ABSTRACT

Influence of site conditions (geological setting and geotechnical properties) on seismic design spectra and site amplification factors is well recognized in modern seismic design codes [1]. The AASHTO bridge design code in the United States [2] caters for the site effects by classifying the site conditions into six classes based on three methods for the top 30 m depth of the strata: (i) travel-time averaged shear wave velocity, \( V_{s30} \) (ii) average standard penetration test (SPT) \( N \) values and (iii) average undrained shear strength, \( \bar{s}_u \). It is to be noted that method (i) i.e. \( V_{s30} \) is the preferred method for site classification in the US practice. Design spectrum for a bridge site is constructed based on its mapped peak ground acceleration (PGA), short period (0.2 s) spectral acceleration, \( S_s \), and long period (1.0 s) spectral acceleration, \( S_l \) along with site amplification factors \( F_{PGA} \), \( F_s \) and \( F_l \) whose values are determined from pertinent sections of the code. The effect of variation in geotechnical properties like plasticity index (PI), over consolidation ratio (OCR), effective stress (\( \sigma' \)), depth of soil strata over bedrock and variation in impedance contrast ratio (ICR) between soil strata and bedrock are currently not included. A number of investigators had taken note of the discrepancy related to the geological setting for particular areas [5-13]. Few studies also attempted to analyze the sensitivity of site amplification factors to variability in soil index properties [14, 15]. This study had attempted to conduct an extensive analysis on the sensitivity of site amplification factors (\( F_s \) and \( F_l \)) to geological variability (strata depth and bedrock properties), geotechnical factors (PI, OCR, \( \sigma' \)) and intensity of seismic ground motion were varied. The results were analyzed to identify the site parameters that impacted \( F_s \) and \( F_l \) values for site classes C and D. The computed \( F_s \) and \( F_l \) values were compared with the corresponding values in the AASHTO bridge design code and it was found that the code-based \( F_s \) and \( F_l \) values were generally underestimated and overestimated respectively.

METHODOLOGY

Background

AASHTO code classifies site conditions into six categories (A to F) based on \( V_{s30} \). Referring to Table 1, these site classes are: (a) rock sites (classes A and B) with \( V_{s30} \) greater than 1500 m/s and 760 m/s respectively, (b) soil-rock (class C) with 360 < \( V_{s30} \) < 760 m/s, (c) stiff soil (class D) having \( V_{s30} \) between 175 and 360 m/s, (d) soft soil (class E) in which \( V_{s30} \) is less than 175 m/s and (e) highly organic or highly plastic soft soils (class F) requiring site-specific studies. Table 1 also presents an and those in the European [16], Japanese [17] and Australia – New Zealand [18] design codes. The US and European codes utilize \( V_{s30} \) for site classification while the Japanese and Australia-New Zealand approximate correspondence between AASHTO.
site classes codes employ fundamental period of the strata overlying the bedrock in site classification as well.

Seismic design spectrum for a bridge site is constructed in the AASHTO code based on seismic hazard at the site, which is given in terms of PGA, short (0.2 s) and long (1.0 s) period spectral accelerations; $S_a$ and $S_l$ respectively and site amplification factors, $F_a$ and $F_l$, which correspond to short and medium periods respectively. $F_a$ and $F_l$ in the AASHTO code were developed for geological conditions and seismic setting prevalent in the western United States with the following values: strata depth of 30 – 40 m, PI of 15 and $V_{s30}$ between 760 and 900 m/s [3].

This study attempted to investigate the influence of variability in geological and geotechnical parameters on $F_a$ and $F_l$ by conducting 1-D site response analysis on soil profiles representatives of AASHTO site classes C and D. These site classes were selected because these are the most commonly occurring soil classes which are suitable for both shallow as well as deep foundations and are characterized by a wide variation in $V_{s30}$ (175 – 760 m/s), PI (0 – 60), OCR (1 – 10) and $\sigma^*$ (20 – 1500 kPa or 0.2 – 15 atm).

Procedure

Sensitivity of soil amplification factors to variation in $V_{s30}$, PI, OCR, $\sigma^*$, depth of soil strata and variation in bedrock shear wave velocity ($V_{rock}$) was undertaken in this study. Considered values of these parameters are listed in Table 2. The analysis procedure consisted of the following steps:

Selection of Soil Profiles

AASHTO site classes C and D encompass a wide range of $V_{s30}$ values (i.e. 175 – 760 m/s). In order to delineate the effect of $V_{s30}$ on seismic site amplification factors, these site classes were further sub-divided into 5 site classes as listed in Table 3. Soil profiles corresponding to the $V_{s30}$ ranges for these five site classes were selected from the literature [21, 22] and are depicted in Fig. 1 for 40 m and 110 m deep strata. These physically-realistic soil profiles were generated based on statistical study of 858 real soil profiles from Japan, western North America and France in [21] while [22] used a maximum-likelihood procedure on 557 soil profiles to statistically generate shear wave velocity profiles corresponding to Geomatrix and US Geological Survey (USGS) site classes.

