SEASONAL EFFECTS ON SEISMIC PERFORMANCE OF HIGH RISE BUILDINGS CONSIDERING SOIL-STRUCTURE INTERACTION

Navid YEGANEH1, Behzad FATAHI2

ABSTRACT

The Seismic Soil-Structure Interaction (SSSI), which is a tangled phenomenon, is concerned with the shear waves in preference to the longitudinal waves on account of a prevalent greater energy content in the former. The need for the high rise buildings in the megalopolises results in the paramountcy of the seismic soil-foundation-building interaction analysis in order to achieve the reliable predictions and mayhap curtail the severe damage and probable partial or total collapse of the superstructures. The seasonal effects could influence the soil moisture content particularly in the vadose zone near the surface, exacerbated by the climate change effects, inducing more frequent floods and drought. Wherefore, a soil-structure model was evaluated in this study, subjected to the soil moisture variations in the vadose zone, by utilizing the 3D finite difference modeling technique through the fully nonlinear dynamic analysis in the time domain considering SSSI during the 1994 Northridge earthquake. In particular, the objective was probing the possible effects of the selected degree of saturation (Sr) values, i.e., 5%, 17.5%, 60%, and 100%, for the noncohesive soil, named “Glacier Way Silt”, in conjunction with the small-strain shear moduli on the seismic performance and its corresponding damage of a 20-story reinforced concrete moment-resisting building frame. It is of note that the said values of Sr were employed for the common 4-m zone of influence in Australia, being a sequel of the natural and artificial wetting-drying cycles. Get to the point, it was concluded that the season, in which an earthquake befalls, is stark prominent insomuch as it is potent to impact the extend of the damage in a superstructure.

Keywords: Soil-structure interaction; Direct method; Nonlinear seismic analysis; Degree of saturation; FLAC3D

1. INTRODUCTION

Be it static or dynamic conditions, the shear modulus at the mightily small strains, below 10^-3% (Georgiannou 1991), is the salient soil property, controlling the behavior of the geotechnical structures on the strength of the fact that the soil, by and large, is the weakest material, involved in the common geotechnical engineering projects. Even though the calculation of the small-strain soil shear modulus (G₀ or G_max) via Equation 1 is per se decent, not directly capturing the impacts of the variation in the degree of saturation was the fountain of conducting the current research project owing to being at variance with the conspicuous change in the moisture content of a soil in the wet and dry season. Note that, in Equation 1, ρ is the soil mass density, and Vₛ is the shear wave velocity of the soil mass; the one and the other immensely depend on the soil moisture content.

\[
G₀ = G_{\text{max}} = \rho Vₘ^2
\]  

(1)

In the wake of the development attributed to the computer analysis tools, the soil-structure interaction throughout agitation, i.e., earthquake, that is referred to the process in which the soil response is told on by the structure motion whereas the latter is affected by the soil motion, has moved to center stage (Sáez et al. 2011; Yeganeh et al. 2015; Nguyen et al. 2016; Rahnema et al. 2016; Fatahi et al. 2018); nonetheless, a myriad of suppositions would give rise to diminishing the quality of the garnered results.

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In that regard, what is being done on a daily basis by the practicing engineers is directly capitalizing on the measured shear wave velocity profiles, derived from the field tests, such as the shooting up hole, shooting down hole, and cross-hole shooting. Over and above to the fact that the pretense of simplicity legitimizes putting aside the changes associated with the degree of saturation, taking place amidst the wetting-drying cycles in the vicinity of the ground level at the vadose zone. On the contrary, experimentally parsing the correlation between the water content and maximum soil shear modulus has shed light on the key role of the degree of saturation in the estimation of $G_0$ (Mendoza et al. 2005; Pham Ngoc et al. 2017). By the same token, Wheeler et al. (2003) adverted to the miscellaneous mechanical behaviors concomitant with the two samples of the same soil, having the same void ratio, with the proviso that the aforesaid samples enjoy the different values for the degree of saturation.

The 3D finite difference numerical modeling was performed in FLAC3D-V5.0 in order to discern whether the ascent or descent in the degree of saturation, appertaining to the vadose zone, could bring forth the markedly shift connected with the performance level of a superstructure in accordance with the Performance-Based Seismic Design (PBSD). To fulfill the foregoing, the results of the soil-structure interaction system, to wit, the base shear, shear force distribution along the building height, response spectra, rocking of the foundation, transient structural lateral deformation, and inter-story drift ratio thereof, were flatly scrutinized.

