Seismic Response of Structure under Nonlinear Soil-Structure Interaction Effect

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Abstract

Seismic response of structure under soil-structure interaction effect (SSI) is an impressive subject in earthquake engineering domain. Many analytical models and methods have been proposed and utilized. These methods can be categorized as direct and substructure (indirect) approach. Due to the simplicity requirement, substructure approach is frequently utilized in practical work and research field. In this approach, the analysis procedure is distinguished into three steps: foundation input motion (FIM), dynamic impedance (Spring-Dashpot), and seismic response of structure. However, the state of problem in this approach was found and needed to improve. In the existing analytical model under substructure approach, SSI problem is performed with equivalent-linear of soil material and motion in frequency domain (FD). This restriction can lead to mismatched response results between SSI analysis and actual response of structure during earthquake disaster.

Therefore, the objective of this paper is to propose an analytical model considering nonlinear response of soil material and motion in time domain (TD), which leads to perform the seismic response of structure under nonlinear SSI effect using substructure approach.

In this paper, the proposed analytical model procedure considering nonlinear response of soil material and motion were presented. An example was provided to validate this proposed analytical model. Moreover, the seismic response of structure under existing analytical model and proposed analytical model considering nonlinear response of soil material and motion were conducted. The seismic response of structure was performed under linear response of base-shear, overturning-moment, acceleration, and relative-displacement. Furthermore, the foundation stiffness-damping and hysteretic curve were also provided.

According to the nonlinear response motion in TD from the proposed analytical model, this motion showed a good agreement compared to the linear and equivalent-linear response of ground motion in FD. This agreement confirmed about the validation of this proposed analytical model. Furthermore, the seismic response of structure, it was showed that the response results under existing analytical model were larger than the responses under the proposed analytical model. These discrepancies showed about the overestimated results of using existing model compared to the actual response of structure under earthquake disaster.

Keywords: Soil-structure interaction, substructure approach, nonlinear response of soil material, nonlinear SSI effect.

Introduction

Soil-Structure Interaction (SSI) problem is regarded as a crucial major in earthquake engineering domain. SSI analysis permits evaluating the seismic response of structure and foundation system including the interaction effect of soil medium. This analysis leads to an understanding the actual response of structure under earthquake disaster and controlling the damage response of structural elements.

In order to perform SSI analysis, there are three significant interaction effects that have to consider: kinematic interaction effect, inertial interaction effect, and soil-foundation flexibility effect [1]. To evaluate these interaction effects, various methods have been proposed and utilized such as Finite Element Method (FEM), Boundary Element Method (BEM), the coupling of FEM-BEM, Discrete Element Method (DEM), etc. However, these methods can be categorized as direct and substructure (indirect) approach [2]. In direct approach, the structure and soil are simulated within the same model and analyzed as a complete solution. This approach can deal with complicated structural geometry and soil condition. Many studies have been conducted base on this approach. However, this approach is rarely used in practical work, especially for complex geometrical structure and nonlinearity behavior of soil medium as a result of large computer-storage, running time, and cost consumption [3]. In substructure approach, SSI problem is commonly distinguished into three evaluation steps, which are combined to a complete solution of the seismic response of the whole structure base on the law of superposition [1]. These evaluation steps include foundation input motion (FIM), dynamic impedance (Spring-Dashpot), and seismic response of structure. This approach is widely used in research and practical works due to the simplicity, time, and cost consumption. However, this approach is commonly performed with equivalent-linear response of soil material and motion in FD. This restriction can cause mismatched structural responses between analysis and actual response of structure under earthquake disaster.

In order to deal with this restriction, the objective of this paper is to propose an analytical model considering nonlinear response of soil material and motion, which facilitates performing seismic response of structure under nonlinear SSI effect using substructure approach. In order to obtain this objective, the 3D RC frame structural model was used in this study. This structure was assumed as a rigid surface foundation and supported by uniform soil medium. The Kobe earthquake record data was used as input motion in this study. Other relevant parameters were presented in the following sections.

Existing Analytical Model of SSI Effect under Substructure Approach

Foundation Input Motion (FIM)

FIM can be derived from the relationship of free field ground motion (FFGM) and transfer function. FIM component is composed by translation and rotation motion that can be expressed in Eq. (1) and (2) [4] [5], respectively.

$$u_{FIM} = f\left(H_u, u_g\right) \tag{1}$$

$$\phi_{FIM} = f\left(u_g, I_\phi, B\right) \tag{2}$$

Where

u_{FIM}, ϕ_{FIM} : translation and rotation of FIM

u_g : free field ground motion

H_u, I_ϕ : translation and rotation of transfer function

B : foundation dimension

In this step, FFGM is performed in FD using equivalent-linear method, which is extensively described in the geotechnical earthquake engineering [6]. This method has been used and described in many programs such as SHAKE [7], EERA [8], etc. According to this method, the equivalent-linear response of soil material (G_{EL}, ξ_{EL}) and the corresponding FFGM at the ground surface (u_g) were achieved.

Regarding for the transfer function (H_u, I_ϕ), various expressions under relationship of foundation and wave motion types were described in NIST guideline for SSI problem [5], Mylonakis et al [4], Nikolaou et al [9], etc.

Based on the description above, the FIM can be achieved corresponding to soil conditions, wave motions, and foundation types.

Dynamic Impedance (Spring-dashpot)

Dynamic impedance function is an interaction function between foundation and soil medium. This function is represented by spring and dashpot of soil-foundation interaction system as shown in Fig. 1. The equation of this function is composed by the stiffness (real part) and damping (imaginary part) as expressed in Eq. (3) and (4) [3]-[5].

$$\bar{K}_i = k_i + i\omega c_i \tag{3}$$

$$\bar{K}_i = k_i\left(1 + i2\beta_i\right) \tag{4}$$

$$\beta_i = \frac{\omega c_i}{2k_i} \tag{5}$$

Where

β_i : radiation damping ratio of foundation

$\bar{K}_i$: complex-valued impedance function

k_i, c_i : frequency-dependent foundation stiffness and dashpot

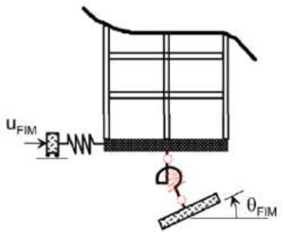


Figure 1. Soil-foundation system [5]

In the Eq. (4), the foundation stiffness k_i can be expressed in function of constant equivalent-linear soil material values (G, υ) from the FFGM analysis in FD and foundation dimensions while the foundation damping can be expressed in function foundation stiffness and radiation damping ratio as shown in Eq. (5).

