Influence of Site Conditions on Seismic Design Spectra for Bridges

Muhammad Tariq A. Chaudhary
Civil Engineering Department, Kuwait University, Kuwait, Kuwait
Email: mtariqch@hotmail.com, tariq.chaudhary@ku.edu.kw

Abstract—Site conditions (geotechnical soil properties and geological setting) influences the surface response of strata to seismic ground motion. This fact is recognized in all seismic design codes by assigning spectral shapes based on site conditions. Due to variability in site properties and a myriad number of their combinations, design codes made a number of limiting assumptions. This paper attempted to test the bounds of some of these parameters (strata depth, Vs30, Tg, PI, ICR) and compared the results with AASHTO bridge design code. A number of limitations in the code were identified and corrections factors for some specific conditions are suggested.

Index Terms—acceleration spectra, bridges, soil properties, shear wave velocity

I. INTRODUCTION

Influence of site conditions on seismic design spectra is well recognized in current codes. AASHTO code [1] caters for these effects by characterizing the site conditions into six classes based on shear wave velocity in the top 30 m depth (Vs30) or alternative procedures based on average SPT N values or undrained shear strength of top 30 m strata. Design spectrum for a bridge is constructed based on the mapped PGA of the site, short period spectral acceleration, Sr and long period spectral acceleration, Ss along with site modification factors Fp, Fa, Fv and Fs. Effect of variation in soil properties like Plasticity Index (PI), Over Consolidation Ratio (OCR), effective stress (‘σ’), depth of soil strata over bedrock and variation in stiffness of the bedrock are currently not included in the code based site characterizing and design spectrum construction processes. This study examined the influence of these geotechnical and site parameters on spectral acceleration, SA, used for seismic design of bridges.

II. METHODOLOGY

A. Background

Site conditions are classified into six categories (A to F) in the AASHTO code based on Vs30. Site Classes A and B are rock sites with Vs30 more than 760 m/s. Shallow spread footings are commonly used for bridges in these site classes. Site class C represents very hard soil or soil-rock with Vs30 between 360 and 760 m/s. Shallow spread foundations are suitable in this site class for upper range of Vs30 while deep pile foundations can be used for the lower values of Vs30. Site class D has Vs30 from 175 m/s to 360 m/s. Pile foundations are commonly used for bridges in this site class. This study focused on the most commonly occurring soil classes C and D which are suitable for both shallow as well as deep foundations and are characterized by a wide variation in Vs30 (175 – 760 m/s), PI (0 – 60), OCR (1 – 10) and σ’ (20 – 1500 kPa). It is to be noted that AASHTO site classes C and D roughly corresponds to JRA soil types SC-II and SC-III [2] and EC-8 soil classes B and C [3] respectively.

B. Procedure

Sensitivity of seismic spectral acceleration response, SA, to variation in Vs30, PI, OCR, σ’, depth of soil strata and variation in bedrock stiffness; characterized by variation in Vsh based on CSIR classification for rocks [4] was undertaken in this study. The seismic bedrock characteristics considered in this study are classified as rock classes I to V in which class I is a very good rock (Vs > 3353 m/s) and class V a poor rock (Vs = 600 m/s). Five generic soil profiles falling within the Vs30 range for soil classes C and D were selected from the literature [5] as depicted in Fig. 1. Variations in PI, OCR, σ’, strata depth and Impedance Contrast Ratio (ICR) between soil strata and bedrock considered in the study are summarized in Table I. Twenty actual far-field ground motions varying in PGA from 0.036 to 0.47g were selected from literature [6], [7] to perform one-dimensional non-linear seismic site response analysis. Acceleration response spectra of these strong motions is depicted in Fig. 2. Table II lists the salient features of the selected ground motions as well as group these into Design Basis Earthquakes (DBE), Functional Evaluation Earthquakes (FEE) and maximum credible earthquake (MCE) based on a design PGA of 0.2g. Ground motions 18 to 20 were scaled down to match MCE for the design PGA.

