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ESTIMATION OF UNCERTAINTIES IN THE DYNAMIC RESPONSE OF URBAN SOILS IN JAPAN.

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ABSTRACT

One of the major sources of uncertainty in design seismic load is the non-linear response of site soil deposits. Soil response uncertainty may dominate the seismic load uncertainties when it is large compared with other sources uncertainties. For code zoning, only a sparse soil data is available, while for microzonation studies or site-specific studies, data quality and quantity are considerably enhanced. This study is carried out to investigate the inherent uncertainties in soil response with different levels of information. It is found that while variability of soil characteristics results in a dominant amplification uncertainty, the latter can be decreased about 40% when additional information is collected for soil characteristics. For an example site in Tokyo, when enough information is collected for soil data, the overall uncertainty, expressed in terms of cov, can be around 30%, 20% for PGA and spectrum intensity respectively. For spectrum values average amplification, it can be as high as 50% around the layers natural period.

KEYWORDS

Uncertainty analysis; Soil amplification; Soil response uncertainty; Soil characteristics uncertainty; Soil non-linear response; Hysteresis model; Ramberg-Osgood model; Seismic zonation; Peak Ground Acceleration; Acceleration response spectrum; Velocity response spectrum.

INTRODUCTION

Uncertainty of soil amplification can be classified into modeling uncertainty, variabilities due to non-stationarity of input motion, and variabilities in soil characteristics. Soil response is inherently non-linear due to non-linear characteristics of soil deposits. Among several available methods for the nonlinear response analysis of soil, 1D equivalent-linear method and lumped mass model with hysteresis characteristics method are selected here for uncertainty study due to their wide practical impact as well as many other advantages (Brad 1994). Thus, here, the calculation of the amplification is based on the assumption that soil layers are horizontal and seismic waves propagate vertically. Due to the complexity of soil amplification function in the non-linear range, Monte-Carlo simulation is used to represent the variability of soil characteristics. Analysis is carried out for general sites configurations as well as a specific site with different levels of information qualities. Results for general configuration can be utilized for regional seismic zonation while these of a

DATA AND METHODS

Input Parameters

The analysis is carried out with simulated ground motions from the spectral shape shown in figure 1-a which was developed for the design spectrum at base layer for the elastic limit criterion of tall buildings (Building Research Institute 1991). One simulated ground motion is a superposition of sinusoidal waves with amplitude related to the spectral density function and a random phase angle. It is then scaled to a deterministic envelope function (Jennings *et al.* 1969). The simulated ground motions are not smoothed nor scaled so that the phase characteristics are maintained. Soil layers data are taken from two sources with different qualities. The first is the general configuration, N-value, and thickness of soil deposits in alluvial plains in Japan (Ohsaki *et al.* 1973) as shown in Fig. 1-b. Such data is called level 1 hereafter. The second is the results of boreholes and seismic investigations of different sites in Kanto plain. For the mechanical characteristics, i.e., relations between strain and each of shear modulus and damping ratio, empirical relations (Imazu *et al.* 1986) are considered here. In these relations, the mechanical parameters are expressed as follows:

$$\frac{G}{G_0} = \frac{1}{1 + a \bullet \gamma^b} \qquad (1)$$

$$h = c \bullet \gamma^d \qquad (2)$$

Where G/Go is the ratio between the shear modulus, G, at any strain, γ, to the initial value, Go, and, h, is the damping ratio. Two relationships between N-value, which comes from standard penetration test, and shear modulus Go are used in this analysis (Ohsaki *et al.* 1973, Ahmed 1996). These relations are expressed in the following general form:

$$Go = e \bullet N^f \qquad (3)$$

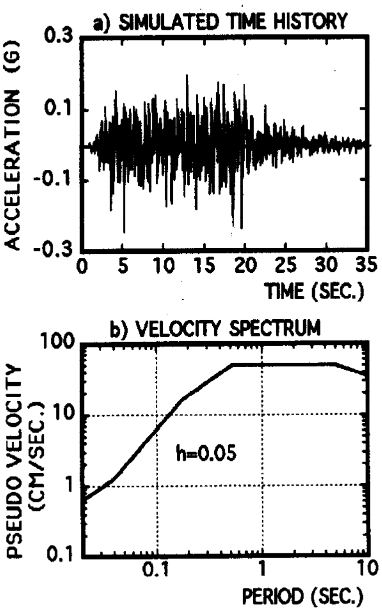


Fig. 1-a. Velocity spectrum (Building Res. Inst. 1991) and simulated motion at base layer.