Various expressions have been proposed for both functions (k_i, c_i) related to different types of foundation and soil conditions. For instance, Mylonakis et al. [4], Gazatas [11] [12], Pais et al. [13] have proposed the solutions for surface and embedded foundation with different types of soil condition while the solution for single pile and group of pile have been discussed in NIST [5].

Seismic Response of Structure

For the seismic response of structure, the structure was assumed to support by spring and dashpot that computed in the second step and subjected to FIM in the first step, as shown in Fig. 2. The seismic response of the structure under SSI effect can be solved under both FD and TD [25] as expressed in Eq. (6) and (7), respectively.

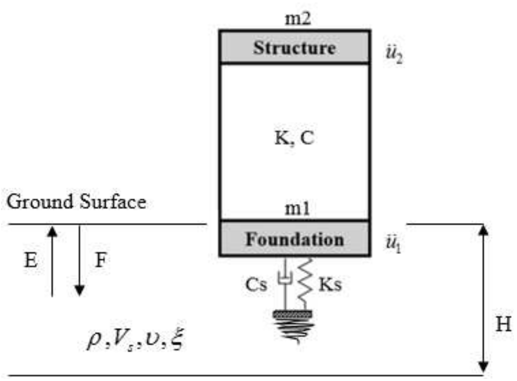


Figure 2. Structure model under SSI effect

-FD Equation:

$$\left(-\omega^2[M]+i\omega[C]+[K]\right)\{U\}=\omega^2[M]\{1\}U_0 \tag{6}$$

-TD Equation:

$$[M]\{\ddot{u}\}+[C]\{\dot{u}\}+[K]\{u\}=-[M]\{1\}\ddot{u}_0 \tag{7}$$

Where

$[M],[C],[K]$: mass, damping, stiffness of the whole structure

U,U_0 : structure displacement and foundation input motion

$\ddot{u},\dot{u},u$: acceleration, velocity and displacement of structure

$\ddot{u}_0$: foundation input motion

According to the description in the existing analytical model above, the seismic response of structure considering SSI effect is solved under equivalent-linear of soil material and FFGM in FD. However, due to this condition, this analytical model might not represent the actual response of structure under earthquake disaster. Therefore, an analytical model of SSI effect considering the nonlinear response of soil material and FFGM in TD was proposed as in the following sections.

Proposed Analytical Model of Nonlinear SSI Effect using Substructure Approach

Free Field Ground Motion Analysis in TD

As described above, the existing analytical model of SSI problem can be performed only with equivalent-linear response of soil material, which can lead to mismatched response of analysis results compared to the actual response of structure under earthquake disaster. Thus, the objective of this paper is to propose an analytical model considering the nonlinear response of soil material and motion that facilitated performing the seismic response of structure under nonlinear SSI effect.

In order to perform FFGM analysis in TD, the cooperation of FFGM analysis in FD was necessary. FFGM analysis in TD was performed by using Newmark's equation [14]-[18], as expressed in Eq. (8). Furthermore, the modified Ramberg-Osgood model [19] was used for hysteretic rule of nonlinear response of soil material. In each layer, soil model can be represented by consistent mass, dashpot, and nonlinear spring, as shown in Fig. 3.

$$[M]\{\ddot{u}\}+[C]\{\dot{u}\}+[K]\{u\}=-[M]\{I\}\ddot{u}_g \tag{8}$$

Where

$[M],[C],[K]$: mass, damping, stiffness matrix

$\{\ddot{u}\},\{\dot{u}\},\{u\}$: acceleration, velocity, displacement vector

$\{\ddot{u}_g\}$: acceleration at the base of column

$\{I\}$: unit vector

The soil element matrix in each layer can be expressed from the Eq. (9) to (11):

$$[M]=\frac{\rho h}{6}\begin{bmatrix}2 & 1\\ 1 & 2\end{bmatrix} \tag{9}$$

$$[K]=\frac{G}{h}\begin{bmatrix}1 & -1\\ -1 & 1\end{bmatrix} \tag{10}$$

$$[C]=\alpha_R[M]+\beta_R[K] \quad (11)$$

Where

ρ : unit weight of soil, h : thickness in each layer

G :shear stiffness of soil, α_R,β_R : coefficient of Rayleigh damping

According to Rayleigh [20], α_R and β_R coefficient can be computed using two significant modes m and n:

$$\frac{1}{2}\begin{bmatrix} 1/\omega_n & \omega_m \\ 1/\omega_m & \omega_n \end{bmatrix}\begin{Bmatrix}\alpha_R \\ \beta_R\end{Bmatrix}=\begin{bmatrix}\xi_m \\ \xi_n\end{bmatrix} \quad (12)$$

This matrix can be solved as the following expressions:

$$\alpha_R=2\omega_m\omega_n\left(\frac{\omega_m\xi_n-\omega_n\xi_m}{\omega_m^2-\omega_n^2}\right) \qquad \beta_R=2\left(\frac{\omega_m\xi_m-\omega_n\xi_n}{\omega_m^2-\omega_n^2}\right)$$

If the damping ratio is frequency independent, α_R and β_R coefficient becomes:

$$\alpha_R=2\xi\left(\frac{\omega_m\omega_n}{\omega_m+\omega_n}\right) \qquad \beta_R=2\xi\left(\frac{1}{\omega_m+\omega_n}\right)$$