2. CHARACTERISTICS OF ADOPTED NUMERICAL MODELS

The finite difference software, FLAC3D, was leveraged so as to numerically simulate the soil-shallow foundation-superstructure system by dint of having the capability of the FISH (shorten for FLACish) programming, to say nothing of being able to mimic the complex problems, warranting the voluminous computational memories.

2.1 Overview of Soil-Foundation-Structure System

Insofar as the soil-structure interaction is concerned, the direct method, in which the whole system of the soil-foundation-superstructure is modeled followed by being analyzed in one single step, accents the weightiness of the nonlinear behaviors connected with a structure and the underlying soil. Stated succinctly, a hypothetical 3D, 20-story (60 m height), 3-span (12 m length) reinforced concrete moment-resisting building frame, designed in SAP2000-V14, plus the 0.25-m thick concrete slabs along with a 14 m×14 m×1 m building foundation, placed on a 30-m thick soil deposit, were considered in the current study. The details of the designed sections concomitant with the 20-story superstructure, adopted in the 3D finite difference modeling, were tabulated in Table 1. It should be noted that the Australian codes, viz, AS/NZS1170.1 (Standards Australia/Standards New Zealand 2002), AS1170.4 (Standards Australia 2007), and AS3600 (Standards Australia 2009), plus AS2870 (Standards Australia 2011), were harnessed with a view to designing the superstructure and foundation thereunto with the elastic-perfectly plastic model, acting as a precursor to making the concrete material behave linear-elastically, provided that the defined yield stress was not breached. In that behalf, a plastic hinge is the epitome of a discontinuity in the rotational motion once the plastic moment capacity ($M_p$), otherwise known as the resisting moment, ciphered out by employing 32 MPa for the concrete compressive strength ($f_c$), was surpassed by the moment, being transmitted between the structural elements. The bulk and shear moduli, plus the mass density of the concrete material were $1.67 \times 10^{10}$ N/m$^2$, $1.25 \times 10^{10}$ N/m$^2$, and 2400 kg/m$^3$, respectively.

Table 1. Designed sections for structural beams and columns of concrete moment-resisting building.

<table>
<thead>
<tr>
<th>Story No.</th>
<th>Column (m×m)</th>
<th>Beam (m×m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>16-20</td>
<td>0.70×0.70</td>
<td>0.55×0.55</td>
</tr>
<tr>
<td>11-15</td>
<td>0.75×0.75</td>
<td>0.60×0.60</td>
</tr>
<tr>
<td>6-10</td>
<td>0.80×0.80</td>
<td>0.65×0.65</td>
</tr>
<tr>
<td>1-5</td>
<td>0.85×0.85</td>
<td>0.70×0.70</td>
</tr>
</tbody>
</table>
The P-Δ effect accompanying the structural components cracking are the indispensable parts for executing the dynamic analysis of a building. Henceforth, the former was heeded by activating the large-strain mode in the software of interest whilst the latter was fairly covered throughout multiplying the second moment of the area associated with the uncracked sections \( l_g \) by the modification coefficients attributed to the slabs, beams, and columns, put forward by ACI318 (ACI Committee 2014), i.e., 0.25, 0.35, and 0.7, consecutively. Apropos the damping for the building and its foundation, there is notable leeway to make use of the local damping coefficient \( \alpha_L \) of 0.157, engendering the 5% damping \( D \), by the use of Equation 2 (Itasca Consulting Group 2012), on grounds of the fact that the hysteretic behavior could be subsumed in the seismic analyses by the foregoing.

\[
\alpha_L = \pi D
\]  

\( (2) \)

It is common knowledge that the interface between a foundation and the soil would witness the sliding or separation during a dynamic analysis. Thereupon, the normal and shear spring stiffness values, i.e., \( k_n \), and \( k_s \), expressed in stress-per-distance units, were estimated as per the good rule-of-thumb, revolving around setting the aforesaid parameters to ten times the equivalent stiffness of the stiffest neighboring zone, posited by Itasca Consulting Group (2012).