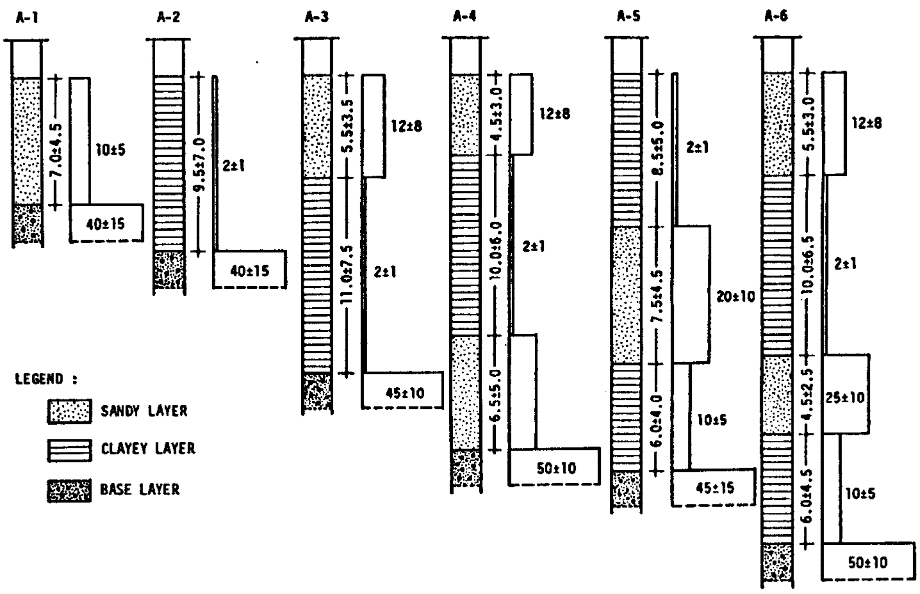


Fig. 1-b. Thickness (in m) and N-value of layers for various soil deposits in alluvial plains (Ohsaki *et al.* 1973).

Considered Output Soil Response Parameters

Three output parameters are selected for the study of uncertainty due to different analysis methods. These parameters are the predominant period T, surface peak ground acceleration PGA and surface strain γ. For the other two types of uncertainties, several output parameters are selected as follows:

1. Predominant frequency of the site (pre-strain frequency).

4. Amplification of the acceleration response spectrum intensity, SAa, which is defined here as the area under acceleration spectrum for the period of 0.1 to 2.5 seconds and damping ratio 5%.
5. Amplification of the velocity response spectrum intensity, SVa, which is defined as the area under velocity spectrum for the period of 0.1 to 2.5 seconds and damping ratio 5%. This area is almost equal to the area under pseudo velocity response spectrum which was originally used (Housner 1975) to define the spectrum intensity.
6. Acceleration (and velocity) spectral values average amplification which is defined as the average amplification in terms of response spectrum for the following four period ranges; 0.3-0.6, 0.6-1.2, 1.2-2.4, and 2.4-4.8 seconds. Such periods are selected to represent the ranges of periods important for different type of structures. Hereafter, the amplification averages are called SXi, where X=A and V for acceleration and velocity, respectively, and i=1,2,3,4 for the pre-mentioned four period ranges respectively.

Nonlinear Analysis Methods

One Dimensional Wave Propagation Equivalent Linear Method. This method is based on the continuos solution to the wave equation. Non-linearity of shear modulus and damping ratio is accounted for by an iterative procedure to obtain values for modulus and damping compatible with the effective strain in each layer (Schnabel *et al.* 1972).

