A Detail Analytical Study on Ground Response Analysis of Subgrade Soil under a Flexible Pavement

Raja Saha[1] Somedeb Saha[2] Rajib Guin[3]
[1,2,3]Lecturer
[1,2,3]Department of Civil Engineering
[1,2,3]Technique Polytechnic Institute, Hooghly, India

Abstract— In Barasat, a long stretch of road exists between Champadali more and Kazipara more on which heavy traffic moves. Due to the previous mild earthquake in West Bengal, the road stretch under consideration experienced a mild ground response due to which some cracks developed resulting in the blockage of the normal traffic. Earthquake shake is the most significant and universally encountered problems in geotechnical earthquake engineering. Correspondingly, the aim is the evaluation of ground response or ground shake response. In this project a detail analytical study on ground response analysis of subgrade soil under a flexible pavement stretch between Champadali and Kazipara is elaborately discussed. Ground response analysis is used to predict site natural periods, assess ground motion amplification, provides ground motions for development of design response spectra, evaluate potential for liquefaction, and to determine the forces induced due to an earthquake which can lead to instability of earth and earth slopes. To estimate the ground response of soil one shall have correct soil model which represent its dynamic properties. For this purpose, field tests (SPT, Seismic down hole tests) and laboratory tests (cyclic triaxial test) will be carried out for soil sites from Barasat. Also 3D analysis is being done for the analysis. Using the laboratory and field test results, it is proposed to develop a model for dynamic deformation characteristics. Some case studies are being given here. The model proposed will be validated by applying recorded time history of bedrock and compares its free field response with the recorded free field time history. Also liquefaction assessment will be carried out using the site characteristics data and site response study results. The effects of soil strain, terrain properties, water table and earthquakes on Peak ground motion will be studied.

Key words: Subgrade Soil, Flexible Pavement

I. INTRODUCTION

Due to the devastations caused by earthquakes, geotechnical engineers face a very pertinent challenge to identify the causes and characteristics of these strong ground motions and the extent to which they can wreck damage. The damage potential of any particular earthquake depends on the properties of soils through which they are transmitted. At a particular site, the safety of the structures is guided by the response of the underlying soil strata subjected to the propagating seismic waves. The response is evaluated based on parameters like peak ground acceleration or PGA (the peak horizontal acceleration at ground level), maximum shear strain in soil profile, spectral acceleration (Sa) to name a few. Thus, performing an extensive GRA study for any site becomes an essential task in characterizing the site.

In this project, the effect of input motion and soil parameters variability on numerical site response evaluation has been studied. In particular, we show the effect of nonlinear soil response on the dispersion of computed ground motion.

The hyperbolic stress-strain model and its subsequent modified forms are used to describe the initial backbone curve for the first loading cycle. The stress-strain behavior in the subsequent cycles is obtained by modeling the soil stiffness degradation, with pore water pressure that develops, as the parameter for modeling the degradation. The non-linear model developed for the analysis with hysteretic damping reduction factor referred to as MRDF procedure, which modifies the Masing and extended Masing rules. Eq. (1) and Eq. (2) are used to represent the loading and unloading/reloading conditions, respectively, to obtain the stress-strain curves.

$$\tau = \frac{G_0 \gamma}{1+\beta\left(\gamma / \gamma_r\right)^s} \quad (1)$$

$$\tau = F(\gamma_m)\left[2\frac{G_0((\gamma-\gamma_{rev})/2)}{1+\beta((\gamma-\gamma_{rev})/2\gamma_r)^s} - \frac{G_0(\gamma-\gamma_{rev})}{1+\beta(\gamma_m-\gamma_r)^s}\right] + \frac{G_0(\gamma-\gamma_{rev})}{1+\beta(\gamma_m-\gamma_r)^s} + \tau_{rev} \quad (2)$$

Conventional static and dynamic structural analyses are based on the assumption that the structure is fixed to the soil, and that there is no interaction between the soil and the structure. In general, the structure interacts with the surrounding soil, so it is necessary to account for soil-structure interaction (SSI), especially in dynamic load cases. Moreover, some dynamic loads like earthquakes, blast and traffic induced vibrations are applied to the soil region around the structure. Therefore, both the structure and the soil region have to be properly modeled. For structural modeling conventional Finite Element Method (FEM) is used. The structure is meshed in order to represent the geometry, boundary conditions, mass and applied loads. In order to model the soil region finite elements can also be used, but since the soil is an unbounded medium, an appropriate boundary has to be introduced. For static loading a fictitious boundary at a sufficient distance from the structure is used, so the obtained finite domain of the soil can be modeled similarly as the structure.