Figure 1 depicts the schematic of the established numerical model in the current project. The free field boundary conditions, whose noteworthiness is crystal clear in mitigating the wave reflections, were assigned to the four vertical lateral boundaries, which ought to be placed at a befitting distance from each others, equal to five times the width of a superstructure (Rayhani and El Naggar 2008; Hokmabadi and Fatahi 2016), which was herein 70 m, as indicated at Part F in Figure 1. It is noteworthy to state that the bearing capacity of the mat foundation satisfied the minimum required factor of safety (FOS) of 3, estimated based on the Meyerhof method (Meyerhof 1963).

In this study, the adopted pre-yield hysteretic damping combined with the Mohr-Coulomb plastic criterion resulted in a nonlinear elastic-perfectly plastic constitutive model. It is of interest to notice that the tangent shear modulus is being updated at each calculation step by implementing the hysteretic damping in FLAC, betokening the degradation of the soil stiffness with the shear strain withal the unloading-reloading behavior in full compliance with the Masing rules. In that light, the two-variable function (Equation 3), termed “Default Model”, was employed, whose parameters, i.e., \( L_1 \), and \( L_2 \), were -3.325, and 0.823 (Itasca Consulting Group 2012), respectively. Likewise, \( \gamma \) is the shear strain in the equation below. In this research project, Glacier Way Silt (Wu et al. 1984), which is a cohesionless soil deposit, was used with the assumed soil parameters as friction angle = 29°, and dilation angle = 0°. The remainder of the required parameters are painstakingly presented in Section 2.2.
It is not unfair to accentuate that the preponderance of seismic codes and regulations foregrounded the preeminence of the shear wave velocity in the first 30 m of a site subsoil. It follows that considering a 30 m depth for the seismic bedrock is a legacy of transpiring the chief part of the amplification and/or attenuation in the aforementioned depth. It is now standard practice for the seismic analysis to apply the acceleration record to the seismic bedrock while performing a time history analysis. The baseline-corrected near-field earthquake excitation, named “1994 Northridge earthquake (USA)”, portrayed in Figure 2, was picked to apply at the base of all the considered numerical models in this paper. Moreover, it is manifest to truncate the length of the cherry-picked excitation with the aim of surmounting the sizable effort in carrying out such nonlinear dynamic analyses.

![Figure 2. Input seismic base motion](image)

### 2.2 Adopted Shear Wave Velocity Profiles

Qiu and Fox (2008) postulated the concept of the effective density ($\rho_{\text{eff}}$), which curbs the velocity of a small-strain shear wave. It is to be asserted that $\rho_{\text{eff}}$ emerges from the relative motion between the solid phase, disporting the soil grains, and fluid phase, representing the pore water, midst the passage of a shear wave in a saturated soil. In the cited paper, the determination of the momentousness attributed to the effective soil density for a given application could be done by taking account of the hydraulic conductivity, effective grain size, and wave frequency. In turn, the effective density is espoused to be equal to the saturated density for the low hydraulic conductivity saturated soils, such as the clays and silts. As a consequence, in this paper, the use of the effective density was neglected in the fully saturated mode courtesy of Glacier Way Silt as the soil type. Additionally, scorning the effective density for the seismic waves, propagating through the saturated soil medium, is generally well-grounded (Santamarina et al. 2001). All told, the soil mass density in Equation 1 was directly quantified by Equation 4 whilst considering the different values for the degree of saturation. $G_s=2.66$, $S_r, e=0.65$, and $\rho_w=1000 \text{ kg/m}^3$ in Equation 4 are the specific gravity of the soil solids, degree of saturation, void ratio, and water mass density, respectively.

$$\rho = \frac{(G_s + S_r e) \rho_w}{1 + e}$$  \hspace{1cm} (4)

The second parameter in Equation 1 is the shear wave velocity, being a requisite for rendering the dynamic analyses of the soil-structure interaction problems. It is of vital importance that the field condition will not abide from the time when the in-situ tests were run till the end of the building’s service.
life; hence, the measured values of $V_s$ ought to be altered by means of the calculated values of $G_0$ in line with the possible range for the degree of saturation. Proceeding on that track, the zone of influence, pertaining to the variation in the water content or the change of the degree of saturation for a soil layer near the ground surface owing to the rain, drying, and inundation, plus the loading-unloading process, was assumed to be 4 m with due attention to the recommended values in AS2870 (Standards Australia 2011).