More than 2400 analysis were carried out using 1-D site response analysis software STRATA [8] for various combinations of soil profiles, soil properties, strata depth and ICR. STRATA performs a 1-D linear/non-linear seismic response analysis of the soil column in the time domain and incorporates strain dependent non-linear shear modulus reduction and damping curves from a number of sources. In this study, modulus reduction and damping curves developed by Darendeli [9] were used.
and are depicted in Fig. 3 for the soil classes included in the study.

![Figure 1. Shear wave velocity profile for various site class.](image1.png)

**Figure 2. Acceleration response spectra of used strong motions.**

![Figure 2.](image2.png)

**TABLE I. VARIATION IN SOIL PARAMETERS**

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>$V_{ss}$ (m/s)</td>
<td>600 (C_high)</td>
</tr>
<tr>
<td></td>
<td>475 (C_avg)</td>
</tr>
<tr>
<td></td>
<td>350 (D_high)</td>
</tr>
<tr>
<td></td>
<td>275 (D_avg)</td>
</tr>
<tr>
<td></td>
<td>175 (D_low)</td>
</tr>
<tr>
<td>PI</td>
<td>0, 15, 60</td>
</tr>
<tr>
<td>$\sigma'$ (atm)</td>
<td>2, 4</td>
</tr>
<tr>
<td>Strata depth (m)</td>
<td>40, 110</td>
</tr>
<tr>
<td>$V_s$ bedrock (m/s)</td>
<td>600, 760, 1350,</td>
</tr>
<tr>
<td></td>
<td>2251, 3353</td>
</tr>
</tbody>
</table>