II. SITE CLASSIFICATION

The amplification of ground motions at a nearly level site is significantly affected by the natural period of the site (Tn = 4H/Vs; where Tn = natural period, H = soil depth, and Vs = shear wave velocity). Both dynamic stiffness and depth are important. Other important seismic site response factors include the impedance ratio between the soil deposit and underlying bedrock, the material damping of the soil deposits, and the nonlinear response of soft a potentially liquefiable soil deposits. The effect of nonlinearity is largely a function of soil type. Factors such as cementation and geologic age may also affect the nonlinear behavior of soils. To account partially for these factors, a site classification scheme should include a measure of the dynamic stiffness of the site and a measure of the depth of the deposit.

The site classification scheme proposed herein is an attempt to account for the factors affecting seismic site response while minimizing the amount of data required for site characterization. The site classification scheme is based on two main parameters and two secondary ones. The primary parameters are:

– Type of deposit, weathered rock, stiff soil, soft soil, and potentially liquefiable soil in some areas.
– These general divisions introduce a measure of dynamic stiffness to the classification scheme.
– However, a generic description of a site is sufficient for classification, without the need for measuring shear wave velocity over the upper 30 m.
– Depth to bedrock or to a significant impedance contrast.
– The secondary parameters are depositional age and soil type. The former divides soil sites into Holocene or Pleistocene groups, the latter into primarily cohesive or cohesionless soils. These subdivisions are introduced to capture the anticipated different nonlinear responses of these soils.

III. Literature Review

Major developments have taken place in many areas of geotechnical earthquake engineering over the last fifteen years. The major areas of practice that are examined are the specification of design ground motions, dynamic response analysis, evaluation of liquefaction potential, evaluation of residual strength of liquefied soil, liquefaction displacement analysis and seismic risk analysis. Also, great improvements have been made in the laboratory measurements and in simulating field loading and stress conditions which have enabled in better understanding of soil behavior. A number of works have been carried out on mitigation measures against liquefaction and in particular to embankments. The major soil properties required in soil dynamics include shear modulus, poisons ratio, damping and attenuation characteristics, liquefaction parameters such as cyclic shearing stress ratio, cyclic deformation, pore pressure response and shear strength. Some of them are best studied in field, others in laboratory and some can be measured both in situ and laboratory. Thus in the present part, mechanism of liquefaction, consequence of liquefaction, mitigation measures against liquefaction, dynamic properties of soil, constitutive models available and seismic effects on embankment are presented in brief Since the objective of the present work is to suggest suitable method to mitigate the liquefaction effects on embankments, proper understanding of the liquefaction phenomenon is essential. The background information mainly focuses on materials related to the explanation of liquefaction of granular soils.

The soil mechanics literature did not show much evidence of liquefaction studies before 1964. However, the classical works of Terzaghi and Peck (1948) indicate that there was at least recognition that static loading could induce liquefaction. The costly lessons learnt from the Alaska earthquake in USA and Niigata earthquake in Japan both in 1964 have given a momentum to earthquake oriented research. The remarkable progress in soil dynamics during the past decade may be attributed to a timely combination of the need and capabilities to meet it. The need has arisen from the increase in construction activities in general. The recently acquired capabilities to solve soil dynamics problems in analyses and experiments owe their effectiveness to the progress in digital computers and electronic measuring and recording systems. The analytical capabilities have stimulated experimental research to seek more accurate stress-strain relationships of soils. Also experimental research gives more insight to the behavior of soil during shaking. The case histories of liquefaction occurrence and liquefaction induced ground deformation during past earthquakes are essential for understanding and characterizing the effects of liquefaction, and for developing physical and empirical models to predict liquefaction damage. Bartlett and Youd (1992) compiled the information and a detailed database of liquefaction induced ground deformation of seven earthquakes (1906 San Francisco - California, 1964 Anchorage - Alaska, 1964 Niigata - Japan, 1971 San Fernando - California, 1979 hnperial Valley-California, 1983 Nihonkai - Chubu - Japan, 1987 Superstition Hills - CaKfomia). They showed the lateral spread in all the regions lay within the boundaries of liquefaction occurrence. The 1971 earthquake in California caused liquefaction flow failure in the upstream slope of the Lower San Fernando dam that endangered the lives of 80,000 people living downstream (Seed et al., 1975). Throughout history, soil liquefaction and flow failure have caused loss of life and spectacular damage in earthquake. Seed (1979b) and Ishihara (1993) are among a few researchers who have performed extensive works and proposed simplified methodologies in liquefaction studies over years.