Where

ξ : damping ratio ω_m,ω_n : two significant frequency modes

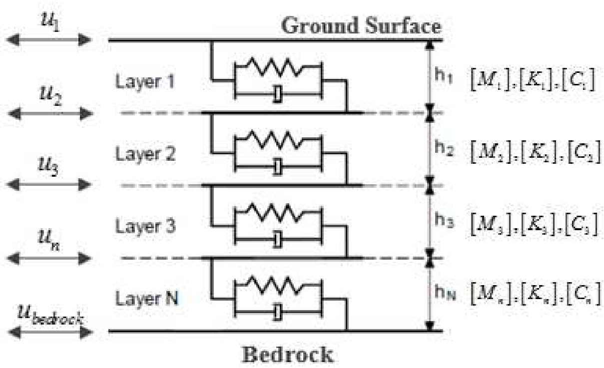


Figure 3. Soil deposit model for TD analysis

As mentioned above, the nonlinear response of soil material was conducted using the modified Ramberg-Osgood model, as shown in Fig. 4, which was proposed by Tatsuoka et al. [19]. The skeleton and hysteretic curve equations were expressed in Eq. (13) and (14), respectively.

$$\gamma=\frac{\tau}{G_0}\left(1+\alpha|\tau|^{\beta}\right) \quad (13)$$

$$\frac{\gamma\pm\gamma_a}{2}=\frac{\tau\pm\tau_a}{2G_0}\left(1+\alpha\left|\frac{\tau\pm\tau_a}{2}\right|^{\beta}\right) \quad (14)$$

Where

$$\beta = \frac{2\pi h_{max}}{2 - h_{max}}, \qquad \alpha = \left(\frac{2}{\gamma_{0.5} G_0}\right)^{\beta}$$

$\gamma_{0.5}$: corresponds to $G/G_0 = 0.5$ $\quad \alpha, \beta$: parameter of modified R-O

γ_a, τ_a : reversal shear strain and stress $\quad G_0$: initial shear soil stiffness

h_{max} : maximum soil damping

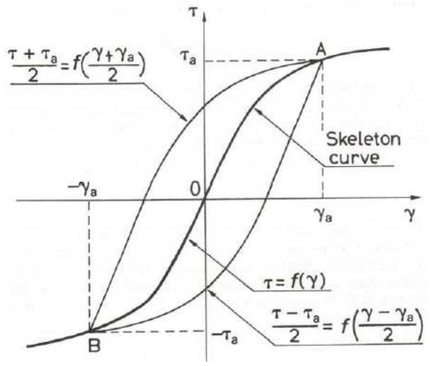


Figure 4. Stress-strain relationship of Ramberg-Osgood model [21]

According to hysteretic rule, the nonlinear response of shear stiffness $G_i(t)$ can be derived from Eq. (15) and shown in Fig. 5.

$$G_i(t) = \frac{\tau_i - \tau_{i-1}}{\gamma_i - \gamma_{i-1}} \tag{15}$$

Where

τ_i, τ_{i-1} : reversal shear stress of point i and i-1

γ_i, γ_{i-1} : reversal shear strain of point i and i-1

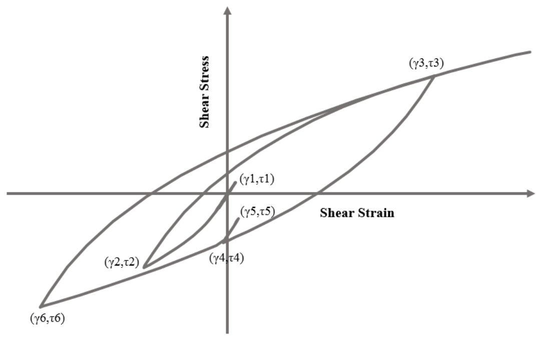


Figure 5. Reversal points of shear stress-strain

Besides this, in order to perform nonlinear response analysis of FFGM in TD, the analytical procedure of input motion at the base of soil column was very important in order to obtain a properly output motion at the ground surface.

Analytical Procedure for Input Motion in TD

In this step, the FFGM in FD was needed to obtain the input motion for FFGM analysis in TD.

In linear analysis (LN), the target earthquake motion was input at the base of soil column (or surface layer) as an outcrop motion (*2E*). Then, the within output motion (*E+F*) was extracted at the base of soil column and applied this motion at the same layer of soil column (as input motion) for TD analysis, as shown in Fig. 6. The within motion (*E+F*) of any location is an actual motion of that location.

In nonlinear analysis (NL), the procedure was the same as linear analysis but it was required to perform in both linear and equivalent-linear (EL) analysis in FD and the within output motion (*E+F*) of both analyses were significant to be the same or almost the same. Due to this requirement, some extra layers might be needed. Then, this within output motion (*E+F*) was applied at the same layer of soil column for TD analysis, as shown in Fig. 6.

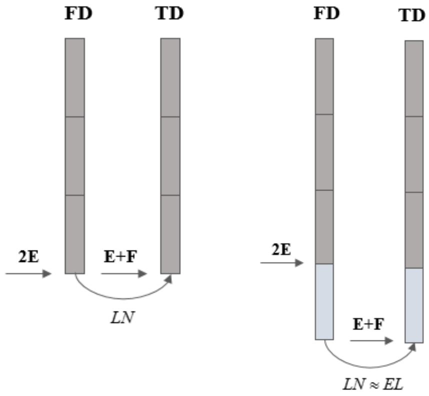


Figure 6. Linear and nonlinear input motion procedure for TD analysis

Furthermore, in order to validate the nonlinear response output motion at the ground surface in TD, a comparison of this motion with linear and equivalent-linear analysis at the ground surface were necessary. This comparison can lead to an understanding how correctly of this nonlinear response motion.

Example of FFGM Analysis in TD

In this study, the uniform soil column in depth 20m was assumed rested on the bedrock. This uniform soil column consisted the same properties as in class E of IBC [22], as shown in table 1. Kobe earthquake record data were assumed as input motion at the base of soil column. The motion in X and Y direction were assumed as the same as the motion in EW and NS of record data as shown in Fig. 7 while the motion in UD direction was ignored in this study.