Wu et al. (1984) proposed the empirical equation, i.e., Equation 5, invoking a curve-fitting procedure, for the normalized small-strain shear modulus ($G_0(\text{moist})/G_0(\text{dry})$) versus the degree of saturation on the basis of executing a set of resonant column tests. The aforesaid trend might have an apex, demonstrating the optimum degree of saturation ($S_{r(\text{opt})}$), on the condition that the clay fraction is infinitesimal; otherwise, the highest value of $G_0/G_0(\text{dry})$ would come into picture around the dry state (Santamarina 2003; Dong and Lu 2016). On the same topic, the data for $S_{r(\text{opt})}$ versus the logarithm of the effective grain size ($D_{10}$) fell nearly on a straight line for the five test materials, including Glacier Way Silt, Glacier Way Sand, Beal Sand, and Brazil Sand, plus Soil 3, which contained the appreciable amounts of shells and mica. Therefore, Equation 6 (Wu et al. 1984), as an empirical equation, was obtained so as to predict the optimum degree of saturation ($S_{r(\text{opt})}$), corresponding to the peaks of the shear wave velocity and small-strain shear modulus or the maximum value of $G_0/G_0(\text{dry})$ for the soils with both the rounded and angular grains.

\[
G_0 = [1 + H(S_r)]G_0(\text{dry})
\]

\[
S_{r(\text{opt})} \, \% = -6.5 \log_{10} D_{10} + 1.5
\]

The value of $G_0$ at any degree of saturation could be established by a class of expressions, illustrated below, along with Equation 5. Furthermore, Equations 5 and 7 possess $H(S_r)$ as some functions connected with the degree of saturation, $G_0(\text{dry})$ as the low-amplitude shear modulus for the completely dry condition, $a$ as the maximum value of $G_0/G_0(\text{dry})$, $b$ as the optimum degree of saturation ($S_{r(\text{opt})}$), and $S_r$ as the degree of saturation expressed in percent. Besides, $H_1(S_r)$ and $H_2(S_r)$ are delineated via Equations 8 and 9.

\[
H(S_r) = \begin{cases} 
(a - 1) \sin \frac{\pi S_r}{2b}, & S_r \leq b \\
(a - 1)[H_1(S_r)][H_2(S_r)], & S_r > b 
\end{cases}
\]

\[
H_1(S_r) = \frac{1}{2} \left( \frac{S_r - b}{100 - b} \right)^2
\]

\[
H_2(S_r) = \sin \left\{ \frac{\pi}{100 - b} \left[ S_r + 50 - \frac{3}{2} b \right] \right\} + 1
\]

That is to say, the optimum degree of saturation for Glacier Way Silt was reported by Wu et al. (1984) to be hovering around 17.5%, in a good agreement with 18.5%, computed by means of Equation 6 plus applying $D_{10}$, equaling 0.0024 mm. To expound on the variation of $G_0/G_0(\text{dry})$ against the degree of saturation, it should be borne in mind that by traveling on the trend whereas aiming $S_{r(\text{opt})}$, the $G_0/G_0(\text{dry})$ ratio proliferates, undergoing the depletion towards the highest possible degree of saturation, intimating the fully saturated state. Consequently, Equation 5 and parameters therefrom were put into action by considering the four different values of $S_r$, encompassing 5%, 17.5%, 60%, and 100%, covering the two branches and acme associated with the trend of $G_0/G_0(\text{dry}) \cdot S_r$.

All the parameters, triggering the changes in the $V_s$ values for the 4-m depth, construed as “zone of influence”, could be reviewed in tabular form via Table 2. Forby, $G_0(\text{dry})$ was adopted as 57.8 MPa as reported by Wu et al. (1984) for Glacier Way Silt. In addition, Figure 3(a) is a pictorial description of the established shear wave velocity profiles in the current research whilst the caption on Figure 3(b) implies the soil mass density profiles on the basis of Equation 4.
Table 2. Adopted soil parameters in top 4-m vadose zone influenced by degree of saturation.