IV. Modeling of Soil Profile & Input Details

For this project, soil profiles from the stretch between Champadali more and Kazipara more have been considered. Soil data have been taken from available borehole log reports for this region. The sub-surface geology profile in this region consists of inter-bedded layers of clayey silts, silty sands and sands. Water table has been observed to be quite close to the ground surface (within 2 meters from the existing ground level) in almost all of the borehole locations under consideration. The soil up to 10-15 m depth is mostly soft or in loose state with SPT-N value much lesser than 30, thereby, making them highly susceptible to liquefaction. The shear wave velocities (Vs) of different layers in the soil profiles have been evaluated from the empirical correlation with SPT-N values.

A. Methodology

Non-linear GRA, incorporating non-Masing criteria, has been performed on the soil strata-graphic profiles using DEEPSOIL v6.0 and, subsequently, liquefaction susceptibility identification has been done.

The modulus reduction and damping curves that needs to be defined for different soil layers, have been obtained from the formulations. These formulations take into account the soil properties like plasticity index, confining pressure, over-consolidation ratio, angle of internal friction to name a few. The obtained curves have been fitted using the MRDF procedure and, subsequently, the parameters for the stress-strain model get defined. The reference strain and small-strain damping ratio of the soil profiles have been considered to be independent of confining pressure.

V. NUMERICAL SIMULATIONS METHOD

It is well known that the shear-wave velocity profile and the nonlinear modulus reduction and damping curves have a significant impact on the soil behaviour subjected to cyclic loading. Furthermore, the estimation of the soil profile layout, the corresponding properties (e.g. Vs, density, shear modulus) and their evaluation in situ and in the laboratory exhibits some degree of uncertainty. Recently some authors (Koutsourelakis et al., 2002; Popescu et al. 2006; Rathje et al. 2010) among others propose to adopt probabilistic approaches in practical earthquake engineering applications. In this way, the variability in soil properties can be incorporated in site response analysis through a Monte Carlo simulation. This method allows estimating the statistical response of a model by computing their response for different input parameters values. Two main sampling procedures can be used to generate these parameters: simple random sampling and Latin hypercube sampling (Xu et al. 2005; Helton et al., 2006). According to Xu et al. (2005) several studies point out that Latin hypercube sampling (LHS) can more exhaustively explore model parameter space than simple random sampling with a smaller sample size.

In this work, the relevant input parameters are the shear wave velocity profile and the shear modulus reduction curves. The studied response parameters concern both the acceleration level and spectral response at free field. According to Griffiths and Fenton (2001), Popescu et al. (2006) among others, there is no clear evidence pointing to any specific model for the probability density function (pdf) of soil properties. However, they proposed to use non-negative functions as Beta, Gamma or lognormal for many material properties. The probabilistic shear-wave velocity profiles generated for Latin hypercube simulations are based on a baseline shear-wave velocity profile. The baseline shear-wave velocity profile used in this study is based on the model proposed for the IWTH08 KiK-net station (see section 3) and it is assumed to be characterized statistically by a lognormal distribution at any given depth. The baseline shear-wave velocity profile defines the mean values of Vs and in order to take into account the uncertainty several values of the coefficient of variation (CV) varying from 10 to 30% are used. Figure 1 displays one of the obtained uncertainty shear-wave profile for CVVs=20%. In this figure, the median, the ± one standard deviation and the range of Vs profiles determined by Latin hypercube sampling are showed. These summarized curves involve 100 sample computations. The range of Vs profiles represents the limits of the probabilistic profiles. It is important to note that the median response obtained is in agreement with the baseline shear-wave.

