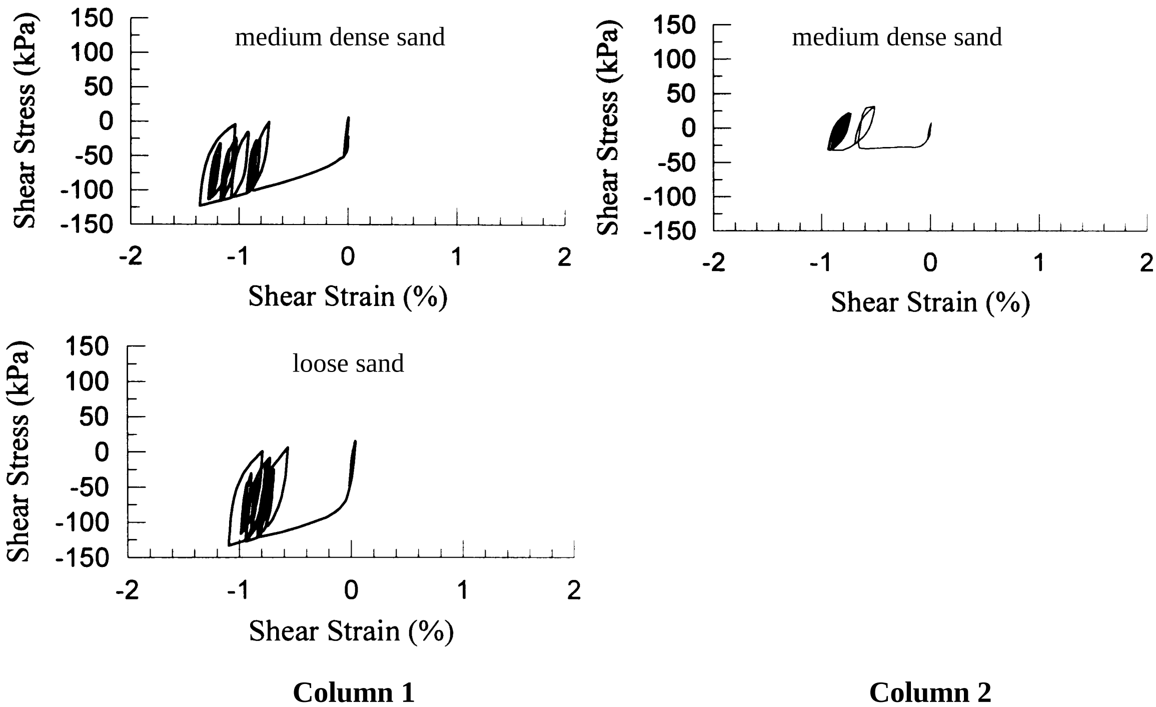


Figure 10. TENSI-MUSC Solutions: Hysteresis Loops - SV Wave Input

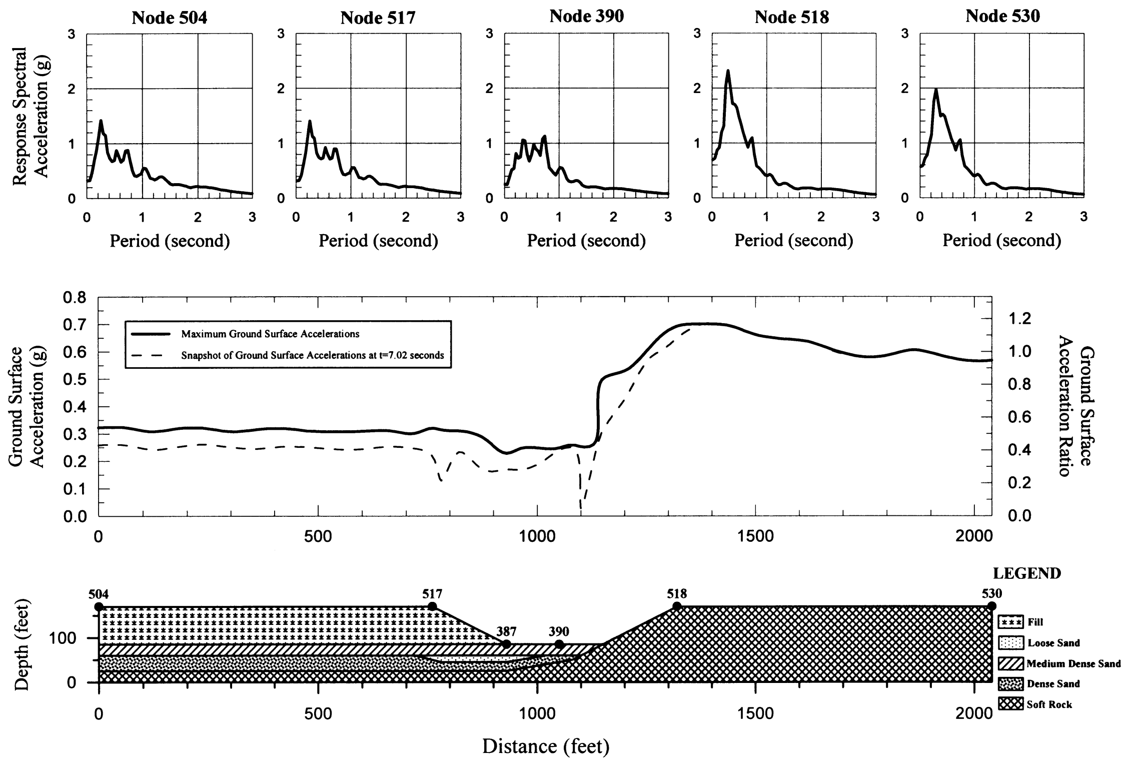


Figure 11. Ground Surface