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FREQUENCY DOMAIN SITE RESPONSE ANALYSIS WITH PORE PRESSURE SOFTENING

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SUMMARY

A frequency domain analysis for seismic site response and liquefaction potential which considers the influence of pore pressure generation on response is developed using the concept of an effective pore pressure. The effective pore pressure is a weighted average of the pore pressure during the earthquake. When calculating soil moduli in the iterative equivalent-linear response analysis, the initial effective confining pressure is reduced by the effective pore pressure to account for the softening due to pore pressure development. Results of analyses are presented to illustrate use of the model.

INTRODUCTION

A probabilistic anaysis has been developed for seismic site response and pore pressure generation in layered horizontal soil deposits based upon non-stationary random vibration theory (Ref. 1). The three major components of the analysis are the non-stationary random vibration model for earthquake ground motions, the probabilistic equivalent-linear analysis for site response, and the probabilistic pore pressure generation model to calculate the effective pore pressure and liquefaction potential.

A simple, two-parameter frequency independent amplitude-modulating function is introduced to describe the non-stationarity in ground motion intensity content. Frequency content in the form of a normalized unit-area Power Spectral Density (PSD) function is characterized in a sectionally stationary manner.

The seismic excitation is assumed to consist of one dimensional vertically propagating shear waves. A random vibration analysis produces time-dependent PSD functions for response at any given depth within the soil profile. Results from the response analysis provide input to the pore pressure generation model. An effective pore pressure is calculated for use in subsequent iterations of the equivalent-linear visco-elastic response analysis.

Results are presented in the form of the variation of the statistical parameters for acceleration, shear strain, shear stress and pore pressure with time and with depth. Liquefaction fragility curves are also developed. The influence of pore pressure softening on seismic site response is illustrated by the results.

NON-STATIONARY GROUND MOTION CHARACTERIZATION

Ground motion intensity is characterized by a deterministic amplitude modulating function and a stationary random process. The stationary process is assigned a unit area so that the expected energy, or time averaged root mean square (RMS) of the acceleration, describes the amplitude of the non-stationary ground motions. The distribution of the amplitude peaks is assumed to conform to a Rayleigh distribution. A two parameter trigonometric function based upon analysis of records from previous earthquakes is used to describe the shape of the RMS time history. The Trifunac-Brady definiton of duration is used, wherein strong shaking is assumed to start when 5 percent of the total energy in the record has arrived at the site and end when 95 percent of the total energy has arrived (Ref. 2).

The magnitude of the non-stationary modulating function is related to stationary RMS so that existing RMS attenuation relationships may be used (Ref. 3). Figure 1 compares the non-stationary intensity function to stationary RMS for a record from the 1971 San Fernando earthquake. Figure 2 compares measured and fitted energy time-histories to the stationary assumption for this same record.

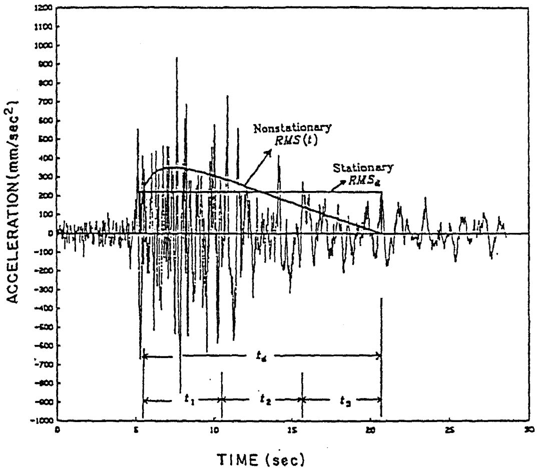

Fig. 1 Stationary and Non-Stationary RMS

Statistical analyses were performed on 122 strong ground motion records to develop distributions for the two intensity shape parameters. Distributions were fit to the two parameters and statistics on the mean and variance are calculated for all 122 sites together and for 74 soil sites and 48 rock sites separately to investigate the influence of local soil conditions. The statistics show that the rate of energy release at soil sites is more uniform (closer to stationary) than at rock sites. Potential correlations of the shape parameters with earthquake magnitude, epicentral distance, peak ground acceleration, duration and stationary RMS were also investigated. Scattergrams showed wide variability and little correlation of the data, except possibly some weak correlations with epicentral distance and duration. Consequently, except for the dependence on site conditions, the two intensity parameters were treated as independent random variables.