The profiles used in this study are representative of soil strata with gradually increasing shear wave velocity with depth. Other

<table>
<thead>
<tr>
<th>Site Class</th>
<th>$V_{s30}$ (m/s)</th>
<th>$\rho$ (kg/m$^3$)</th>
<th>$\nu$</th>
<th>$G$ (MPa)</th>
<th>$\beta$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C: Shallow soil</td>
<td>400</td>
<td>2060</td>
<td>0.35</td>
<td>741</td>
<td>3</td>
</tr>
<tr>
<td>C: Deep soil</td>
<td>700</td>
<td>2020</td>
<td>0.35</td>
<td>456</td>
<td>4</td>
</tr>
<tr>
<td>D: Shallow soil</td>
<td>1000</td>
<td>1980</td>
<td>0.40</td>
<td>243</td>
<td>5</td>
</tr>
<tr>
<td>D: Deep soil</td>
<td>1500</td>
<td>1900</td>
<td>0.40</td>
<td>144</td>
<td>7</td>
</tr>
</tbody>
</table>

Table 2: Variation in soil parameters considered in the study.
permutations of soil profiles with sharp differences in layer properties ($V_{s30}$, unit weight, PI etc.) were not considered in this study. Mechanical properties, i.e. $V_{s30}$, density ($\rho$), Poisson’s ratio ($\nu$), shear modulus ($G$) and damping ratio ($\beta$), of the soil profiles are listed in Table 3. $V_{s}$ values in some layers of these profiles were scaled to match the target $V_{s30}$ for each site class. Refer to [23] for details.

**Selection of Bedrock Parameters**

In this study, shear wave velocity of the rock mass ($V_{\text{rock}}$) was chosen as the defining parameter for rock classification. South African Council on Scientific and Industrial Research (CSIR) classification for rocks [24] was used to classify the bedrock into classes I to V as listed in Table 4. According to this classification, quality of rock decreases from class I to V as $V_{\text{rock}}$ decreases from 3353 m/s for class I rock to 600 m/s for class V rock. Mechanical properties, i.e. $V_{\text{rock}}$, $\rho$, $\nu$, $G$ and allowable bearing pressure ($q_a$), of the bedrock are listed in Table 4. Damping ratio of the bedrock ($\beta_{\text{rock}}$) was taken as 1% [25].

**Variation in Soil Geotechnical Parameters**

Table 2 summarizes the variations in soil geotechnical parameters, i.e. PI, OCR, $\sigma'$, strata depth and $V_{\text{rock}}$ used in the study. The PI values chosen in the study (i.e. 0, 15 and 60) correspond to the generally accepted limits for non-plastic (i.e. sand), medium plastic and highly plastic soils [26]. A constant value of OCR ($= 1$) was adopted for all soil profiles based on the recommendations of [27].

![Figure 1: Shear wave velocity profile for various site classes (after [21, 22]). (a) 40m deep strata and (b) 110m deep strata.](image-url)
Confining pressure of soil layers increases with depth when density of the strata is assumed to stay constant. However, only two values of effective stress, \( \sigma' \), (2 atm for sands and 4 atm for clays) were used in the study. This simplification was based on the work of [27] who compared modulus reduction and damping (MRD) curves proposed by [28 – 30] and demonstrated that MRD curves of [28, 29] provided better fit for engineering applications than [30]. MRD curves of [29] do not account for the confining pressure effect, while it is captured in curves of [28]. In the current study, confining pressure values that matched the MRD curves of [29] and the median value of curves of [28] were used (i.e. 2 atm for sands and 4 atm for clays). This decision was also supported by the work of [8] who showed that variability in MRD curves did not significantly affect the site response variability in comparison to other factors. Restricting the range of \( \sigma' \) to two values in defining the MRD curves thus allowed reduction of the number of variables in the study without significantly affecting the final conclusions.

Selection of Seismic Ground Motions

The selected seismic ground motions were representative of typical far-field records that were recorded more than 10 km away from the epicenter [31]. Seismic ground motions were sorted into three groups based on the median PGA values. Groups 1, 2 and 3 had median PGA values of 0.17g, 0.31g and 0.43g respectively. These seismic ground motion groups approximately correspond to design basis earthquake (DBE), functional evaluation earthquake (FEE) and maximum considered earthquake (MCE) for a site with design PGA of 0.2g.

It is required that in order to use the mean (or median) response as the design value, a ground motion suite consisting of seven or more different records should be used in the analysis for a single hazard level [16, 32]. If fewer records are used, then the design value is defined as the maximum observed response and mean value cannot be used. Use of eleven seismic records is recommended by [33] so that the mean response parameters are within 30% and 70% confidence levels. Therefore, a suite of eleven ground motions for each of the three levels of earthquake intensities were selected from the literature [34 – 36] to perform one-dimensional non-linear seismic site response analysis. The seismic ground motions were downloaded from PEER strong motion database website [37].

More than 3950 1-D seismic site response analysis were carried out for the 33 seismic ground motions, 5 site classes, 3 values of PI, 4 \( V_{\text{rock}} \) values and 2 strata depths included in this study. 1-D seismic site response analysis was carried out using the STRATA software [25]. STRATA is capable of performing a 1-D equivalent-linear seismic site response analysis of the soil column in the time domain while utilizing strain dependent non-linear MRD curves from multiple references. This study used MRD curves of [28] for reasons mentioned earlier. Fig. 3 depicts the used MRD curves for the site classes included in the study (i.e. AASHTO C and D).