Lumped Mass Model With Hysteresis Characteristics. This method can be adopted for soil analysis if the soil layers can be simplified as lumped masses. The hysteresis characteristics of such lumped mass can be generally modeled with Ramberg-Osgood hysteresis model (Osgood *et al.* 1943, Jennings 1963). The skeleton curve is usually expressed by Masing's law. The initial loading curve is written in terms of the shear strain, γ, yield shear strain, γ_y, shear stress, τ, yield shear stress, τ_y, and the model parameters α and β as:

$$\frac{\gamma}{\gamma_y} = \frac{\tau}{\tau_y} \pm \alpha \left|\frac{\tau}{\tau_y}\right|^{\beta} \tag{4}$$

Experimental results and empirical relations between strain and each of shear modulus and damping show some deviation from this hysteresis model. To accommodate such empirical relations, Ohsaki-Hara hysteresis model was proposed (Ohsaki *et al.* 1977). When such model is used, some differences still exist between the mechanical characteristics estimated from such model and the one estimated from the statistics of experimental results. A modification for Ramberg-Osgood hysteresis model is proposed here to accommodate the empirical relation of mechanical models. From the initial loading curve of Ramberg-Osgood model, G and h can be expressed as follows:

$$\frac{G}{G_0} = \frac{1}{1+\alpha\left(p/p_y\right)^{\beta-1}} \tag{5}$$

$$h = \frac{2}{\pi}\frac{\beta}{\beta+2}\left(1-\frac{G}{G_0}\right) \tag{6}$$

Where p and p_y are shear force and yield shear force respectively. Then skeleton curve parameters α , β, γ_y can be estimated by minimizing the following error function:

$$\text{Error} = \sum_{\gamma}\left[\left(\frac{G_i}{G_0}-\frac{G}{G_0}\right)^2+(h_i-h)^2\right] \tag{7}$$

Where G_i, h_i, G, and h are the values given by equations (1) and (2), (5), and (6), respectively. In this modified model, skeleton curve parameters α, β are estimated by minimizing equation (7) and, then, used for initial loading. At each reverse point, new α, β are estimated such that h value calculated from equation (2)

UNCERTAINTY DUE TO ANALYTICAL METHOD

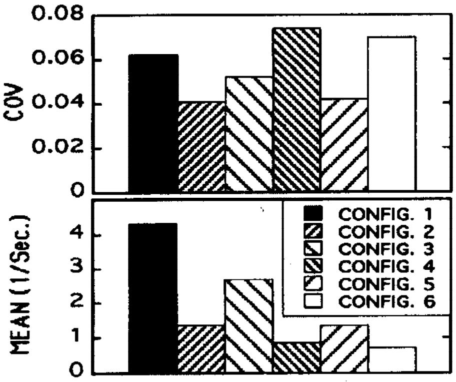
Three analytical methods are applied using six general configurations. Soil characteristics of these configurations are taken from the mean values shown in Fig. 1-b. In the first method, one dimensional wave propagation theory with equivalent linear model is used. In the second one, a simplified lumped mass model with the modified R-O hysteresis model is used. In the third one, the same simplified lumped mass model with Ohsaki-Hara hysteresis model is used. Results are shown in table 1. It can be seen from the table that the first two methods give similar results. In the last method, Ohsaki-Hara model overestimated the damping at the strain levels relevant to the analysis and consequently, resulted in generally lower PGA values at surface. Thus, when the same mechanical model is used, results are similar regardless of the used method. Nevertheless, 1D equivalent linear method gives slightly larger values for PGA in most cases. Although only three methods are not enough for statistical study of the response variation due to the used model, the results indicate that PGA amplification uncertainty due to the analytical method is around 15% in terms of cov. Moreover the first method can be considered adequate for the study of the remaining uncertainties.