Frequency content of the earthquake excitation is described by the two-parameter Tajimi-Kanai (T-K) PSD function. A sectionally stationary process is assumed and the unit T-K function is developed for each of three equal duration sections of the strong motion record. Figure 3 presents the probability density functions (PDF) for the central frequency of the T-K functions fitted to the three duration intervals. This figure demonstrates the evaluation of frequency content with time. The frequency parameters from 80 of the 122 records analyzed in the study of the intensity parameters were analyzed statistically. Histograms were also generated for 50 soil sites and 30 rock sites separately. Results show a much lower central frequency for soil sites than for rock sites and a decrease in central frequency with time for both generic site conditions. The shape factor, or "damping" parameter, of the PSD function appeared to depend only marginally on time and site conditions, and may reasonably be assumed constant.

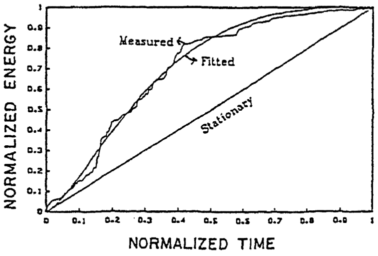

Fig. 2 Energy Time History

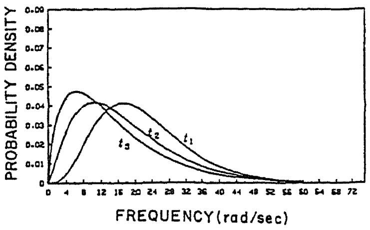

Fig. 3 PDF's for Central Frequency

SITE RESPONSE ANALYSIS

Both impulse and frequency response transfer functions for a horizontally layered visco-elastic soil deposit subject to vertically propagating shear waves were developed. A "complete" analysis using the impulse response method to include the transient free vibration response characteristics of the site, and a cost-effective "simplified" steady state response analysis using the frequency response method and assuming stationarity in frequency content were developed. Both analyses produce time-dependent PSD functions for the response parameters at any given depth.

SOIL RESPONSE CHARACTERIZATION

An iterative, equivalent-linear approach is used to account for non-linearities in soil response due to strain and pore pressure development. The effective shear strain is used to account for strain softening and an effective pore pressure development. The effective shear strain is used to account for strain softening and an effective pore pressure is introduced to account for softening due to pore pressure development by reducing the initial effective confining pressure. The effective shear strain and effective confining pressure are used to estimate soil modulus and damping from "equivalent-linear" modulus and damping relationship.

Pore pressure development is accounted for using a stress-ratio dependent pore pressure generation model developed by Wang and Kavazanjian (Ref. 4) to account for non-uniform cyclic loading. A stress-ratio independent pore pressure generation model developed by Chameau and Clough (Ref. 5) that uses the stationary RMS and the number of positive zero crossings as the loading parameters was modified to include non-stationary RMS and stress-ratio dependence. The non-stationary seismic loading is assumed to be a Gaussian narrow-band and locally stationary process to determine the time-dependent expected energy and number of zero crossings for input to the pore pressure model. Seismic resistance is defined by the distribution of cyclic shear strength and a set of deterministic stress-ratio dependent normalized pore pressure generation curves. The pore pressure development model yields the time-dependent cumulative distribution of excess pore pressure. The expected pore pressure is calculated using a weighted averaging method based upon the expected incremental pore pressure for each cycle of loading.

RESULTS FROM THE MODEL

Results can be presented in the form of the variation of the statistical parameters for acceleration, shear strain, shear stress, and pore pressure with time and depth. Liquefaction potential can be expressed by a set of seismic fragility curves, where the probability of liquefaction at any depth within the given soil profile is plotted as a function of the RMS and duration of the earthquake excitation.

A series of four site response analyses were performed to illustrate application of the model. Three analayses on hypothetical sites were developed to demonstrate the influence of pore pressure development and to explore the differences between the complete and simplified formulations. A fourth analysis was performed to develop liquefaction fragility curves for a pore pressure monitoring station to demonstrate application of the model to a case history and provide predictions for future events.

Results of the hypothetical analyses illustrate the sensitivity of predicted shear strains and soil moduli to the effect of pore pressure generation. Figure 4 compares results of an analysis in which pore pressure softening is considered to one in which softening is supressed for a 100-foot thick deposit of Monterey No. 0 sand at a relative density of 53 percent. Shear stresses are, in general, predicted reasonably accurately even when softening due to pore pressure generation is neglected. However, neglecting softening reduces shear strains and increases liquefaction potential. A primary influence of softening on response was a change in frequency content of the motion. The increase in liquefaction potential when softening is neglected may be attributable to the higher frequency response of the stiffer site, which results in more cycles of loading.