Table 1. Uniform soil column properties

H (m)	Vs (m/s)	γ (kN/m^3)	ξ (%)
0.00-20.0	180	18.0	5
Bedrock	900	21.0	1

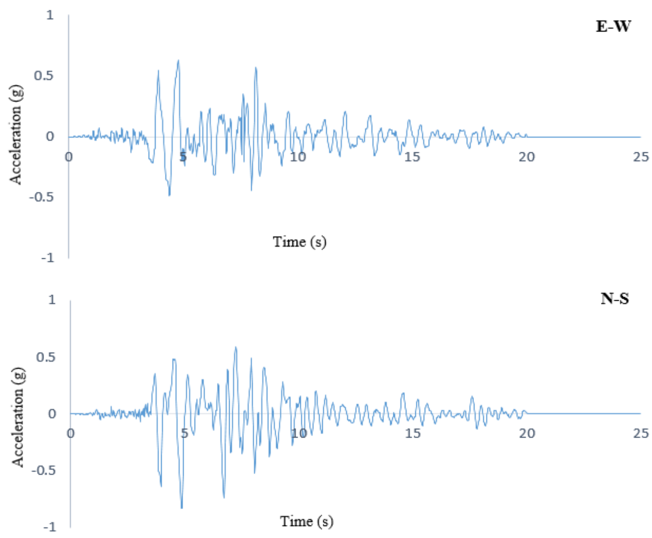


Figure 7. Kobe earthquake record motion data

For linear analysis, according to the procedure described above, the output result at the ground surface for both analyses was shown in Fig. 8.

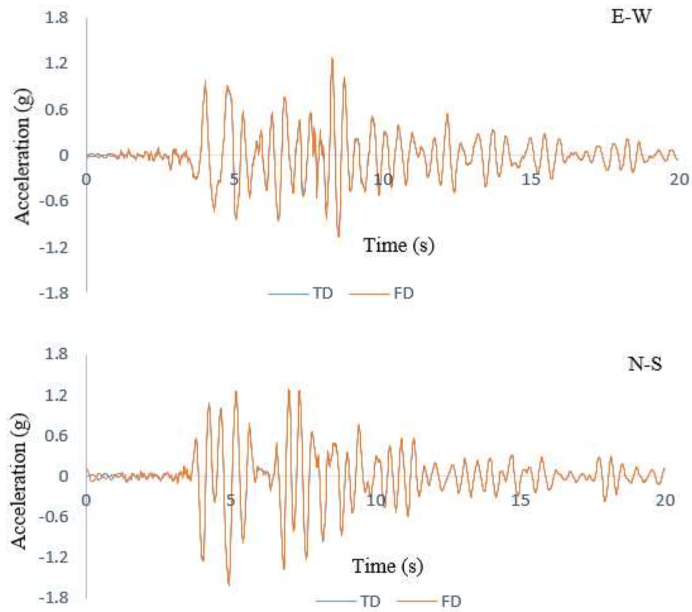


Figure 8. Linear analysis of FFGM in FD and TD

For nonlinear analysis, according to the procedure described above, two extra layers were needed for this study case, as shown in Fig. 9. The first layer consisted 5m in depth and 900 m/s for shear velocity while the second layer consisted 800m in depth and 8km/s for shear velocity above the bedrock, which consisted 8 km/s for shear velocity. The within output motion in FD was shown in Fig. 10 and the output motion at the ground surface in TD was shown in Fig. 11.

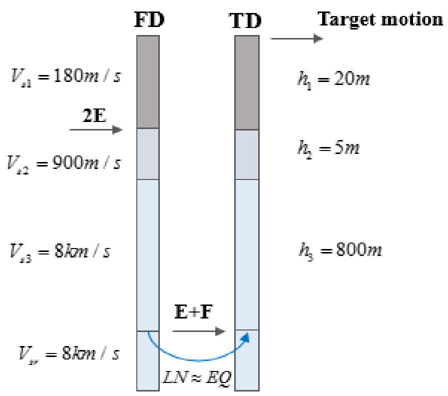
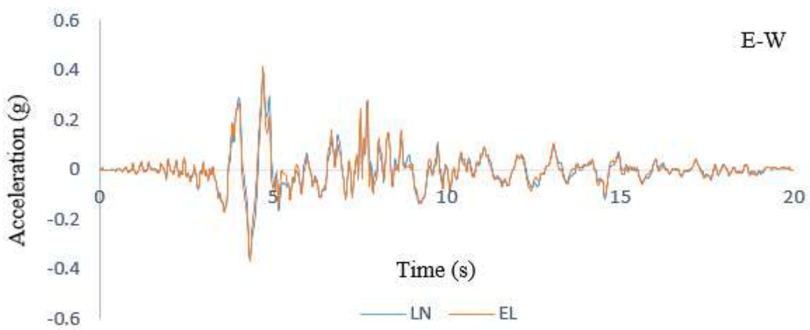


Figure 9. Analytical procedure for input motion in TD



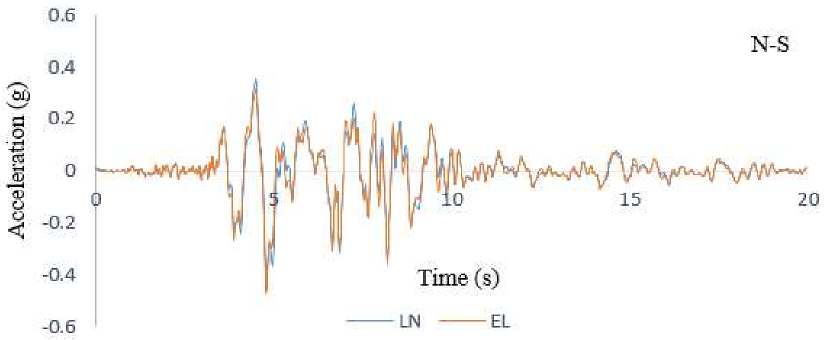


Figure 10. Within input motion (E+F) in TD

As shown in Fig. 10, the within output results (E+F) from both analyses showed a good agreement and adequate for input motion for TD analysis. These within outputs (E+F) were applied at the same layer and property for TD analysis. The FFGM at the ground surface for both directions and nonlinear response of soil stiffness (G_{NL}) were obtained.