Fig. 2 depicts the acceleration response spectra for the three seismic intensity levels, while Table 5 lists the salient seismic event details of the used seismic ground motions. Some ground motions were scaled to match the targeted median value and such values are identified in Table 5 with a footnote. Shear wave velocity of the sites in the selected ground motion set shown in Table 5 varied between 600 m/s and 1428 m/s with an average value of 747 m/s. It is understood that the bedrock \( V_s \) used in the study varied between 760 m/s and 3353 m/s and \( V_s \) of the used input motions should ideally match these values. However, non-availability of recorded ground motions on very hard rock sites that also satisfy the far-field fault distance criterion (>= 10 km), PGA variation from 0.1g to 0.5g and the required number of the ground motions (minimum 7 and preferably 11 to use the median response value as representative of the actual response with statistical confidence) precluded this effort. Additionally, the author did not want to scale very weak motions recorded on hard rock sites to match PGA values targeted in the study.

This is the reason that the suite of ground motions presented in Table 5 is representative of ‘weak to hard rock’ motions instead of ‘hard rock’ conditions (\( V_s > 760 \) m/s). Nonetheless, average \( V_s \) of the suite of ground motions (747 m/s) is very close to the threshold value of 760 m/s assigned to \( V_s \) of seismic bedrock. Therefore, use of the ground motion data set of Table 5 should provide values of the response parameters sufficiently close to the ‘real’ values corresponding to the data set recorded on ‘hard rock’. Additionally, it has been demonstrated that use of input ground motions with different frequency content that resulted from recordings made on soil or rock sites, did not cause significant discrepancy in the response quantities [38].

1-D Seismic Site Response Analysis

More than 3950 1-D seismic site response analysis were carried out for the 33 seismic ground motions, 5 site classes, 3 values of PI, 4 \( V_{\text{rock}} \) values and 2 strata depths included in this study. 1-D seismic site response analysis was carried out using the STRATA software [25]. STRATA is capable of performing a 1-D equivalent-linear seismic site response analysis of the soil column in the time domain while utilizing strain dependent non-linear MRD curves from multiple references. This study used MRD curves of [28] for reasons mentioned earlier. Fig. 3 depicts the used MRD curves for the site classes included in the study (i.e. AASHTO C and D).
Table 5: Seismic ground motions used in the study.

<table>
<thead>
<tr>
<th>EQ Record ID</th>
<th>Seismic event and station details</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>San Fernando, 2/9/1971, Lake Hughes #9, 291</td>
<td>6.61</td>
<td>0.14</td>
<td>17</td>
<td>671</td>
</tr>
<tr>
<td>2</td>
<td>Kozani Greece-01, 5/13/1995, Kozani, T</td>
<td>6.40</td>
<td>0.14</td>
<td>24</td>
<td>296</td>
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<td>3</td>
<td>Northridge-01, 1/17/1994, Vasquez Rocks Park, 0</td>
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<td>996</td>
</tr>
<tr>
<td>4</td>
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</tr>
<tr>
<td>5</td>
<td>Duzeck Turkey, 11/12/1999, Lamont 531, N</td>
<td>7.14</td>
<td>0.16</td>
<td>23</td>
<td>638</td>
</tr>
<tr>
<td>6</td>
<td>Lytle Creek, 9/12/1970, Devil's Canyon, 90</td>
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<td>0.17</td>
<td>18</td>
<td>667</td>
</tr>
<tr>
<td>7</td>
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<td>0.17</td>
<td>26</td>
<td>671</td>
</tr>
<tr>
<td>8</td>
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<td>0.2</td>
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<td>0.21</td>
<td>14</td>
<td>650</td>
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<tr>
<td>10</td>
<td>San Fernando, 2/9/1971, Santa Anita Dam, 273</td>
<td>6.61</td>
<td>0.22</td>
<td>31</td>
<td>667</td>
</tr>
<tr>
<td>11</td>
<td>Northridge-01, 1/17/1994, LA - Chalon Rd, 70</td>
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<td>740</td>
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<tr>
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<td>Northridge-01, 1/17/1994, LA 00, 180</td>
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<td>0.26</td>
<td>19</td>
<td>706</td>
</tr>
<tr>
<td>13</td>
<td>Hector Mine, 10/16/1999, Hector, 0</td>
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<td>0.27</td>
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<td>726</td>
</tr>
<tr>
<td>14</td>
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<td>0.28</td>
<td>19</td>
<td>602</td>
</tr>
<tr>
<td>15</td>
<td>Loma Prieta, 10/18/1989, San Jose - Santa Teresa Hills, 225</td>
<td>6.93</td>
<td>0.28</td>
<td>15</td>
<td>672</td>
</tr>
<tr>
<td>16</td>
<td>Morgan Hill, 4/24/1984, Gilroy Array #6, 90</td>
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<td>0.29</td>
<td>10</td>
<td>663</td>
</tr>
<tr>
<td>17</td>
<td>Kobe, Japan, 1/16/1995, Kobe University, 90</td>
<td>6.90</td>
<td>0.31 [0.29]*</td>
<td>16</td>
<td>828</td>
</tr>
<tr>
<td>18</td>
<td>Tabas Iran, 9/16/1978, Dayhook, L</td>
<td>7.35</td>
<td>0.32</td>
<td>14</td>
<td>660</td>
</tr>
<tr>
<td>19</td>
<td>Loma Prieta, 10/18/1989, Gilroy - Gavilan Coll., 337</td>
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<td>0.33</td>
<td>10</td>
<td>730</td>
</tr>
<tr>
<td>20</td>
<td>Loma Prieta, 10/18/1989, San Jose - Santa Teresa Hills, 315</td>
<td>6.93</td>
<td>0.33 [0.26]*</td>
<td>15</td>
<td>672</td>
</tr>
<tr>
<td>21</td>
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<td>730</td>
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<td>0.38</td>
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<td>602</td>
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<tr>
<td>23</td>
<td>Coyote Lake, 8/6/1979, Gilroy Array #6, 320</td>
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<td>0.37 [0.42]*</td>
<td>18</td>
<td>714</td>
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<tr>
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<td>0.38</td>
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<td>706</td>
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<tr>
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<td>0.39 [0.31]*</td>
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<td>714</td>
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<tr>
<td>26</td>
<td>Tabas Iran, 9/16/1978, Dayhook, T</td>
<td>7.35</td>
<td>0.41</td>
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<td>660</td>
</tr>
<tr>
<td>27</td>
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<td>0.42</td>
<td>10</td>
<td>1428</td>
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<td>28</td>
<td>San Fernando, 2/9/1971, Lake Hughes #12, 21</td>
<td>6.61</td>
<td>0.43 [0.38]*</td>
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<td>602</td>
</tr>
<tr>
<td>29</td>
<td>Manji, Iran, 6/20/1990, Abbar L</td>
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<td>0.46 [0.51]*</td>
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<td>0.46</td>
<td>9</td>
<td>609</td>
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<tr>
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<td>Chi-Chi Taiwan, 9/20/1999, TCU045, E</td>
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<td>0.47</td>
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<td>705</td>
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<td>32</td>
<td>Loma Prieta, 10/18/1989, Gilroy Array #1, 90</td>
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<td>33</td>
<td>Kobe Japan, 1/16/1995, Nishi-Akashi, 0</td>
<td>6.90</td>
<td>0.48</td>
<td>9</td>
<td>609</td>
</tr>
</tbody>
</table>