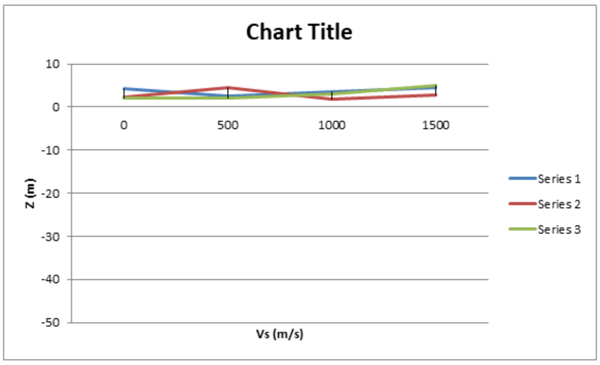

Fig. 1:

Concerning the probabilistic shear modulus degradation (i.e. G-γ) curves, according to the used backbone stress-strain model (i.e. the hyperbolic model) the nonlinear relation is controlled by the γref parameter following the hyperbolic model (Konder and Zelasko, 1963). Thus the randomness in the dynamic properties of the soil is introduced through this parameter. It is assumed that γref is characterized statistically by a lognormal distribution with a coefficient of variation (CV) varying from 20 to 30%. Figure 2a shows the mean, the ± one standard deviation and the range of G/Gmax curves determined by LHS. The choice of simulation (i.e. only one random parameter) implies a variation of the CV of G/Gmax for each γ level. It means that for lower γ values the randomness of these curves is due principally to Vs dispersion and for higher strain levels the randomness is a combination.

VI. RESULTS

A. Probabilistic Shear Waves Velocities

We compare the deterministic PGAs and the mean PGAs calculated with different coefficient of variation associated to the random generation of soil profiles. The shear wave velocity of each layer is first considered as random whereas the soil nonlinear properties are fixed. Figure 2 shows PGA values at the surface ground motion as a function of the PGA input motion (PHA). When PHAs are small, the ground motion is amplified in the soil column and computed PGA is predominantly greater than PHA. When the input increases, PGA increases more slowly due to soil nonlinearity (i.e. the soil damping increases affecting the higher frequencies). Above a PHA of 0.25g, PGA values at surface are lower than those at depth; nonlinear soil behavior de-amplifies the input motion. This saturation effect is characteristic of nonlinear effects (e.g. Bonilla et al., 2011).

Moreover, deterministic PGAs at surface are most of the time greater than mean PGAs whatever the coefficient of variation. Additionally, higher coefficients of variation lead to lower PGAs at surface as shown in Figure 3 displays the PGA coefficients of variation at surface as a function of PHA. When PHA increases, the coefficient of variation for PGA also increases and becomes higher than the Vs profiles coefficients of variation.

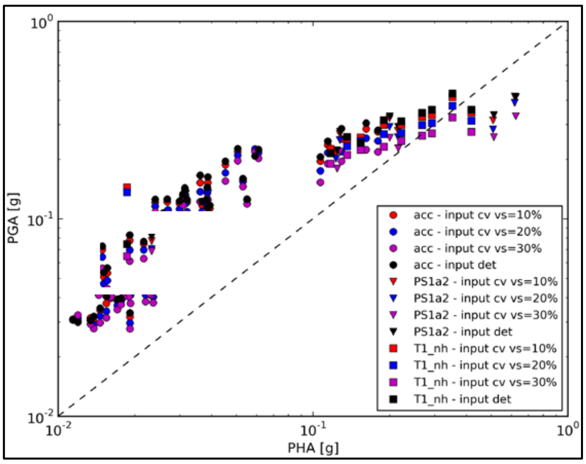

Fig. 2:

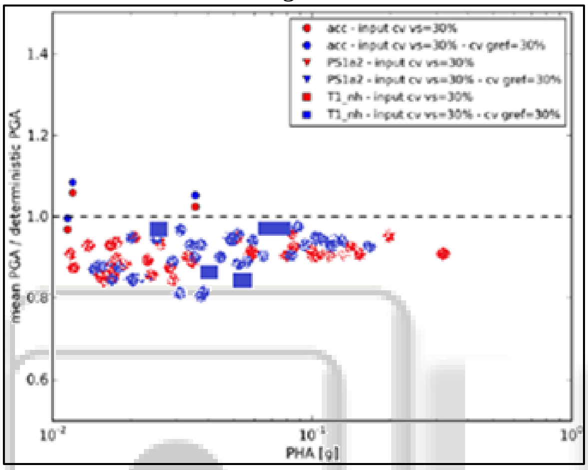

Fig. 3:

VII. DESCRIPTION OF SOIL PROFILES

Soil profiles from three borehole locations, referred to as BH1, BH2 and BH3, have been studied.

- BH1 soil profile consists of a layer of sandy silt overlying a layer of fine to medium sand with a total depth of 23 m. Ground water table is observed at a depth of 1.7 m from the existing ground level.
- BH2 soil profile consists of a top fill layer underlain by inter-bedded layers of clayey silt, sandy silt and sand up to a depth of 30 m. Ground water table is observed at a depth of 1.5 m from the ground surface.
- BH3 soil profile consists of inter-bedded layers of clayey sand, silty sand and fine to medium sand up to a depth a 15 m. Ground water table is observed at a depth of 0.7 m from ground surface.