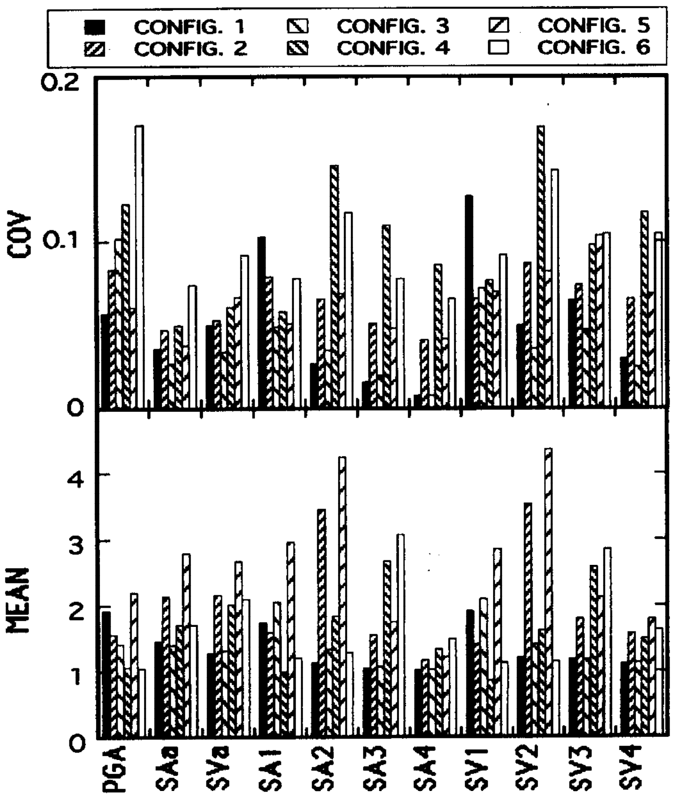
Table 1: Comparison between results of the analysis using different methods.

Soil config.	parameter	method 1	method 2	method 3
1	T (Sec.)	0.14	0.15	0.15
	PGA (Gals)	521	-----	279
	γ	0.09	-----	0.17
2	T (Sec.)	0.37	0.37	0.37
	PGA (Gals)	414	423	277
	γ	0.07	0.10	0.06
3	T (Sec.)	0.48	0.59	0.59
	PGA (Gals)	299	266	237
	γ	0.03	0.03	0.17
4	T (Sec.)	0.54	0.57	0.57
	PGA (Gals)	288	257	228
	γ	0.02	0.02	0.09
5	T (Sec.)	0.54	0.44	0.44
	PGA (Gals)	611	500	372
	γ	0.15	0.17	0.14
6	T (Sec.)	0.61	0.64	0.64
	PGA (Gals)	360	265	281
	γ	0.04	0.02	0.16

UNCERTAINTY DUE TO PHASE CONTENT OF GROUND MOTION

Ten simulated ground motions from the spectral shape shown in figure 1-a, are used to study the variability due to phase content of the ground motion. 1D equivalent linear analysis is carried out using the general six configurations shown in Fig. 1. Results of the mean and cov for different configurations are shown in Figs. 2 and 3.





For different period ranges, max. mean and cov values of SXi usually fall within the period range that include the predominant period of the configuration. In all configurations, SAi cov values are generally lower than those of SVi. Values of cov due to variation of random phase angle are less than 17% for all configurations and response parameters. Since PGA was not fixed for each simulated motion, the analysis is carried out with PGA in the range of 230 to 300 Gals. Thus, estimated cov values include also the uncertainty due to the variation of PGA in this range.

UNCERTAINTY DUE TO SOIL CHARACTERISTICS

Mean and cov values of some soil input parameters are collected from literature, calculated or assumed. Then the parameters with uncertain nature that are believed to affect the soil response are considered as probabilistic parameters. These parameters are assumed to be independent random variables with log-normal distributions. Monte-Carlo technique is then used to simulated independent 1000 sets of these parameters. Mechanical models vary according to site and depth even for the same soil type (Imazu *et al.* 1986). Mechanical model parameters, such as a, b, c, and d that are given in equations (1) and (2), are assumed to be independent random variables although all these parameters are actually correlated. Moreover, a full dependence between G/Go values at different strain levels is assumed since they are always calculated through equation (1) using one pair of the simulated a and b parameters. The same is applied for the damping ratios at different strain levels. Thus, the results here may be on the conservative side, i.e., cov values are increased due to this independent treatment of the parameters and the assumed dependence of the values at different strains. Nevertheless, since the same relations and assumptions are used in the three studied levels, relative values are not affected. The used analysis method is 1D wave propagation equivalent linear analysis.

Analysis For General Site Configurations (Level 1)

Response analysis is carried out using the general six configurations shown in Fig. 1-b. The following parameters are considered:

1. Deterministic parameters such as input ground motion, ratio of effective strain which was taken 0.65, number and arrangement of layers, analysis method, f coefficient in Go-N relationship.
2. Probabilistic parameters such as shown in table 2.