*a[recorded]*

Site Amplification Factors and Sensitivity Analysis

Site amplification factors for the short period ($F_s$) and medium period ($F_m$) ranges were computed from the ratio of spectral acceleration between the surface and bedrock locations. A detailed analysis was carried out to find impact of variation in PI, soil strata depth above the bedrock and variation in bedrock properties on soil amplification factors for various site classes and EQ intensity levels. These steps are explained in the following sections.

Comparison with AASHTO Code

The computed soil amplification factors ($F_s$ and $F_m$) were compared with the AASHTO code values and conclusion were drawn in the last section.

COMPUTATION OF SOIL AMPLIFICATION FACTORS

Seismic design spectrum is constructed using 2 soil amplification factors (i.e. $F_s$ and $F_m$) in the AASHTO code [2]. $F_s$ and $F_m$ represent soil amplification in the short period (0
Researchers have proposed a variety of approaches to compute these amplification factors \([39–42]\). However, in the current study, the method proposed by \([43]\) and modified by \([44]\) was used to compute \(F_a\) and \(F_v\) values given by the following expressions:

\[
F_a = \frac{1}{0.4} \int_{0.1}^{0.5} \frac{R_{S_{\text{soil}}}(T)}{R_{S_{\text{rock}}}(T)} \, dT \quad (1)
\]

\[
F_v = \frac{\int_{0.1}^{0.5} \frac{R_{S_{\text{soil}}}(T)}{R_{S_{\text{rock}}}(T)} \, dT}{0.4} \quad (2)
\]

Herein, the term \(\frac{R_{S_{\text{soil}}}(T)}{R_{S_{\text{rock}}}(T)}\) represents the median value of acceleration spectral ratio for the suite of eleven seismic ground motions for a particular analysis case at a given earthquake intensity level (i.e. DBE, FEE or MCE). Equations 1 and 2 were numerically integrated to get the values of \(F_a\) and \(F_v\). It is to be noted that the \(F_a\) and \(F_v\) factors in the AASHTO code were computed using the same procedure \([45]\).

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**Figure 3:** Modulus reduction and damping curves for soils in site classes C and D.

**Figure 4:** Distribution of \(F_a\) and \(F_v\) values for 40 m deep strata for various soil classes, \(V_{\text{rock}}\), earthquake intensity and PI.

**Figure 5:** Distribution of \(F_a\) and \(F_v\) values for 110 m deep strata for various soil classes, \(V_{\text{rock}}\), earthquake intensity and PI.

*Note: Numbers on the X-axis represent \(V_{\text{rock}}\).*
Figs. 4 and 5 present variation in the values of $F_a$ and $F_v$ for the five site classes as a function of three levels of seismic intensity, PI and $V_{rock}$ for 40 m and 110 m deep strata respectively. It can be observed that $F_a$ and $F_v$ values showed considerable variation with respect to site class, $V_{rock}$, PGA and variation in PI. Influence of each of these parameters is discussed in the following section.

**SENSITIVITY ANALYSIS FOR SOIL AMPLIFICATION FACTORS**

This section analyses the sensitivity of soil amplification factors ($F_a$ and $F_v$) with respect to: (i) PI, (ii) $V_{rock}$, (iii) earthquake intensity, (iv) strata depth and (v) site classification.

In order to conduct this sensitivity analysis, the results shown in Figs. 4 and 5 were desegregated in Figs. 6 and 7 for $F_a$ and $F_v$ respectively. Figs. 6 and 7 present variation in $F_a$ and $F_v$ for the three considered levels of earthquake intensity respectively.