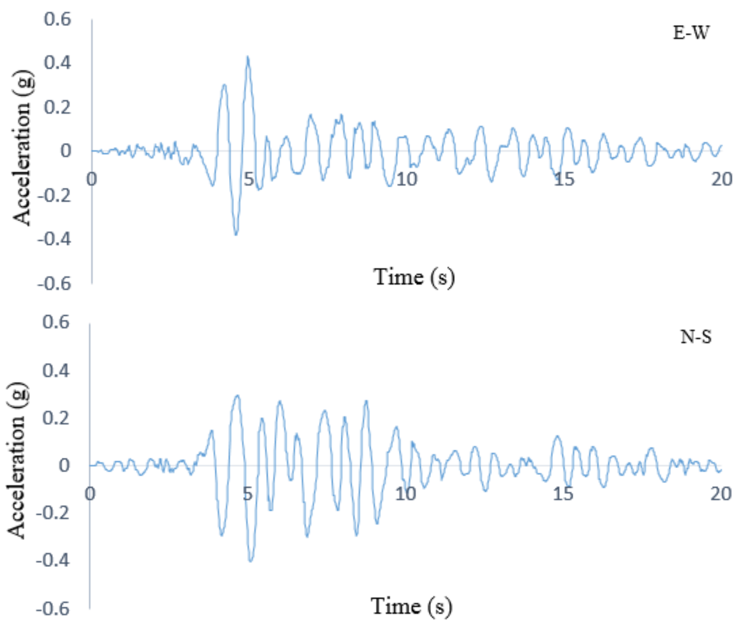
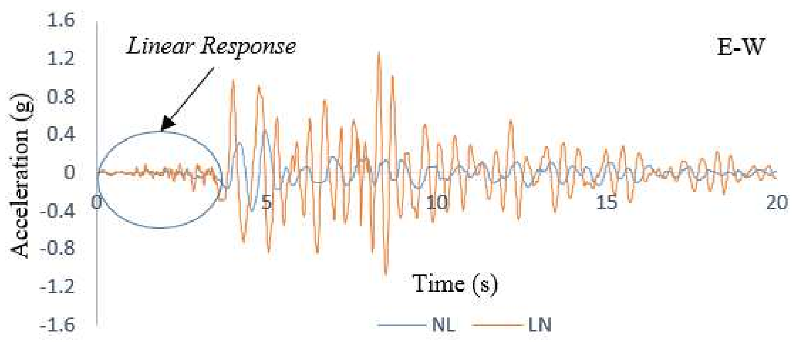


Figure 11. FFGM at the ground surface in TD

However, as described above, the comparison of these nonlinear response motions with linear and equivalent-linear motions at the ground surface in FD was necessary. These comparisons were shown in Fig. 12 and 13.



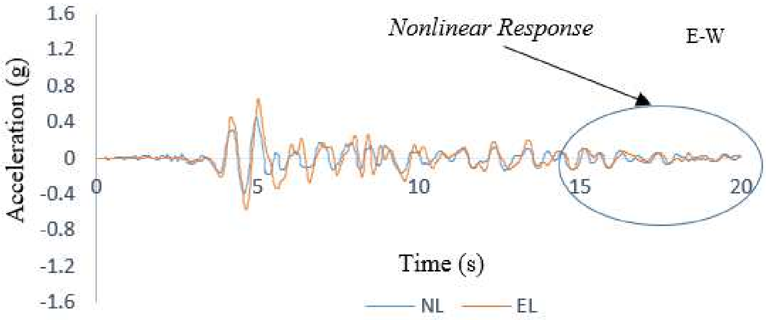


Figure 12. Comparison between LN, EL, and NL motion in E-W direction

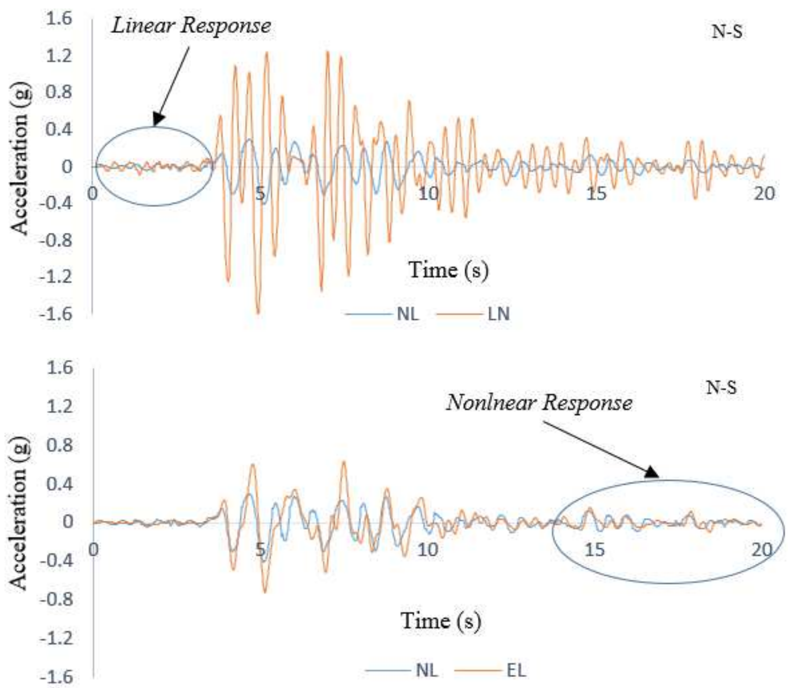
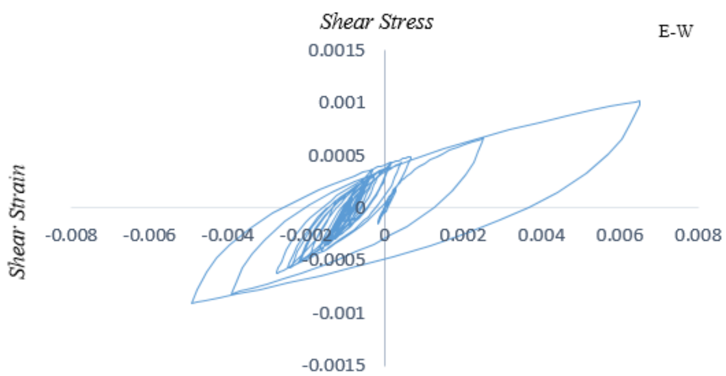


Figure 13. Comparison between LN, EL, and NL motion in N-S direction

As shown in Fig. 12 and 13, the nonlinear response result showed a good agreement with linear motion response for a few seconds from starting point and with equivalent-linear motion response for the last several seconds. These agreements confirmed that the nonlinear motion response at the ground surface in TD started from linear to nonlinear motion response. This confirmation showed about the validation of the proposed analytical model considering the nonlinear response of soil material and motion. The hysteretic curve of nonlinear response of soil medium at the ground surface was also provided, as shown in Fig. 14.