Table 2: Probabilistic parameters considered for level 1.

Parameter	mean data source	cov data source
N-value	(Ohsaki *et al.* 1973)	(Ohsaki *et al.* 1973)
Layer thickness	(Ohsaki *et al.* 1973)	(Ohsaki *et al.* 1973)
Unit weight (ton/m^3)	Assumed as 1.85 for sand and 1.5 for clay	Assumed 15%
mechanical model parameters a, b, c, d	(Imazu *et al.* 1986)	(Imazu *et al.* 1986)
Go-N relation parameter e	(Ohsaki *et al.* 1973)	Assumed 15%

Results of the mean and cov for different configuration are shown in Figs. 4 and 5.

If a single site at Tokyo is selected for which only the geological information is available, it can be approximately classified as configuration 4. In such configuration, the diluvium base layer is followed by a mixture of different soil types but mainly sandy soil, clayey soil, sandy soil mixed with some clay layers. This site is considered here as an example for level 1 information for which the results of cov values are compared with those of other information levels in the following sections.

Analysis For Specific Site Configuration (Level 2)

For a site at Tokyo, it is assumed that a single borehole is available. If a project is to be constructed on this site, then, for the area around the borehole site, the soil parameters can be estimated from this borehole data.

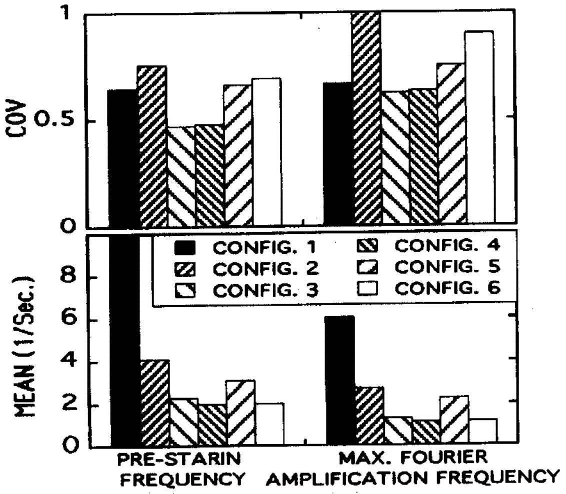


Fig. 5. Frequencies mean and cov due to variation of soil characteristics for different soil configurations with level 1 information.

Table 3: Probabilistic parameters considered for level 2.

Parameter	mean source	cov source
N-value	Actual data	Assumed 20%
Layer thickness	Actual data	Assumed 20%
Unit weight (ton/m^3)	Actual data	Assumed 10%
mechanical model parameters a, b, c, d	(Imazu *et al.* 1986)	(Imazu *et al.* 1986)
Go-N relation parameter e	Kanto data (Ahmed 1996)	Kanto data (Ahmed 1996)

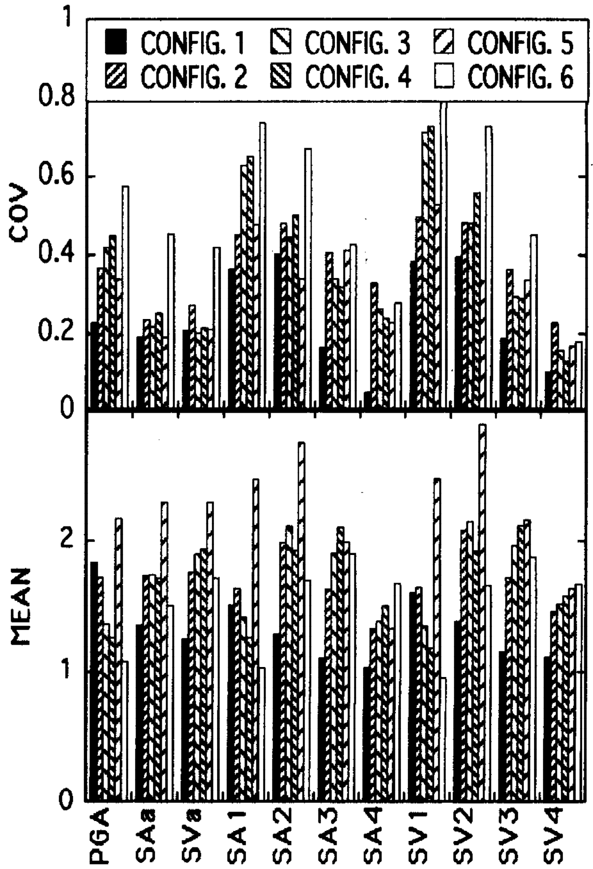


Fig. 4. Amplification mean and cov for soil response due to variation of soil characteristics (level 1 information.)