The amplification factors are presented side-by-side for two depths of strata for direct comparison. Code based values of $F_a$ and $F_v$ are also marked in these figures. Discussion on sensitivity of soil amplification factors for the above-mentioned parameters is presented below.

**Effect of PI**

It is well understood that soils with different PI values exhibit significantly varying shear stress-strain and damping behavior with increasing seismic acceleration [28 – 30]. However, the AASHTO code considers only $V_{s30}$ values while assigning site amplification factors. Therefore, soil index properties are not taken into account while selecting the values of these parameters despite the fact that soils with vastly different index properties can have the same $V_{s30}$.

The effect of variation in PI on soil amplification factors ($F_a$ & $F_v$) is examined by computing the percent difference in these

![Figure 6: Influence of PI and $V_{rock}$ on short period amplification factor ($F_a$) for various soil profiles of 40 m and 110 m deep strata with earthquake intensity.](image-url)
parameters for a particular analysis case of strata depth, earthquake intensity, site classification and $V_{\text{rock}}$. The following expression was used to compute the difference in which $F_a$ is taken as the example parameter:

$$\% \text{ diff. due to PI variation} = \frac{\max \text{diff in } F_a \text{ for 3 PI values}}{\text{avg } F_a \text{ for 3 PI values}}$$  \hspace{1cm} (3)$$

Fig. 8 presents the variation of this difference for $F_a$ and $F_v$ values for 40 m and 110 m strata. It was noted that the difference in $F_a$ due to variation in PI was less than 10% for site class $C_{\text{high}}$ for both strata depths for all values of $V_{\text{rock}}$. This variation was less than 10% for all cases in site class $C_{\text{avg}}$ for 40 m deep strata and for only three cases of 110 m strata, its value was slightly more than 20%. This means that variation in PI can be disregarded for computing $F_a$ for site classes $C_{\text{high}}$ and $C_{\text{avg}}$ for both strata depths.

However, the impact of variation in PI on $F_v$ values was very pronounced (20% to more than 100%) for site classes $D_{\text{high}}, D_{\text{avg}}$ and $D_{\text{low}}$ for all three levels of earthquake intensities except for DBE level earthquake in site class $D_{\text{high}}$ where the maximum difference was less than 20% for both strata depths. This implies that neglecting the influence of PI for site classes $D_{\text{high}}, D_{\text{avg}}$ and $D_{\text{low}}$ could lead to significant under/over-estimation of true $F_a$ values. This deviation increased with decreasing $V_{\text{soil}}$ of strata and increasing level of earthquake intensity.

Observations related to variation in $F_v$ are slightly different than those mentioned for $F_a$ and are depicted in Fig. 8(b). The variation in $F_v$ for site classes $C_{\text{high}}$ and $C_{\text{avg}}$ was more than that observed for $F_a$ but the value was less than 20%. For site classes $D_{\text{high}}$ and $D_{\text{avg}}$, the maximum difference was slightly more than 20% that was less than half the difference for $F_a$ values. $F_v$ values in site class $D_{\text{low}}$ were affected the most by the PI variation as the maximum difference was more than 60% for both strata depths. However, this difference was almost half that was noted for $F_a$ values.

![Figure 7: Influence of PI and $V_{\text{rock}}$ on medium period amplification factor ($F_v$) for various soil profiles of 40 m and 110 m deep strata with earthquake intensity.](image-url)
In conclusion, it is noted that $F_a$ values were affected more than $F_v$ values due to variation in PI. The effect of PI variation on $F_a$ and $F_v$ was negligible to low for site classes $C_{high}$ and $C_{avg}$. However, for site classes $D_{high}$, $D_{avg}$ and $D_{low}$, the effect of PI variation on $F_a$ was moderate to high and should not be neglected. Similarly, this effect was low to moderate for $F_v$ values in site classes $D_{high}$ and $D_{avg}$ while it was high for site class $D_{low}$.

**Effect of $V_{rock}$**

It was observed in Fig. 6 that $F_a$ values showed an increasing trend with increasing value of $V_{rock}$ for all site classes except $D_{low}$. Similar examination of Fig. 7 revealed that $F_v$ values exhibited the same trend for all site classes except $C_{high}$. This strong correlation between $V_{rock}$ and amplification factors ($F_a$ and $F_v$) has its theoretical background in the elastic wave theory. It has been shown that the maximum amplification corresponding to resonance in shear in a soil layer overlying a rock occurs approximately at the fundamental frequency of the soil layer and is approximately given by the following expression [46]:

$$\text{max. amplification} \approx \frac{1}{\left(\frac{1}{f_{peak}}\right)^2 + \gamma^2}$$

(4)

In this equation, ICR is the impedance contrast ratio defined as:

$$ICR = \frac{\gamma V_{rock}}{\gamma_s V_{strat}}$$

where $\gamma$ is the unit weight, $V_i$ is the shear wave velocity and subscripts $R$ and $S$ refers to parameters of the bedrock and the soil layer above it respectively. Whereas, $\beta$ is the damping ratio of the soil layer.

For the analysis cases investigated in this study, ICR varied between 1 and 4 for site class $C$ soils and between 1 and 18 for site class 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 [23].