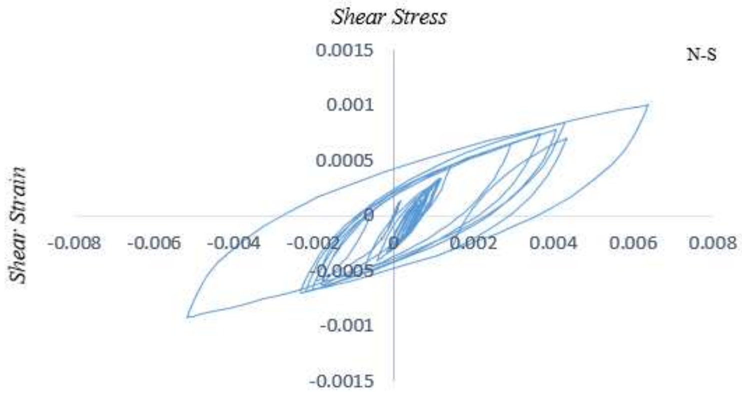


Figure 14. Hysteretic curve of nonlinear response of soil medium

The response of soil stiffness under both analytical models, FD and TD, were shown in Fig.15.

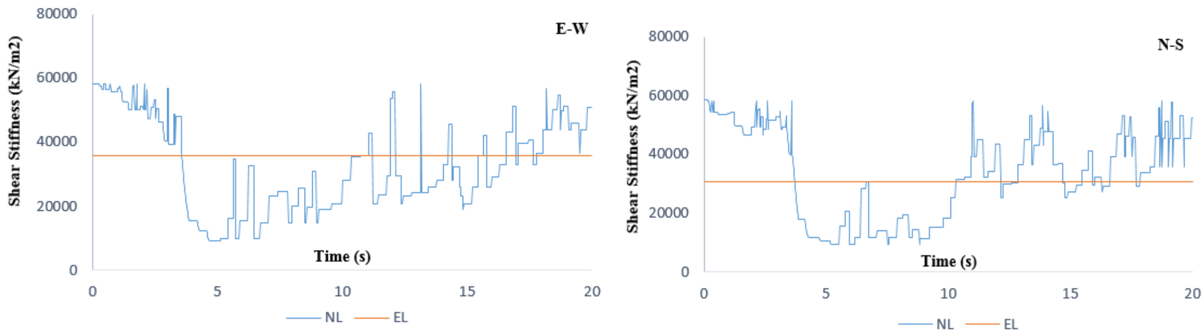


Figure 15. Soil stiffness response analysis in TD and FD

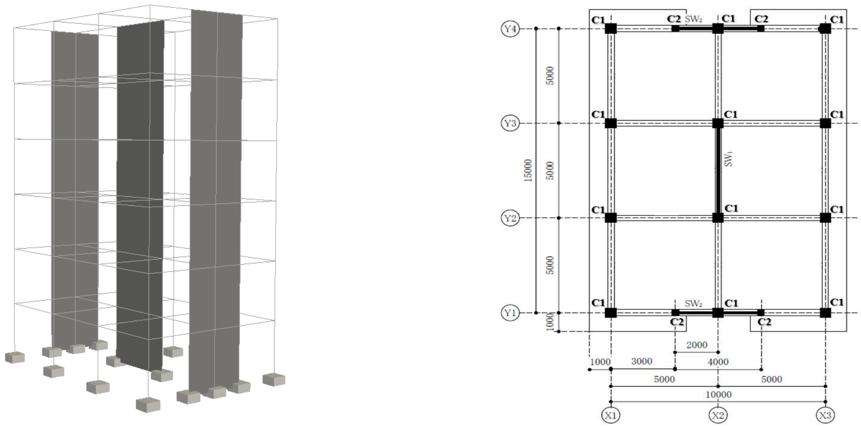
Seismic Response of RC Frame Structure under Nonlinear SSI Effect

After obtaining the nonlinear response of soil materials and FFGM at the ground surface, the linear response of RC frame structure under equivalent-linear and nonlinear SSI effect were conducted.

In order to achieve this objective, the 3D RC frame structural model (from E-defense test [23] [24]) was used in this study. This structure was supported by rigid surface foundation and was assumed to rest on uniform soil deposit subjected to vertically incident S wave. The relevant parameters were shown in the following sections.

Structural Model Assumption

As mentioned above, the model of 3D frame structure was used in this study as shown in Fig. 16. This structural model consisted six stories and 3.5m for height in each story. There were two spans in X-direction and three spans in Y-direction with the same length 5m in each span. In this study, the column C1 section was 0.5mx0.5m with 8-D19 and C2 section was 0.3mx0.3m with 4-D19, beam section was 0.3mx0.5m with 5-D19, and both shear-wall and sidewalls thickness were 0.15m with doubly reinforcing bar D10@300. Furthermore, the shear reinforcing bar of column was D10@100 while beam element was D10@200. The nominal strength of reinforcing bars were SD345 and SD295 for D19 and D10, respectively, and concrete strength was 21MPa for all structural elements. Besides this, the non-structural element load was assumed 3.0 kPa and live load 2.5kPa for each story.



(a) Perspective view (b) Top view [24]

Figure 16. E-Defense test structure model

Soil Property and Input Motion

The uniform soil property and input motions were assumed to be the same as described in the previous sections. The equivalent-linear and nonlinear of soil stiffness values were shown in Fig. 15.