For such simplification, a criterion based on the change of unit weight, soil type, and N- value are utilized (Ohsaki *et al.* 1981). In such case, two types of parameters can be considered as follows; deterministic parameters such as in level 1 and probabilistic parameters such as shown in table 3.

Analysis For Specific Site Configuration (Level 3)

For the site used in level 2, it is assumed that several boreholes are available for the project site. Also, the seismic investigation of the site is assumed to be carried out so that the value of shear wave velocity can be accurately determined. Thus, uncertainty due to the spatial variation of soil characteristics as well as due to the simplification of the soil layers can be decreased. In such case, two types of parameters can be considered as follows:

1. Deterministic parameters such as in level 1 as well as the shear wave velocity values.
2. Probabilistic parameters such as unit weight, mechanical model parameters, and layer thickness. The first two are taken similar to level 2 while layer thickness is taken from actual data with assumed cov =10%.

Response analysis is carried out for the same site when three levels of information are available. Results of the mean and cov for the three levels are shown in Figs. 6 and 7.

In general, SAi cov values are generally lower than those of SVi. For different period ranges, max. mean and cov values of SXi usually fall within the period range that include the predominant period of the configuration. It is interesting to notice the immigration of SAi maximum mean and cov from SA1 to SA2 as the predominant period moves in the same direction when more information is introduced. Dynamic excitation decreases the dominant frequencies mean and increases their cov for the three levels. However, the relative information quality. The reduction of cov is very significant for most response parameters. However, some parameters uncertainty does not decrease from level 1 to 2 such as velocity and acceleration spectrum intensities. If these parameters are the main interest of the designer, then more data is needed to decrease the cov values as shown for level 3 results.

For this example site, amplification uncertainty due to different analytical methods is estimated from table 1 as, approximately, 11% in terms of PGA cov. Amplifications of spectrum intensities are found to have lower cov values due to different analytical methods. Amplification uncertainty due to the phase content is estimated as 12%, 5% and 6% for PGA, SAa, and SVa, respectively, as shown in figure 2. Amplification uncertainty due to the varaibilities in soil characteristics is estimated in terms of cov, as the values in the range 30% to 45%, 19% to 30%, and 18% to 28% for PGA, SAa, and SVa, respectively, as shown in figure 6. The overall soil amplification uncertainty, expressed in terms of cov, is estimated with square root of the sum of square cov values due to the pre-mentioned three uncertainty sources as shown in figure 8. From this figure and figure 6, it can be shown that while variability of soil characteristics results in a dominant amplification uncertainty, such variability and its effect can be considerably decreased when additional information is introduced. About 40% reduction in the overall cov can be achieved as more information is collected. Reduction is larger for SXi cov at short periods.

The whole analysis is carried out with simulated ground motion of which PGA is in the range of 230 to 300 Gals. For larger PGA values, uncertainties might be different due to the effect of non-linearity on the amplification ratio for large PGA.