Fig. 9 depicts the relationship between ICR and amplification factor ($F_a$ and $F_v$) for 40 m deep strata. Despite the scatter in the data points, a clear trend of increasing $F_a$ and $F_v$ values can be noted with increasing ICR. The coefficient of determination ($R^2$) had a value of 0.52 and 0.49 for $F_a$ and $F_v$ respectively, whereas, the coefficient of correlation ($R$) had a value of ±0.72 and ±0.70 for $F_a$ and $F_v$ respectively. As $R$-value is in between 0.5 and 0.8, therefore a moderate correlation exist between $F_a$ and $F_v$ with respect to ICR [47]. Amplification given by Eq. (4) for $\beta = 2\%$ and 20% are also plotted in Fig. 9(a) to provide a comparison with the theoretical expression.

The decreasing trend for $F_a$ values for site class $D_{low}$ in Fig. 6 was due to the dominant period of the soil strata having its peak outside the interval for which $F_a$ was computed. Similarly, $F_v$ values in Fig. 7 showed a weak correlation with $V_{rock}$ for site class $C$, which was due to the natural period of the strata being outside the interval over which $F_v$ was computed. However, $F_v$ values for soils $D_{avg}$ and $D_{low}$ exhibited a relatively stronger correlation with $V_{rock}$ as the natural period of the strata was within the interval over which $F_v$ was calculated.

**Effect of Earthquake Intensity**

Figs. 10 and 11 depicts the relationship between $F_a$ and $F_v$ with three levels of earthquake intensity (i.e. DBE, FEE and MCE) respectively for 40 m strata for various $V_{rock}$ values. Relationships for 110 m strata are similar and are not included herein. The following observations were made.

![Figure 8: Effect of PI on $F_a$ and $F_v$ for various earthquake intensities, site classes and $V_{rock}$](image-url)
Effect of Earthquake Intensity on $F_a$

$F_a$ values for site class C_high were very slightly affected by increasing level of earthquake intensity for all values of $V_{rock}$ and all three values of PI as depicted in Fig. 10. Similarly, for site class C_avg, there was no effect on $F_a$ values for lower values of $V_{rock}$, while a slight decreasing trend was observed for higher $V_{rock}$ values. For site class D_high, the trend was similar to site class C_avg but the trend for decrease in $F_a$ values with increasing earthquake intensity is moderate to high for higher values of $V_{rock}$. On the other hand, sharp decrease in $F_a$ values are observed with increasing earthquake intensity for all $V_{rock}$ values for site classes D_avg and D_low. This decrease in the amplification factor $F_a$ can be attributed to non-linearity in these

Figure 9: Relationship between Impedance Contrast Ratio (ICR) and soil amplification factors, $F_a$ & $F_v$ for 40 m deep strata.

Figure 10: Variation of $F_a$ with earthquake intensity for various $V_{rock}$ and site classes for 40 m strata.

Figure 11: Variation of $F_v$ with earthquake intensity for various $V_{rock}$ and site classes for 40 m strata.
weaker soils due to which high material damping was mobilized, which reduced the amplification factor.

**Effect of Earthquake Intensity on $F_v$**

Relationship between earthquake intensity and $F_v$ for various site classes and $V_{rock}$ values is presented in Fig. 11. Similar to $F_a$ values, $F_v$ values in site class C_high were also unaffected by level of earthquake intensity. $F_v$ values for site class D_avg exhibited a mixed trend of slightly increasing for lower $V_{rock}$ values and slightly decreasing for higher $V_{rock}$ values. Contrary to the trend for $F_a$ values, $F_v$ values for site classes C_avg and D_high showed an increasing trend with increasing earthquake intensity. This trend was more pronounced in site class D_high. $F_v$ values for site class D_low showed a decreasing trend with increasing earthquake intensity which was similar to the $F_a$ values for this site class. However, this decreasing trend reduced with increasing PI values for this site class.

**Effect of Strata Depth**

The influence of strata depth on the values of amplification factors ($F_a$ and $F_v$) was examined by plotting the ratio of the values for 110 m and 40 m strata as depicted in Fig. 12. The following observations were made.

*Figure 12: Influence of strata depth on short and medium period amplification factors $F_a$ and $F_v$ for various soil profiles.*

*Figure 13: Effect of site class on $F_a$ and $F_v$ for various earthquake intensities.*
Effect of Strata Depth on $F_v$

Referring to Fig. 12, it was observed that the influence of strata depth on $F_v$ values for site classes C_avg and D_avg is minor as the ratio is within ±10% of unity. There is a moderate influence of strata depth on $F_v$ values for site class D_high as the ratio is about +20% above unity. However, there is a strong influence of strata depth on $F_v$ for site classes C_high and D_low as the ratio is more than 20% above and below unity for these site classes respectively. It is to be noted that site class D_low is the only site class in which the ratio is below unity (i.e., values are more for 40 m strata) for all data points. This anomaly could be attributed to the higher ICR values for the 40 m deep strata compared to the 110 m strata.

Effect of Strata Depth on $F_v$

Examination of Fig. 12 for $F_v$ revealed that $F_v$ values for all site classes were moderately to strongly influenced by the strata depth. Moderate influence (+20%) was noted for site class C_high and some cases of C_avg. However, strong positive influence (+20%) was noted for site classes D_high and D_avg and a strong negative influence (>−20%) was observed for site class D_low. It was noted that site class D_low was the only site class in which the ratio was below unity (i.e., values were more for 40 m strata) for all data points. This anomaly could be attributed to the higher ICR value for the 40 m deep strata as compared to the 110 m strata.