Foundation Input Motion (FIM)

Due to the condition of rigid surface foundation and subjected to the vertically incident S wave, the FIM was the same as the FFGM [4]. The FIM response of both FD and TD were shown in Fig. 17 and 18, respectively.

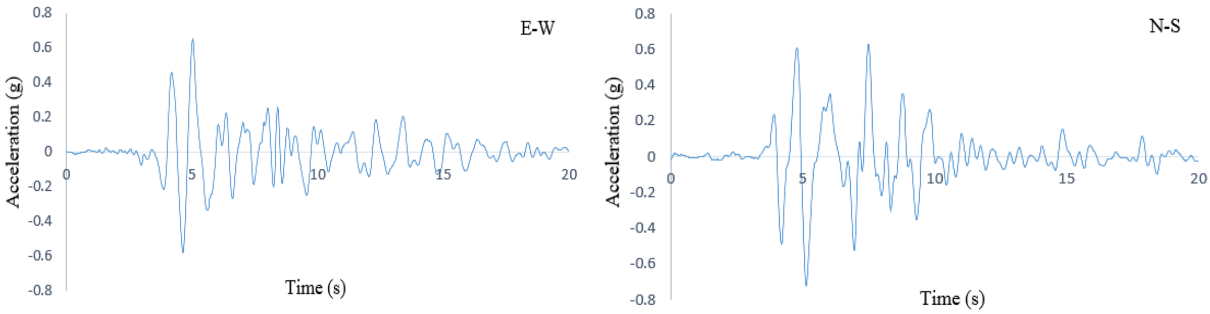


Figure 17. Foundation Input Motion in FD

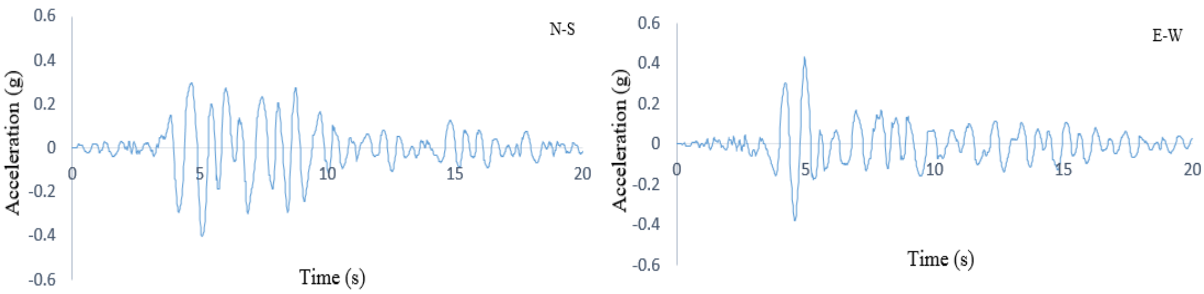


Figure 18. Foundation Input Motion in TD

Dynamic Impedances

The dynamic impedance of rigid surface foundation was expressed in Eq. (3) and (4). The static stiffness, dynamic stiffness modifier and radiation damping can be expressed in the following equations:

1. Static Stiffness

$$K_x = K_y - \frac{0.2}{0.75-\upsilon}GL\left(1-\frac{B}{L}\right) \qquad K_{xx} = \frac{G}{1-\upsilon}\left(I_x\right)^{0.75}\left(\frac{L}{B}\right)^{0.25}\left[2.4+0.5\left(\frac{B}{L}\right)\right] \tag{16}$$

$$K_y = \frac{2GL}{2-\upsilon}\left[2+2.5\left(\frac{B}{L}\right)^{0.85}\right] \qquad K_{yy} = \frac{G}{1-\upsilon}\left(I_y\right)^{0.75}\left[3\left(\frac{L}{B}\right)^{0.15}\right] \tag{17}$$

$$K_z = \frac{2GL}{1-\upsilon}\left[0.73+1.54\left(\frac{B}{L}\right)^{0.75}\right] \qquad K_{zz} = GJ_t^{0.75}\left[4+11\left(1-\frac{B}{L}\right)^{10}\right] \tag{18}$$

2. Dynamic Stiffness Modifier

$$\alpha_x = 1.0 \qquad \alpha_{xx} = 1-\left[\frac{\left(0.55+0.11\sqrt{\frac{L}{B}-1}\right)a_0^2}{\left(2.4-\frac{0.4}{\left(\frac{L}{B}\right)^3}+a_0^2\right)}\right] \tag{19}$$

(19)

$$\alpha_y = 1.0 \qquad \alpha_{yy} = 1-\left[\frac{0.55a_0^2}{\left(0.6+\frac{1.4}{\left(\frac{L}{B}\right)^3}\right)+a_0^2}\right] \tag{20}$$

$$\alpha_z = 1-\left[\frac{\left(0.4+\frac{0.2}{\left(\frac{L}{B}\right)}\right)a_0^2}{\left(\frac{10}{1+3\left(\frac{L}{B}-1\right)}\right)+a_0^2}\right] \qquad \alpha_{zz} = 1-\left[\frac{\left(0.33-0.33\sqrt{\frac{L}{B}-1}\right)a_0^2}{\left(\frac{0.8}{1+0.33\left(\frac{L}{B}-1\right)}\right)+a_0^2}\right] \tag{21}$$

3. Radiation Damping

$$\beta_x = \left[\frac{4\left(\frac{L}{B}\right)}{\frac{K_x}{GB}}\right]\left[\frac{a_0}{2\alpha_x}\right] \qquad \beta_{xx} = \left[\frac{\left(\frac{4\psi}{3}\right)\left(\frac{L}{B}\right)a_0^2}{\left(\frac{K_{xx}}{GB^3}\right)\left[2.2-\frac{0.4}{\left(\frac{L}{B}\right)^3}+a_0^2\right]}\right]\left[\frac{a_0}{2\alpha_{xx}}\right] \tag{22}$$