**TABLE II. GROUND MOTIONS USED IN THE STUDY**

<table>
<thead>
<tr>
<th>EQ Record ID</th>
<th>Seismic event</th>
<th>Year</th>
<th>Station</th>
<th>Magnitude</th>
<th>PGA (g)</th>
<th>Fault distance (km)</th>
<th>$V_s$ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Edgecomb, NZ</td>
<td>1987</td>
<td>Maraenui Primary School</td>
<td>6.6</td>
<td>0.036</td>
<td>69</td>
<td>425</td>
</tr>
<tr>
<td>2</td>
<td>Oroville-04</td>
<td>1975</td>
<td>Medical Center</td>
<td>4.37</td>
<td>0.078</td>
<td>9.2</td>
<td>519</td>
</tr>
<tr>
<td>3</td>
<td>Irpinia, Italy</td>
<td>1980</td>
<td>Calitri</td>
<td>6.9</td>
<td>0.14</td>
<td>17</td>
<td>600</td>
</tr>
<tr>
<td>4</td>
<td>Chi-Chi, Taiwan</td>
<td>1999</td>
<td>CHY015</td>
<td>7.6</td>
<td>0.183</td>
<td>38.1</td>
<td>229</td>
</tr>
<tr>
<td>5</td>
<td>Spitak- Armenia</td>
<td>1988</td>
<td>Gukasian</td>
<td>6.77</td>
<td>0.205</td>
<td>24</td>
<td>275</td>
</tr>
<tr>
<td>6</td>
<td>Kobe, Japan</td>
<td>1995</td>
<td>Shin Osaka</td>
<td>6.9</td>
<td>0.21</td>
<td>19</td>
<td>256</td>
</tr>
<tr>
<td>7</td>
<td>Kocaeli, Turkey</td>
<td>1999</td>
<td>Ambarli</td>
<td>7.51</td>
<td>0.23</td>
<td>69.6</td>
<td>175</td>
</tr>
<tr>
<td>8</td>
<td>San Fernando</td>
<td>1971</td>
<td>Castaic - Old Ridge Route</td>
<td>6.61</td>
<td>0.266</td>
<td>23</td>
<td>450</td>
</tr>
<tr>
<td>9</td>
<td>Landers</td>
<td>1992</td>
<td>Joshua Tree</td>
<td>7.28</td>
<td>0.28</td>
<td>11</td>
<td>379</td>
</tr>
<tr>
<td>10</td>
<td>Dinar, Turkey</td>
<td>1995</td>
<td>Dinar</td>
<td>6.4</td>
<td>0.30</td>
<td>3.4</td>
<td>220</td>
</tr>
<tr>
<td>11</td>
<td>Superstition Hills</td>
<td>1987</td>
<td>Poe Road (temp)</td>
<td>6.5</td>
<td>0.31</td>
<td>11.7</td>
<td>208</td>
</tr>
<tr>
<td>12</td>
<td>Tabas, Iran</td>
<td>1978</td>
<td>Dayhook</td>
<td>7.35</td>
<td>0.328</td>
<td>13.9</td>
<td>660</td>
</tr>
<tr>
<td>13</td>
<td>Hector Mine</td>
<td>1999</td>
<td>Hector</td>
<td>7.1</td>
<td>0.34</td>
<td>12</td>
<td>685</td>
</tr>
<tr>
<td>14</td>
<td>Imperial Valley</td>
<td>1979</td>
<td>Elcentro Array #11</td>
<td>6.5</td>
<td>0.36</td>
<td>13.5</td>
<td>196</td>
</tr>
<tr>
<td>15</td>
<td>Cape Mendocino</td>
<td>1992</td>
<td>Rio Dell Overpass - FF</td>
<td>7.0</td>
<td>0.38</td>
<td>13</td>
<td>312</td>
</tr>
<tr>
<td>16</td>
<td>Loma Prieta</td>
<td>1989</td>
<td>Gilroy Array #2</td>
<td>6.93</td>
<td>0.40</td>
<td>11</td>
<td>271</td>
</tr>
<tr>
<td>17</td>
<td>Northridge-01</td>
<td>1994</td>
<td>Beverly Hills - 14145 Mulhol</td>
<td>6.69</td>
<td>0.43</td>
<td>17</td>
<td>356</td>
</tr>
<tr>
<td>18</td>
<td>Manjil, Iran</td>
<td>1990</td>
<td>Abbar</td>
<td>7.4</td>
<td>0.51 (0.44)</td>
<td>13</td>
<td>724</td>
</tr>
<tr>
<td>19</td>
<td>Duzce, Turkey</td>
<td>1999</td>
<td>Duzce</td>
<td>7.14</td>
<td>0.52 (0.45)</td>
<td>6.6</td>
<td>276</td>
</tr>
<tr>
<td>20</td>
<td>Kobe, Japan</td>
<td>1995</td>
<td>Takatori</td>
<td>6.9</td>
<td>0.64 (0.45)</td>
<td>1.5</td>
<td>256</td>
</tr>
</tbody>
</table>
Figure 3. Modulus reduction and damping curves for soils.

40 m soil strata

- C high (V_s30 = 600 m/s)
- C avg. (V_s30 = 475 m/s)
- D high (V_s30 = 350 m/s)
- D avg. (V_s30 = 275 m/s)

110 m soil strata

- C high (V_s30 = 600 m/s)
- C avg. (V_s30 = 475 m/s)
- D high (V_s30 = 350 m/s)
- D avg. (V_s30 = 275 m/s)
Figure 4. Median value of spectral acceleration at surface for EQ 1 to 8 (DBE, median PGA_{rock} = 0.2 g).

---

40 m soil strata

C_high (V_{s30} = 600 m/s)

D_high (V_{s30} = 350 m/s)

C_avg. (V_{s30} = 475 m/s)

D_avg. (V_{s30} = 275 m/s)
The choice of PI values as listed in Table I were based on the generally accepted limit for sand, medium plastic and highly plastic soils [10]. OCR was kept constant (= 1) for all soil profiles based on the observations of Guerreiro et al. [11]. Two values of confining pressure were considered: 2 atm for sandy soils (PI=0) and 4 atm for soils with PI= 15 and 60. The choices of confining pressure were made based on the observations of Guerreiro et al. [11] and to limit the number of simulation cases to a realistic number.