VIII. DESCRIPTION OF SEISMIC LOADING

The mild earthquake strong motion recorded at the stretch between Champadali more to Kazipara more with peak bedrock level acceleration (PBRA) of 0.013g, and three scaled-up components of the motion with peak bedrock level accelerations of 0.06g, 0.18g and 0.36g, have been used as input for analyses. The 0.36g PBRA motion (extreme case) has been selected based on the peak accelerations recorded at some of the sites around the hypocenter of the earthquake in Sikkim, where peak bedrock level accelerations as high as almost 0.4g were recorded.

IX. RESULTS & OBSERVATIONS

Each of the three soil profiles has been analyzed with all the four input motion components and the response has been expressed in terms of PHA profile, PGA, amplification factor, spectral acceleration (Sa). Subsequently, liquefaction susceptibility of the three sites has also been looked into and the zone of liquefaction has been identified.

A. Site-BH1

It is observed that the amplification factor, i.e. the ratio of PGA to PBRA, decreases with increase in PBRA value of input motion. For the 0.02g and 0.06g PBRA input motions, amplification of motion is observed from bottom of the soil profile to the top whereas for 0.18g and 0.36g PBRA motions, PGA value at the top of the soil profile is less than the PBRA value. PHA vs. Depth profiles for site BH1 are shown in Fig. 4 for the four input motion components. Fig. 5 shows the 5% damped surface response spectra plots for the four motion components.

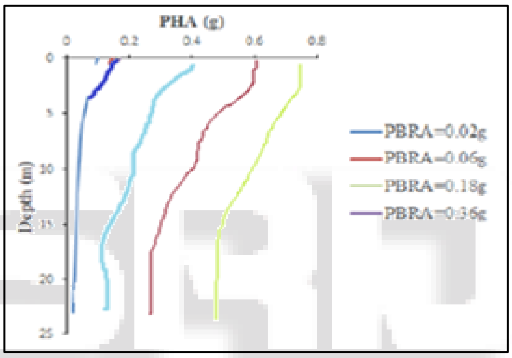

Fig. 4: PHA vs. Depth

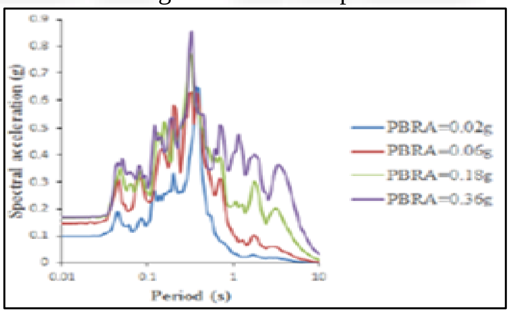

Fig. 5: Response Spectra

The amplification factor is observed to decrease with increase in PBRA value of input motion. Amplification of ground motion is observed from bottom of the soil profile to the top for the motions with PBRA values of 0.02g, 0.06g and 0.18g, whereas for the 0.36g PBRA input motion, PGA value at the top of the soil profile is less than the PBRA value. PHA vs. Depth profiles for site BH2 are shown in Fig. 6 for the four input motion components.

For the two motion components with PBRA of 0.02g and 0.06g, chances of liquefaction of the soil column are less. However, for the motion components of 0.18g and 0.36g PBRA, it is seen that liquefaction is likely to occur in the soil column up to a depth of 25 m from the ground level (FOS < 1).

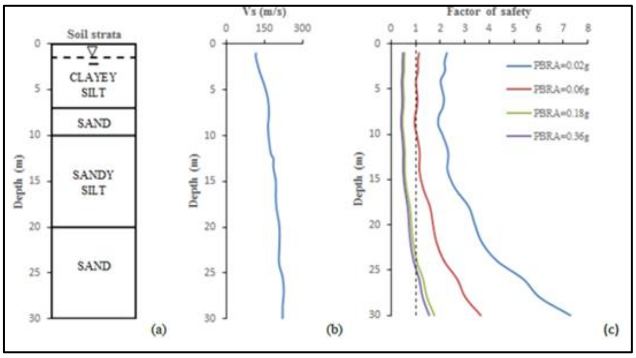

Fig. 6:

X. CONCLUSIONS

Different sets of 1D nonlinear ground response analyses, incorporating non-Masing criteria, have been performed on three borehole sites in the IIT Guwahati region, subjected to input motions having four different peak bedrock level acceleration (PBRA) values. It has been observed that the amplification factor decreases with increase in PBRA value of input motion for all the three sites which means the amplification of ground motion waves, as it travels upwards from the base of the profile to the top, decreases as the PBRA value increases. For the 0.18g PBRA motion, PGA value that has been obtained is almost same as the PBRA value, i.e. an amplification factor of 1, for the two borehole sites BH1 and BH3. For the site BH2, ground motion amplification has been observed for the input motion with PBRA of 0.18g. For the motion with PBRA of 0.36g, amplification factor lesser than 1 has been observed for all the three sites. Thus, it can be stated that attenuation of ground motion is expected to occur in the soils found in this region, for input motions having PBRA in the range 0.18g-0.36g. The 5% damped surface response spectra plots for the three sites, subjected to four input motions having different PBRA values, have also been presented.

A site classification scheme based on a general geotechnical characterization of the site that includes depth to bedrock as a key parameter is introduced. Two important conclusions were reached:

- Current attenuation relationships use as the baseline site condition a generic “rock” class that groups soft rock/shallow stiff soil and competent rock sites. The results shown in this paper indicate a significant difference in responses of these two site classes. This, in turn, highlights the need to review the database to redefine the baseline site condition for "rock" attenuation relationships.
- The standard deviations resulting from the proposed classification system are comparable with the standard deviations obtained using a more burdensome average shear wave velocity classification system. This illustrates that depth of the soil deposit is an important parameter for the estimation of seismic site response. A site classification scheme should account both for site stiffness and profile depth.

The spectral amplification factors presented in this work can be used in general probabilistic seismic hazard assessment. However, caution should be exercised when using these factors, because they are obtained from a data set containing only two earthquakes. Hence, intra-event scatter could be assessed for these two earthquakes, but inter-event scatter could not be evaluated conclusively.

Furthermore, liquefaction study has been performed for the three sites using cyclic stress approach. It has been observed that for the 0.02g PBRA input motion, liquefaction is not expected to occur in any of the sites. However, for motions having PBRA values greater than 0.06g, all the three soil sites are susceptible to liquefaction. The depth and zone of liquefaction in the soil columns have been observed to increase with increase in PBRA value of input motion. Thus, it is seen that widespread liquefaction is likely to occur in the entire region. A more recent liquefaction potential study of Barasat region has also indicated high chances of liquefaction in this region. Finally, a comparative study of equivalent linear and nonlinear approach of GRA has been presented. It has been observed that equivalent linear approach results in a higher PGA value as compared to nonlinear approach and this difference in computed PGA for the two approaches increases with increase in the PBRA of input motion.

REFERENCES

[1] Abrahamson, N. A. and Silva, W. J. (1997). "Empirical Response Spectral Attenuation Relationships for
[2] Shallow Crustal Earthquakes." Seis. Res. Letters, Vol. 68(1), pp. 94-127.
[3] Borcherdt, R. D. (1994). "Estimates of Site-Dependent Response Spectra for Design (Methodology and
[4] Justification)." Earthquake Spectra, Vol. 10(4), pp. 617-653.
[5] Bray, J. D., and Rodriguez-Marek, A. (1997). "Geotechnical Site Categories." Proceedings. First PEERPG&
[6] E Workshop on Seismic Reliability of Utility Lifelines, San Francisco, CA, 1997.
[7] Dobry, R., Martin, G. M., Parra, E., and Bhattacharyya, A. (1994). "Development of Site-Dependent Ratios of
[8] Elastic Response Spectra (RRS) and Site Categories for Building Seismic Codes." Proceedings of the
[9] NCEER/SEAOC/BSSC Workshop on Site Response during Earthquakes and Seismic Code Provisions,
[10] University of Southern California, Los Angeles, November 18-20, edited by G. M. Martin.
[11] Rodriguez-Marek, A., Bray, J. D., and Abrahamson, N. (1999). “Task 3: Characterization of Site Response:
[12] General Site Categories,” PEER Report 1999/03. Pacific Earthquake Engineering Research Center, Univ. of
[13] California, Berkeley, CA.
[14] Seed, H. B. and Alonso, J. L. (1974). "Soil-structure interaction effects in the Caracas Earthquake of 1967,"
[15] Report No. 73023, Earthquake Engineering Research Center, Engineering Department, University of California,