<table>
<thead>
<tr>
<th>Sr (%)</th>
<th>p (kg/m³)</th>
<th>a</th>
<th>b</th>
<th>H(Sr)</th>
<th>H₁(Sr)</th>
<th>H₂(Sr)</th>
<th>G₀/G₀(dry)</th>
<th>G₀(dry) (MPa)</th>
<th>G₀ (MPa)</th>
<th>Vₛ (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>1630</td>
<td>2.05</td>
<td>17.5</td>
<td>0.456</td>
<td>-</td>
<td>-</td>
<td>1.456</td>
<td>57.8</td>
<td>84.2</td>
<td>230</td>
</tr>
<tr>
<td>17.5</td>
<td>1680</td>
<td>2.05</td>
<td>17.5</td>
<td>1.050</td>
<td>-</td>
<td>-</td>
<td>2.050</td>
<td>57.8</td>
<td>118.5</td>
<td>270</td>
</tr>
<tr>
<td>60</td>
<td>1850</td>
<td>2.05</td>
<td>17.5</td>
<td>0.133</td>
<td>0.133</td>
<td>0.952</td>
<td>1.133</td>
<td>57.8</td>
<td>65.5</td>
<td>190</td>
</tr>
<tr>
<td>100</td>
<td>2000</td>
<td>2.05</td>
<td>17.5</td>
<td>0</td>
<td>0.500</td>
<td>0</td>
<td>1</td>
<td>57.8</td>
<td>57.8</td>
<td>170</td>
</tr>
</tbody>
</table>

Figure 3. (a) Adopted shear wave velocity profiles; (b) Variations of soil mass density with depth for selected values of degree of saturation

3. RESULTS AND DISCUSSIONS

As might be expected, both the cohesionless and cohesive soil layers in the vicinage of the ground surface for sundry reasons consistently encounter the change in the degree of saturation, being the backbone of the G₀ estimation in the geotechnical earthquake engineering field. In that view, the FLAC history records, exhibiting the changes in the variables in question as the time-stepping proceeds, were leveraged in order to finalize the results, hereinafter presented as well as coming under scrutiny.

In the first instance, the envelope of the shear forces on all the building levels for the four models, i.e., Sr=5%, Sr=17.5%, and Sr=60%, plus Sr=100%, as displayed in Figure 4, under the applied earthquake accelerogram, bore a close resemblance in order that the maximum difference was roughly 2.5 MN, taking place on the level 19. Such a fine difference would be set forth throughout the response spectra,
exposed in Figure 5 considering a 5% damping ratio, in conjunction with the fundamental translational period of the structure ($T'$), and natural period of the vibration concomitant with the soil deposit ($T''$), plus the second mode period of the entire SSI system ($T_2$). Making use of the handy rule-of-thumb that the fundamental period of an N-story building is approximately $N/10$ sec (Bungale 2016), plus Equation 10 (Kramer 1996) aroused 2 and 0.4 seconds for $T'$ and $T''$, respectively, since the soil deposit thickness ($H$), and weighted average shear wave velocity ($\bar{V}_s$) in Equation 11 (ASCE Standard 2016) were 30 m and about 300 m/s, consecutively. As a result, the fundamental period of the SSI model, i.e., $T_1 = T' + T''$, was 2.4 seconds. It is previously reported that the second mode period is often bracketed by the one-quarter and one-third of the fundamental period so long as the building frame is regular (NIST 2011). Accordingly, $T_2$ was estimated rather 0.6 to 0.8 seconds. Furthermore, Equation 11 has $d_i$ and $V_{si}$, named the thickness of any layers from 0 to 30 m depth and shear wave velocity of each layer in the said depth.

$$T' = \frac{4H}{\bar{V}_s}$$  \hspace{1cm} (10)

$$\bar{V}_s = \frac{\sum_{i=1}^{n} d_i}{\sum_{i=1}^{n} \frac{d_i}{V_{si}}}$$  \hspace{1cm} (11)

In general, the contribution of the first and second modes would signpost the seismic energy towards generating the shear forces in a superstructure. In that respect, all the trends in Figure 5 coincided at $T_1$,
i.e., 2.4 seconds, while the Sr=17.5% trend sat atop the others in the second mode period range, i.e., 0.6-0.8 seconds, being responsible for locating the shear force distribution of the model, having the optimum degree of saturation, on the far right side of Figure 4. A conclusion was reached on the occurrence of the substantial fluctuation by inspecting the trends along the superstructure height. The conventional belief (Veletsos and Meek 1974; Stewart et al. 2003) revolves around that the SSI system immunes to the higher modes of the vibration, connoting that only the structural parameters of the fundamental mode are needed for the analysis of SSI. Contrarily, the upsurge in the shear forces happened on the upper levels in Figure 4, signifying the irrevocable impacts of the structure’s higher modes on its dynamic response in the SSI analysis. Moreover, the lateral sliding resistance of the foundation under the applied earthquake was checked. Considering the foundation-soil interface friction angle of 26 degrees, corresponding to the friction coefficient of 0.5 as recommended by Applied Technology Council (ATC) (1997), and the total dead load, totaled up from the weight of the building and foundation, the sliding resistance was well more than the base shear induced by the earthquake, whose maximum value was approximately 9.5 MN (see Figure 4), i.e., FOS against sliding being 2.