III. RESULTS & DISCUSSIONS

Salient results of the numerical simulations are presented and compared with the current code provisions in this section. Fig. 4 and Fig. 5 present the results of response spectra for the selected five site classes for 40 m and 110 m depths of soil strata at the strata surface. Fig. 4 presents the results for ground motions 1 to 8 which had a maximum PGA value of 0.266g and a median value of 0.2g and were representative of DBE ground motions. Fig. 5 depicts the results for ground motions 9 to 20 with a maximum PGA of 0.45g and a median value of 0.38g and were representative of MCE.

Design spectra based on AASHTO code [1] is also superimposed on the computed spectra in Fig. 4 and Fig. 5. In the legend of Fig. 4 and Fig. 5, the first number refers to the PI value and the second to the bedrock shear wave velocity. The following observations were made regarding the computed response spectra.

A. Influence of Soil Type

Fig. 6 depicts the variation in surface spectral acceleration for various soil types. Maximum spectral acceleration increased with a decrease in $V_{s30}$ for soil class C while it decreased with a decrease in $V_{s30}$ for soil class D. This observation is contrary to the current AASHTO code practice that calls for an increased spectral acceleration value for softer soils with less $V_{s30}$.

B. Influence of Bedrock Vs on Spectral Acceleration

Shear wave velocity of the bedrock ($V_{s\_bedrock}$) seems to have a pronounced effect on the spectral acceleration as depicted in Fig. 7. This is especially true for soil types with higher $V_{s30}$ values. The effect of bedrock shear wave velocity on the spectral response of the soil strata can be characterized through impedance contrast ratio (ICR). ICR is defined as:

$$ICR = \frac{\gamma V_S R}{\gamma V_S S}$$

where $\gamma$ is the unit weight, $V_S$ is the shear wave velocity and subscripts $R$ and $S$ refers to parameters of the bedrock and the soil layer above it respectively. For the analysis cases investigated in this study, ICR varied between 1 and 4 for type C soils and between 1 and 18 for type D soils for 110 m deep strata, while it was between 1 and 7 for type C soils and between 1 and 16 for type D soils respectively for 40 m soil profiles. It can be noted in Fig. 7 that the effect of ICR is very negligible for soil $D\_low$ as maximum spectral acceleration varied between 0.6g and 0.9g. The maximum impact was observed for soil type $D\_high$ with a variation between 1.2g and 3.3g. Referring to Fig. 5 and Fig. 6, it was observed that increase in SA was only due to the increase in ICR for soil $C\_high$ and there was almost negligible
contribution to increase in SA due to increase in PI of soil. Soil C_avg also exhibited a similar trend. However, in this case, PI of soil also played a little role in increasing the SA. An increase in the bedrock shear wave velocity generally resulted in increased spectral acceleration. An increase in spectral acceleration of more than 2.5 times was noted for a bedrock $V_s$ increase from 998 m/s to 3353 m/s for 40 m soil depth while it was 1.83 times for the case of 110 m deep profile as depicted in Fig. 6 and Fig. 7.

C. Influence of PI on Spectral Acceleration

Referring to Fig. 6, it was observed that spectral acceleration was independent of PI for soil class C_high and moderately influenced soil C_avg and D_low for both depths of strata. SA was strongly influenced by PI for soil classes D_high and D_avg.

The trend observed for soil Class C was reversed for soil class D for which dependency shifted from being strongly dependent on bedrock $V_s$ to strongly dependent on PI with a decrease in $V_s$. For example, soil class D_low showed a weak dependency on bedrock $V_s$ but was strongly influenced by PI. Spectral acceleration for this soil increased with increasing PI.