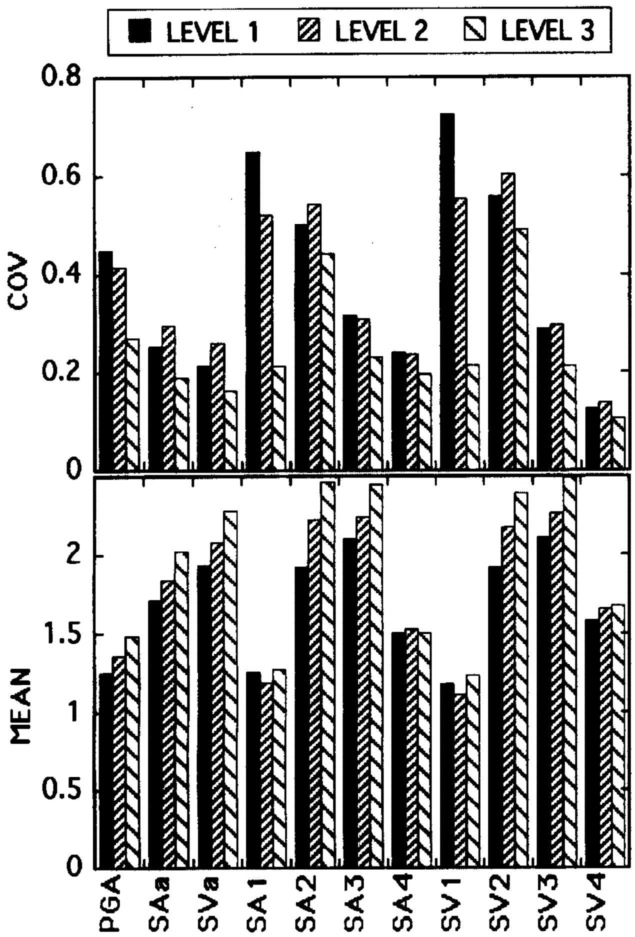


Fig. 6. Amplification mean and cov for soil response due to variation of soil characteristics for three levels

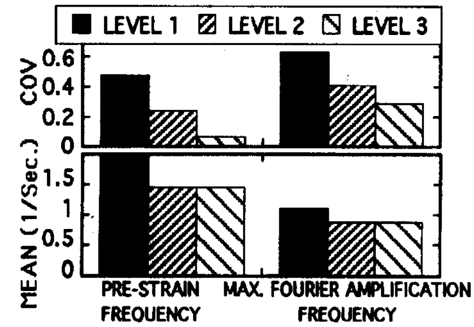


Fig. 7. Frequencies mean and cov for soil response due to variation of soil characteristics for three levels of available information.

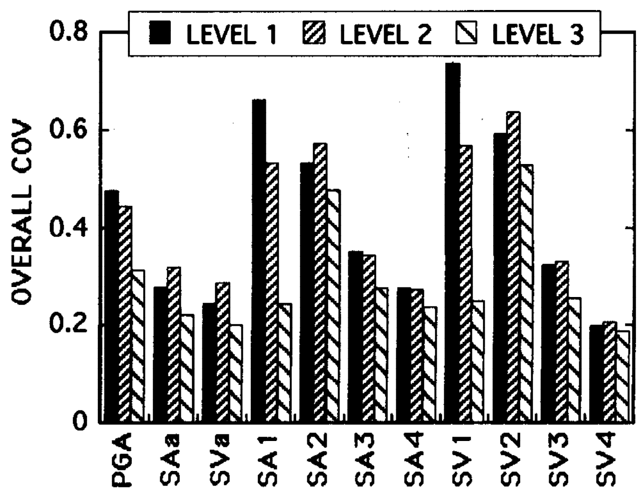


Fig. 8. Amplification overall mean and cov for soil response parameters when three levels of information

DISCUSSION AND CONCLUSIONS

Uncertainty of soil amplification is estimated considering the variabilities due to analytical method, non-stationarity of input motion, and soil characteristics. A modification for Ramberg-Osgood hysteresis model is proposed to accommodate the empirical relation that is based on the statistics of mechanical parameters. The modified hysteresis model is found appropriate for the simulation of mechanical models relationships. Analysis is carried out for general sites configurations as well as a specific site with different levels of information qualities. It is found that while variability of soil characteristics results in a dominant amplification uncertainty, the latter can be decreased about 40% when additional information is collected for soil characteristics. For an example site in Tokyo, when enough information is collected for soil data, the overall uncertainty, expressed in terms of cov, can be around 30%, 20% for PGA and spectrum intensity respectively. For spectrum values average amplification, it can be as high as 50% around the natural period. Thus, it cannot be neglected when uncertainty of seismic load is discussed.

When high safety level is desired for a certain structure, large design loads are generally expected. However, if some efforts are done to collect more data about the soil, soil amplification cov can be decreased and, thus, the same level of safety can be achieved with lower design loads. That is possible if the seismic hazard cov is originally low. However, if hazard cov is high, the decrease in soil amplification cov will not result in higher safety or lower design loads.

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