Effect of Site Class

Effect of site class on amplification factors $F_v$ and $F_v$ for both strata depths was examined in Fig. 13 by plotting the maximum, median and minimum values of the amplification factors for a given earthquake intensity. The maximum, median and minimum values were determined for all PI and $V_{rock}$ values for a particular site class (e.g., C_high) and earthquake intensity (e.g., DBE).

It was observed that $F_v$ values for both 40 m and 110 m strata slightly increased from site class C_high to C_avg, then attained peak values for site class D_high and then showed a steady decrease for site classes D_avg and D_low. The trend for $F_v$ values was different from $F_v$, as $F_v$ values continued to increase from site classes C_high to C_avg to D_high and attained the peak values for site class D_avg. Afterwards, $F_v$ values decreased for site class D_low.

Effect of Soil Non-linearity on Site Amplification Factors

An equivalent linear 1-D site response analysis was carried out in the presented study. It is understood [48 – 50] that use of non-linear site response analysis may be required under certain situations of site class and seismicity level. It is demonstrated in [50] that the equivalent linear response is essentially the same as the non-linear response for peak soil shear strains ($\gamma$) of less than 0.1% and equivalent linear response analysis can be used with high confidence for such cases. For strains between 0.1% and 1%, the equivalent linear response starts to diverge from the non-linear response and equivalent linear response analysis should be used with caution. Whereas, for soil shear strains greater than 1%, use of non-linear response analysis is essential [50].

It was noted in the study presented herein that the median peak soil shear strain ($\gamma$) value for site classes C_high and C_avg was less than the lower threshold value of 0.1% for all PI values, ground motions and for both strata depths. Therefore, using equivalent linear response analysis is not expected to cause any discrepancy in the results for these site classes. For site class D_high and D_avg, $\gamma$ exceeded the lower threshold of 0.1% for most of the MCE ground motions in a limited depth (<6 m at the top) with a maximum median values of 0.16% and 0.3% for site classes D_high and D_avg respectively. However, for site class D_low, $\gamma$ exceeded the lower threshold of 0.1% for all ground motions and registered a maximum value of 0.8% for earthquake ID #32. This means that a non-linear response analysis may have resulted in different values of amplification factors for only a limited number of site classes under certain earthquake intensities. Therefore, the conclusions reached in this study are still valid despite the use of an equivalent-linear seismic site response analysis.

Summary of the Sensitivity Analysis

Tables 6 and 7 summarize results of the sensitivity analysis for $F_v$ and $F_v$ respectively for various parameters based on the discussion presented earlier in the section. The influence of a parameter on soil amplification factors is quantified as follows: Ignore (<10%), Low (10−20%), Moderate (20−50%), High (50−80%), Very high (>80%). The following observations were made:

i - Referring to Fig. 8, it was noted that $F_v$ and $F_v$ values for soil profiles in site class C were unaffected due to variation in the PI values, whereas these values for soil profiles in site class D were highly dependent on the PI values. The influence of PI on $F_v$ and $F_v$ values increased with decreasing $V_{so}$ of the site classes.

ii - It was observed in Figs. 6 and 7 that variation in $V_{rock}$ had a high influence on $F_v$ values for soil profiles in site class C and for $F_v$ values in site class D, while it had low to moderate influence on $F_v$ values in site class D and $F_v$ values in site class C. The influence of $V_{rock}$ on $F_v$ values was opposite to that of the PI and its impact decreased with decreasing $V_{so}$ of the site class. Whereas, for $F_v$ values, $V_{rock}$ influence increased with decreasing $V_{so}$ of the site class.

iii - As depicted in Figs. 10 and 11, variation in earthquake intensity had high influence on $F_v$ values in site class D only.

### Table 6: Summary of sensitivity analysis for $F_v$

<table>
<thead>
<tr>
<th>Site Class</th>
<th>PI</th>
<th>$V_{rock}$</th>
<th>Earthquake intensity</th>
<th>Strata depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>C_high</td>
<td>Ignore</td>
<td>High</td>
<td>Ignore</td>
<td>Moderate</td>
</tr>
<tr>
<td>C_avg</td>
<td>Ignore</td>
<td>High</td>
<td>Ignore</td>
<td>Moderate</td>
</tr>
<tr>
<td>D_high</td>
<td>Moderate</td>
<td>High</td>
<td>Low</td>
<td>Moderate</td>
</tr>
<tr>
<td>D_avg</td>
<td>High</td>
<td>Moderate</td>
<td>High</td>
<td>Ignore</td>
</tr>
<tr>
<td>D_low</td>
<td>Very high</td>
<td>Ignore</td>
<td>High</td>
<td>Moderate</td>
</tr>
</tbody>
</table>

### Table 7: Summary of sensitivity analysis for $F_v$

<table>
<thead>
<tr>
<th>Site Class</th>
<th>PI</th>
<th>$V_{rock}$</th>
<th>Earthquake intensity</th>
<th>Strata depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>C_high</td>
<td>Ignore</td>
<td>Low</td>
<td>Ignore</td>
<td>Moderate</td>
</tr>
<tr>
<td>C_avg</td>
<td>Ignore</td>
<td>Moderate</td>
<td>Low</td>
<td>Moderate</td>
</tr>
<tr>
<td>D_high</td>
<td>Moderate</td>
<td>High</td>
<td>Low</td>
<td>Moderate</td>
</tr>
<tr>
<td>D_avg</td>
<td>Low</td>
<td>High</td>
<td>Low</td>
<td>Moderate</td>
</tr>
<tr>
<td>D_low</td>
<td>High</td>
<td>Moderate</td>
<td>Moderate</td>
<td>Moderate</td>
</tr>
</tbody>
</table>