D. Effect of Strata Depth

It was observed in Fig. 6 and Fig. 7 that maximum value of spectral acceleration was more or less the same for both strata depths for all soil types with SA value being slightly larger for 110m deep strata. However, for the three soil types with the largest $V_{s30}$ values (C_high, C_avg, and D_high), the ‘plateau’ of spectral acceleration was generally higher in the deeper strata for both DBE and MCE as depicted in Fig. 4 and Fig. 5 respectively.

E. Effect on Natural Period of Strata

Peaks of spectral acceleration occurred at different periods for the considered soil profiles. This is expected as time period of the soil column can be computed from the well-known quarter wavelength principle as: $T_g = 4H/V_s$ and with a decrease in $V_s$, $T_g$ is expected to be more. Comparison of theoretical and numerically computed values of ground period are listed in Table III. Numerical values of $T_g$ were found by the ‘peak picking’ method from the spectra in Fig. 4. It can be noted that the theoretical values of $T_g$ computed based on a strata depth of 30 m and $V_{s30}$ compared well the numerically determined values for 40m deep strata. However, for 110 m deep strata, travel time based shear wave velocity for the whole strata depth ($V_{s110}$) gave $T_g$ values that compared with the spectra values for all soil types except D_low.

All soil types in the 110 m deep strata exhibited multi-modal response except for soil C_high. This multi-modal response was attributed to the reflections of seismic waves within the deeper strata which was absent in the shallower strata.

IV. COMPARISON WITH CODE BASED DESIGN SPECTRA

A. AASHTO Design Acceleration Spectra

Design spectra for DBE and MCE calculated using AASHTO code are overlaid on the SA graphs in Fig. 4 and Fig. 5 respectively. A large disparity was observed between the AASHTO design spectra and the numerically computed spectra for various soil types. Generally, the maximum spectral acceleration occurred for soil with highest PI and highest ICR for both strata depths. Current AASHTO code does not consider dependency of spectral acceleration on PI or ICR in the creation of design spectrum. Several researchers have pointed out discrepancies in the construction of design acceleration spectra in current codes [12]–[14]. However, these studies did not consider the combined influence of PI, strata depth, $T_g$ and ICR on surface SA. Salient differences in the AASHTO design spectra with the current study are summarized below.

1) Influence of PI

Sand and clays exhibit significantly different shear stress-strain and damping behavior with increasing seismic acceleration. AASHTO code assigns spectral acceleration coefficients based on $V_{s30}$ only. Therefore, the fact that soil type (sand or clay) is not considered in the selection of these parameters despite the fact that either type of soil can have the same $V_{s30}$.

Influence of PI is generally insignificant for soils with very high $V_{s30}$. However, variation in PI has significant impact on SA in other soils as discussed earlier.

2) Influence of strata depth

The current study included strata depths of 40m and 110m and it was found that strata depth did not significantly influence the maximum magnitude of SA. Therefore, this effect can be neglected for SA amplitude as in the current AASHTO code. However, it was observed that deeper strata depth elongates the SA ‘plateau’ over a longer period corresponding to larger $T_g$ of deeper strata. Hence, a correction is required to extend the maximum SA value over a broader range of T.

<table>
<thead>
<tr>
<th>Soil Type</th>
<th>$V_{s30}$ (m/s)</th>
<th>$V_{s110}$ (m/s)</th>
<th>$4H/V_{s30}$</th>
<th>$4H/V_{s110}$</th>
<th>$T_g$ (sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C_high</td>
<td>600</td>
<td>983</td>
<td>0.20</td>
<td>0.45</td>
<td>0.15</td>
</tr>
<tr>
<td>C_avg</td>
<td>475</td>
<td>720</td>
<td>0.25</td>
<td>0.61</td>
<td>0.26</td>
</tr>
<tr>
<td>D_high</td>
<td>350</td>
<td>649</td>
<td>0.34</td>
<td>0.67</td>
<td>0.31</td>
</tr>
<tr>
<td>D_avg</td>
<td>275</td>
<td>510</td>
<td>0.44</td>
<td>0.86</td>
<td>0.50</td>
</tr>
<tr>
<td>D_low</td>
<td>175</td>
<td>310</td>
<td>0.69</td>
<td>1.41</td>
<td>0.49/1.00</td>
</tr>
</tbody>
</table>