Accelerations from TENSI-MUSC Total Stress Analysis – SH Wave Propagation

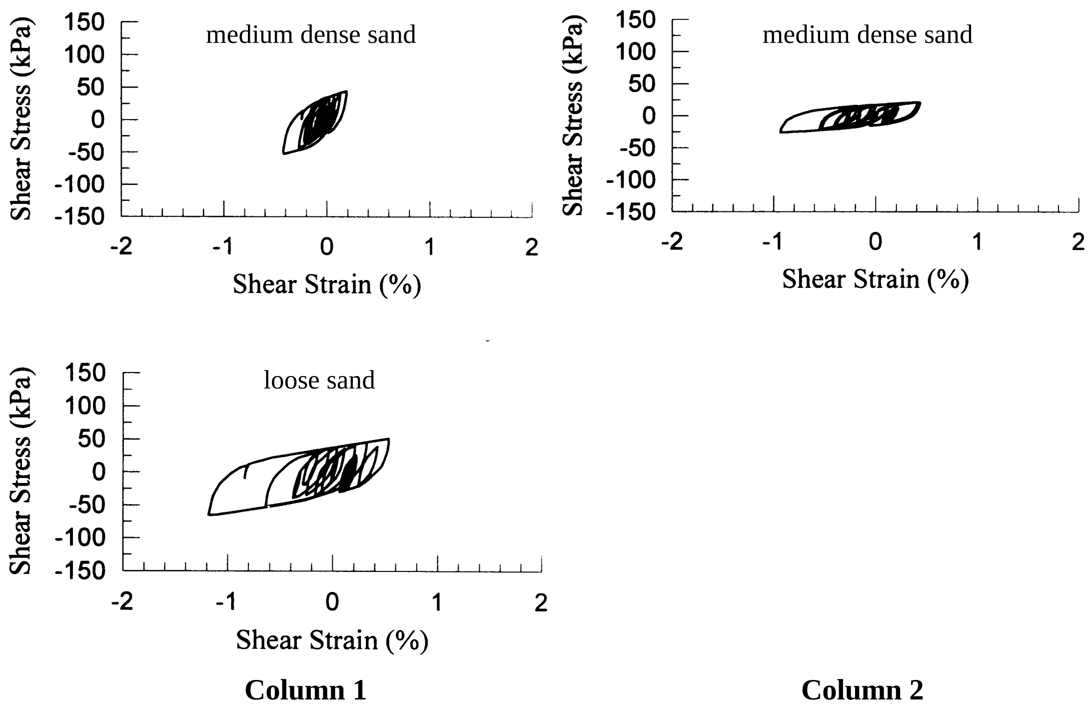


Figure 12. TENSI-MUSC Solutions: Hysteresis Loops – SH Wave Input