**Key:**
- Ignore (<10%)
- Low (10−20%)
- Moderate (20−50%)
- High (50−80%)
- Very high (>80%)

Value of $F_v$ or $F_v$ increases with increasing value of the sensitivity parameter unless noted by the indicator in parenthesis.
and its influence was low to moderate for $F_v$ values in all site classes. The trend of the earthquake intensity influence was similar to that of PI and its impact increased with decreasing $V_{30}$ of site class for both $F_v$ and $F_s$ (refer to Figs. 6 and 7). This higher influence of earthquake intensity on $F_s$ and $F_v$ values in site class D could be due to higher level of shear strains induced in this site class as noted in Section 4.6.

iv- Strata depths considered in the study had a low to moderate influence on $F_s$ and $F_v$ values for all site classes. The effect of strata depth could be even ignored for $F_v$ values in site classes C_avg and D_avg. No discernible trend in the influence of strata depth on $F_s$ and $F_v$ values was noticed with respect to $V_{30}$ of the strata.

**COMPARISON OF COMPUTED VALUES OF AMPLIFICATION FACTORS WITH AASHTO CODE**

Median values of soil amplification factors ($F_s$ and $F_v$) as computed earlier were compared with the values stipulated in the AASHTO code in Fig. 14. In these figures, median values for 40 m and 110 m strata were compared with the code values. Additionally, soil amplification factor values for parameters similar to the ones that were used to derive the $F_v$ values, i.e., strata depth of 40 m, PI =15 and $V_{lock} = 760 - 900$ m/s [3], were also plotted in this figure. The following observations were made:

**$F_s$ Values [Refer to Fig. 14(a)]**

i- Median values of amplification factor $F_s$ computed in this study were 50% to 160% more than the code values for site classes C_high, C_avg, D_high and D_avg.

ii- Median values of $F_s$ computed in this study for site class D_low were 30% to 60% smaller than the code values.

iii- $F_s$ values computed for parameters similar to code conditions were within +11 % to +65% of the code values for site classes C_high, C_avg, D_high and D_avg. and -18% to -40% for site class D_low. This implies that the procedure adopted in the study is reliable and the values reported for other cases and the conclusions drawn from the sensitivity analysis are valid.

iv- Smaller values of $F_s$ for the weakest site class (D_low) as compared to the code value are also no surprise as others had also reported similar findings [11, 13]. Physical explanation for this fact may be the increased soil shear strains and soil non-linearity as noted earlier.

**$F_v$ Values [Refer to Fig. 14(b)]**

i- $F_v$ values exhibited less variation w.r.t. code values as compared to the $F_s$ values. Most of the computed values were smaller than the code values by 1% to 42% for all site classes except site class D_avg for which the computed values were 3% to 34% higher than the code values.

ii- $F_v$ values computed for parameters similar to code conditions were within -7% to -50% of the code values for all site classes.

**CONCLUSIONS**

The following conclusions are drawn from this study, which are applicable only to the site and geotechnical parameters considered in the study. Extrapolation to other site and geotechnical conditions should be done with caution while exercising engineering judgement.

i- It is concluded from the results of more than 3950 1-D site response analysis, computation of soil amplification factors and results of sensitivity analysis that the soil amplification factors for AASHTO site classes C and D showed varying degree of dependence on geological setting and geotechnical properties as well as strata depth and earthquake intensity.

ii- Bedrock properties and soil PI were found to be the two most influential parameters affecting soil amplification factors, $F_s$ and $F_v$. Bedrock properties affected the $F_s$ and $F_v$ values for sites with higher $V_{30}$ while variation in PI influenced these parameters for sites with smaller $V_{30}$.

iii- Earthquake intensity did not have an appreciable influence on $F_s$ and $F_v$ values for sites with higher $V_{30}$ but for other sites, $F_v$ values decreased with increasing earthquake intensity. However, earthquake intensity did not affect $F_s$ values for sites with lower $V_{30}$.

iv- Strata depth had a low to moderate influence on $F_s$ and $F_v$ values in all site classes except for site classes C_avg and D_avg for which its effect on $F_v$ values could be ignored. However, this conclusion was based on the study conducted on two strata depths (i.e. 40 m and 110 m) only. Additional strata depths should be analysed to fully appreciate the influence of strata depth on $F_s$ and $F_v$.

v- Soil amplification factors computed for conditions similar to the one used for finding these factors in AASHTO code varied between 11 – 60% for $F_s$ and -7 – -48% for $F_v$ with an average variation of 35% and -27% for $F_s$ and $F_v$ respectively. It is to be noted that the computed $F_v$ values were generally lower than the code values.
vi—There is a need to include a disclaimer in the AASHTO code for use of the specified $F_1$ and $F_v$ values for site conditions not included in the derivation of these factors. Users should be asked to seek alternative soil amplification factors for sites with harder bedrock ($V_{rock} > 900$ m/s), strata depth other than 30 – 40 m and PI different than 15-30.

ACKNOWLEDGEMENTS

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