The foremost point of departure between the SSI models and fixed-base ones is the consideration of the rocking component associated with a foundation slab, potentially engendering a significant portion of the superstructure lateral deflection. It is reasonable to believe that a foundation rocking is a corollary of the alternate settlement and uplifting, coming off on the two sides of a foundation in light of the seismic excitation-induced inertial forces in a building frame. The foundation rocking, on the whole, induces lessening the energy, stemming from the seismic wave, that pinpoints a superstructure in an earthquake-prone zone. Indeed, the rocking of a foundation depends on both the overturning moments, induced by the shear forces in the superstructure, plus the stiffness and strength attributed to the underlying soil deposit. Said otherwise, the higher shear forces in a structure provoke the more foundation rocking whereas the higher soil stiffness results in the lower rocking. Referring to Figure 4, in spite of the fact that the shear forces in the structure and therefore the foundation overturning moments were largest when Sr=17.5% was adopted, the soil stiffness for this case was also the highest (see Table 2). Continuing on that line, the largest rocking, appertaining to the Sr=5% case, as observed in Figure 6, could be utterly demystified by knowing that the shear forces, reported in Figure 4, interacted with the soil deposit with the given stiffness as in Table 2.

Referring to Figure 6, it is evident that the change in Sr from 5% to the fully saturated state not only resulted in the fourfold alleviation but also a nearly 25-second shift in the occurrence time of the maximum foundation rocking. For the sake of simplicity, the soil friction angle was presumed to be constant amid all the seismic analyses for the noncohesive soil of interest. The aforementioned assumption is in agreement with how the practicing engineers are grappling with the soil dynamics problems. Yet, taking a step further, the main aim of the current study was explicating how the seismic performance of a high rise building would be told on by the variation connected with the small-strain...
shear modulus, embraced by the changes in the degree of saturation. In addition to the effect of the foundation overturning moments, ascending $G_0$ by descending the degree of saturation would enforce the soil plasticity in the lower cyclic shear strains in a soil medium with the Mohr-Coulomb plastic yardstick as a soil constitutive model. It stands to reason that when the soil plasticity commences at the lower cyclic shear strains, the more permanent soil deformation under the cyclic loading would be expected as can be noted in Figures 7(a) and 7(b). It has to be kept in mind that taking place the excessive soil plasticity from the onset of the loading could basically force the differential settlement to ratchet down, which happened in the $S_r=17.5\%$ case.

At this juncture, the transient lateral displacements of the building frame by putting the variation of degree of saturation into practice, when the maxima took place at the top level, were sketched in Figure 8. Thus, each single trend instanced the authentic deformation, experienced by the high rise building. What is more, the lateral deflections in the models of interest were relative to the movements of the concrete foundation on the soil surface. Henceforth, the lateral displacements of the building, depicted in Figure 8, were courtesy of the combination concomitant with the foundation rocking and structural distortion. As it is discerned in Figure 8, the model, having the smallest value of the degree of saturation, i.e., $S_r=5\%$ in the vadose zone, hit the 1.3-m lateral displacement, which outnumbered those of the other models, in accord with the foundation rocking angles, shown in Figure 6.

The thought-provoking point in Figure 8 was that there was a well-nigh coincidence of deflections between the results of $S_r=60\%$ and $S_r=100\%$. In point of fact, the shear force-induced building distortion in the $S_r=100\%$ model compensated the 0.33-degree difference between the foundation rocking values of the above-mentioned models. In the bargain, Figure 9 conveyed the outstanding structural distortion, taking place in the fully saturated state.