TABLE III. THEORETICAL AND NUMERICAL VALUES OF GROUND PERIOD ($T_g$)
3) Influence of $T_g$

Natural period of the strata strongly influenced the SA around the $T_g$ resonance peaks. Current AASHTO code attempts to consider this effect through the parameter T0 which is defined as SD1/SDS. $T_g$ is the period up to which the maximum SA stays constant and starts to decrease after this time period. It was observed in Fig. 4 and Fig. 5 that the current definition of $T_g$ is insufficient to cover the period range with maximum SA.

4) Influence of ICR

The current provisions of AASHTO code do not seem to cater for values of ICR more than 1 as evidenced from Fig. 4 and Fig. 5. ICR influences the soil strata with relatively larger values of $V_{s30}$ and has a minimum influence on soils with smaller $V_{s30}$.

![Figure 8. Modified spectrum for soil C_avg. for DBE.](image)

![Figure 9. Modified spectrum for soil D_avg. for DBE.](image)

B. Proposed Changes to the AASHTO Design Spectra

1) Correction for PI

Correction for PI is required for soil classes D_high and D_avg only. Although, soil type D_low exhibited the most dependence on PI, the current AASHTO code design spectra adequately caters for this effect in this type of soil.

2) Correction for $T_g$

Correction for $T_g$ is needed for soils with deeper strata and lower $V_s30$ values. This correction is applied by extending the limit of the flat part of the AASHTO design spectra to $T_g$ based on deeper strata.

3) Correction for ICR

ICR affects SA the most for soil types with higher $V_s30$. It is proposed to increase the SA value in the flat part of the AASHTO design spectra corresponding to ICR of 2 which also coincides with the median value of computed SA for both strata depths in such soil types.

4) Modified design spectra

Fig. 8 and Fig. 9 presents the modified design spectra for soil type C_avg and D_avg respectively.

V. CONCLUSIONS

The following conclusions are drawn from this study:

- Acceleration response spectra for two levels of earthquakes (DBE with PGA $\leq 0.2$ g and MCE with PGA between 0.2g and 0.38g) were computed for two depths (40m and 110m) of soil strata for five soil types with $V_{s30}$ varying between 175 m/s and 600 m/s.
- Acceleration response spectra exhibited dependence on soil PI, $V_{s30}$, strata depth, $T_g$ and ICR to varying degrees in different soil types.
- Current AASHTO code does not consider influence of all variables in constructing design acceleration spectra and leads to non-conservative results for some soil types.
- Correction factors were proposed to the current AASHTO procedure and example of modified design spectra were provided.

ACKNOWLEDGEMENT

This work was supported by Kuwait University, Research Grant No. EVO1/15.

REFERENCES


Muhammad Tariq Chaudhary, born in Pakistan, received his BS (Hons) degree in civil engineering from University of Engineering and Technology, Lahore, Pakistan (1990); MS in structural engineering from University at Buffalo, NY, USA (1992) and PhD in civil engineering from the University of Tokyo, Japan (1999). He is a practicing structural engineer and academic currently working in the Department of Civil Engineering at Kuwait University, Kuwait. He has previously worked as Project Manager and Senior Structural Engineer with Obayashi Corporation, NESPAK and DNCE. He has published more than forty research papers in refereed journals and international conferences. His areas of research and interest are: seismic design, structural health monitoring, sustainable design and construction, soil-structure interaction and structural condition evaluation and rehabilitation. Dr. Chaudhary is a registered Professional Engineer in USA and Canada and a LEED Accredited Professional in USA. He is a Fellow of the Institution of Engineers Pakistan and a Member of American Society of Civil Engineers.