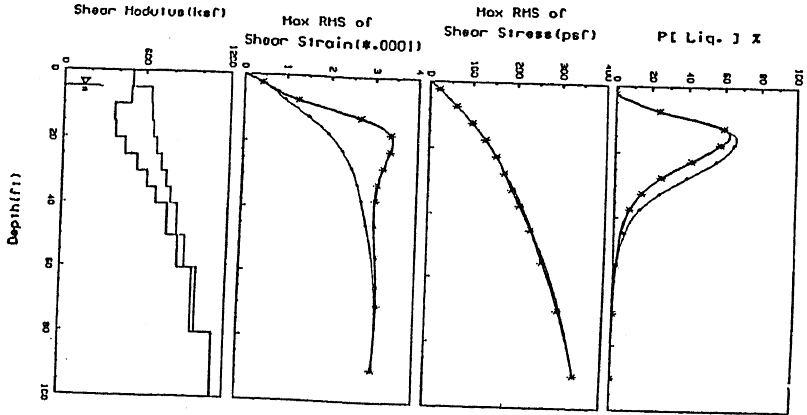

Fig. 4 Influence of Pore Pressure Softening

Analyses on the hypothetical sites indicate the economical simplified method provides excellent results for many situations. Fig. 5 compares pore pressure response from the simplified formulation to results from the complete formulation for a 100-foot thick deposit of dense sand.

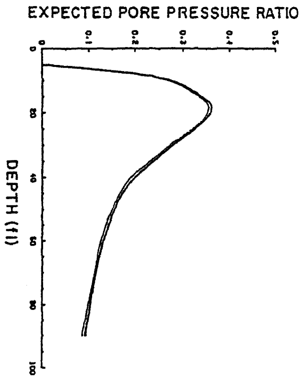

Fig. 5 Pore Pressure Response From Simplified and Complete Model

Predictions of liquefaction potential were made for the U.S.G.S. "Wildlife" permanent pore pressure monitoring station in the Imperial Valley. Fig. 6 presents the fragility curves developed for this site, giving probability of liquefaction as a function of stationary RMS and duration. These fragility

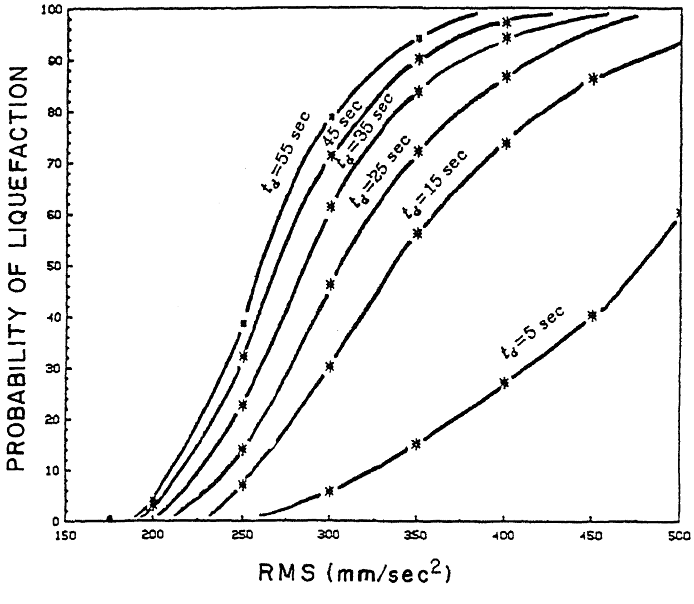

Fig. 6 Fragility Curves for Wildlife Monitoring Station

curves are consistent with response of the site to two recent earthquakes. No evidence of liquefaction was observed at the site following a distant magnitude 6.6 earthquake in October, 1979 of duration 27 seconds and RMS of 177 mm/sec^2. After a nearby magnitude 5.6 event in April, 1981 with estimated RMS equal to 400 mm/sec^2 and duration of 13 seconds at the site, numerous sand boils and ground fissures were observed. From the fragility curves, the probability of liquefaction is about zero for the 1979 event and approximately 0.7 for the 1981 event.

CONCLUSIONS

Comparisons with recorded strong ground motions demonstrate that the non-stationary characterization of the excitation developed herein is, in general, a more realistic representation of the earthquake excitation than the conventional stationary characterization. A frequency domain site response analysis which incorporates pore pressure softening has been developed to accommodate the non-stationary loading model. Results of analyses demonstrate that pore pressure softening can significantly influence seismic site response and liquefaction potential. While the magnitude of seismic shear stress was not significantly influenced by pore pressure softening, the resulting shear strains were very sensitive to seismically induced pore pressure. This sensitivity indicates that stress-history based site response analyses may be more reliable than strain-history based analyses. The fragility curves provided by the model are useful tools for a risk-based approach to evaluation of liquefaction potential. Complete details of model development and application are presented by Wang and Kavazanjian (Ref. 1).

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