![Figure 7. Vertical displacement histories associated with: (a) Right side of foundation; (b) Left side of foundation](image-url)
In the Performance-Based Seismic Engineering (PBSE) methodology, it is imperative that the inter-story drift ratio, as the most germane parameter to evaluate the seismic performance of a building, be compared with the 2% Life Safety (LS) yardstick according to FEMA273 (Applied Technology Council (ATC) 1997); otherwise, the structure design must be overhauled so as to garner the PBSE goal line. As exemplified in Figure 9, excelling the degree of saturation sparked off the safety-threatening design inasmuch as the outcomes concomitant with the optimum degree of saturation, i.e., \( S_r = 17.5\% \), and higher values, i.e., \( S_r = 60\% \) and \( S_r = 100\% \), emplaced in the safe demesne based on the LS touchstone. In view of the fact that the inter-story drifts generally follow the same pattern as the lateral deflections, the structural distortion in the fully saturated model was lucidly in agreement with the explanation, pronounced for Figure 8, which hinged around the rationale behind why the lateral deflections of models with \( S_r = 60\% \) and \( S_r = 100\% \) were more or less similar. In Figure 9, the drift ratios were reckoned via Equation 12 as to AS1170.4 (Standards Australia 2007), where, \( d_i \) and \( d_{i+1} \) are the deflections at the \( (i) \)th and \( (i+1) \)th levels, respectively, and \( h \) is the story height.

\[
\text{Drift} = \frac{(d_{i+1} - d_i)}{h}
\]  

On turning the pages of this paper, the focal role of the seasonality of the earthquakes became readily apparent. To give further details, the variation of the degree of saturation near the ground surface would emphatically effectuate the response of an entire SSI system. Thus, assigning the soil properties, e.g., the shear wave velocity, acquired from the field and/or laboratory tests, to the numerical models in compliance with the spasmodic changes, occurring in the course of the superstructures’ useful life, should be taken into account in the seismic design of the high rise buildings.
4. CONCLUSIONS

The dynamic response of a soil and accordingly the seismic performance of a superstructure in the soil-structure interaction system are extensively affected by the small-strain shear modulus ($G_0$ or $G_{\text{max}}$), which is oftentimes estimated via its correlation with the in-situ measured shear wave velocity of a soil medium. It would be expected that the variation in the degree of saturation, due to the climate change and seasonal variations, loading-unloading situations, and so on, midst the buildings’ service life, effectuates the field shear wave velocity measurement in the vadose zone. To investigate how the variations in $S_r$-$V_s$-$G_0$ could exert the influence on the seismic response of the high rise building frames incorporating SSI, the 3D model of a 20-story reinforced concrete moment-resisting superstructure on a 30-m thick soil deposit was numerically simulated using the fully nonlinear time history dynamic analysis by adopting the direct method in FLAC software. At this scope, the four values of $S_r$, including 5%, 17.5%, 60%, and 100%, were assigned to the top 4 m of the soil profile, whose corresponding shear wave velocities were 230, 270, and 190, plus 170 m/s, consecutively. It is not unfair to underline that every design has pretensions of being safe and well-founded. In this context, the results associated with the current research project such as to leave no doubt on the necessitousness of utilizing the required soil properties, encompassing the density and shear wave velocity in accordance with the degree of saturation in the vadose zone in advance of running a dynamic analysis of a coupled soil-structure system.

In the performance-based design, the damage should be controlled over the minor to moderate earthquakes while the collapse ought to be prevented in an intense seismic event. In the current study, the “Life Safety (LS)” performance requirement was harnessed as a design criterion, put forward for the rehabilitation of the existing frames. Interestingly enough, the outcomes of this study refuted disregarding the changes in the soil characteristics in the vadose zone due to the seasonal changes as time goes by. In this regard, the $S_r$=5% model illustrated the situation, being devoid of the safety, as per the LS benchmark; because mounting the soil stiffness is the culprit of minimizing the length of the pre-failure path, bespeaking the more deformation, whilst assuming the constant shear strength for a soil. Indeed, the results of this research indicated that the seasonal and climate changes, related to the variation of the soil moisture content in the vadose zone, could influence the extent to which an earthquake can impact a building.

Simply put, the results of this numerical study divulged the import of putting the correlation of degree of saturation-density-shear wave velocity into active for conducting the seismic analyses of the soil-foundation-high rise building interaction models, as exceeding the 2% Life Safety limit, hinging around the post-earthquake damage state, is a building cancer. Needless to say, an ounce of prevention is worth a pound of cure.

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