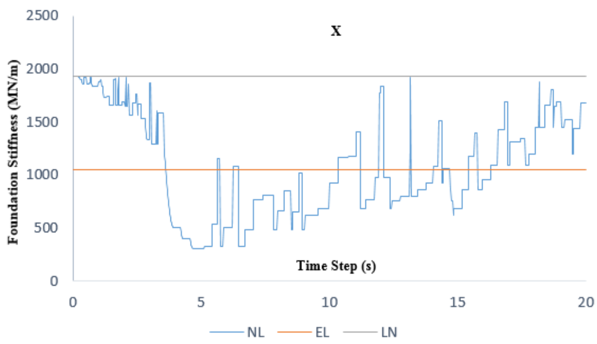
$$\beta_y = \left[\frac{4\left(\frac{L}{B}\right)}{\frac{K_y}{GB}} \right] \left[\frac{a_0}{2\alpha_y} \right] \qquad \beta_{yy} = \left[\frac{\left(\frac{4\psi}{3}\right)\left(\frac{L}{B}\right)^3 a_0^2}{\left(\frac{K_{yy}}{GB^3}\right)\left[\left(\frac{1.8}{1+1.75\left(\frac{L}{B}-1\right)}\right)+a_0^2\right]} \right] \left[\frac{a_0}{2\alpha_{yy}} \right] \tag{23}$$

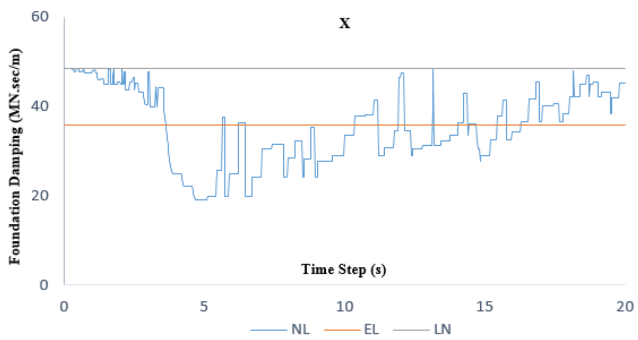
$$\beta_z = \left[\frac{4\psi\left(\frac{L}{B}\right)}{\frac{K_z}{GB}} \right] \left[\frac{a_0}{2\alpha_z} \right] \qquad \beta_{zz} = \left[\frac{\left(\frac{4}{3}\right)\left[\left(\frac{L}{B}\right)^3+\left(\frac{L}{B}\right)\right] a_0^2}{\left(\frac{K_{zz}}{GB^3}\right)\left[\frac{1.4}{1+3\left(\frac{L}{B}-1\right)^{0.7}}+a_0^2\right]} \right] \left[\frac{a_0}{2\alpha_{zz}} \right] \tag{24}$$

Linear, Equivalent-linear and Nonlinear Foundation Stiffness-Damping

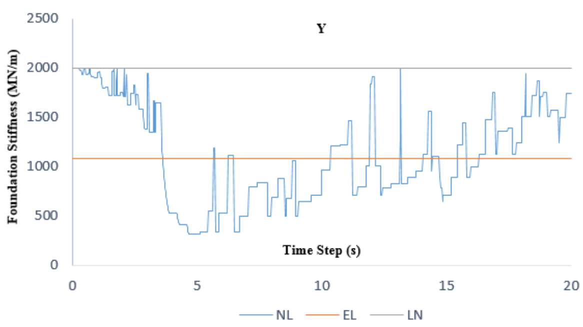
In this comparison, there are six directions for foundation stiffness k_i and damping c_i for rigid surface foundation as shown in Fig. 19.

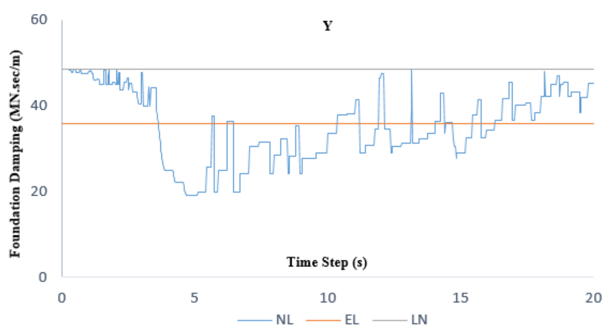
- Direction X:



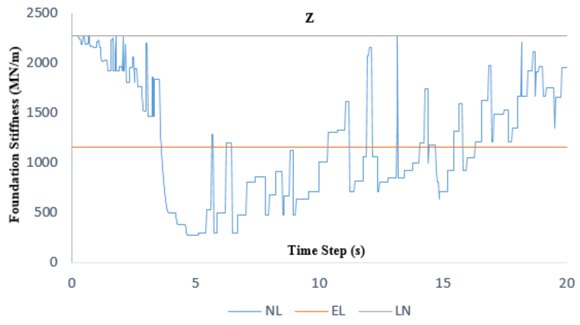


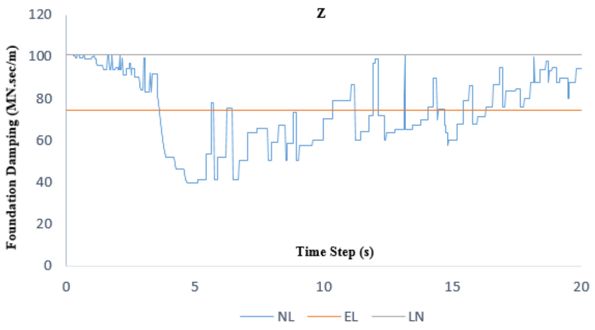
- Direction Y:



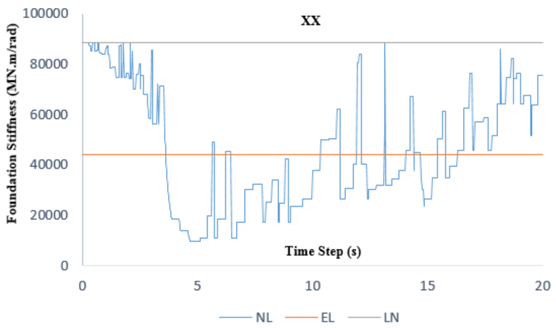


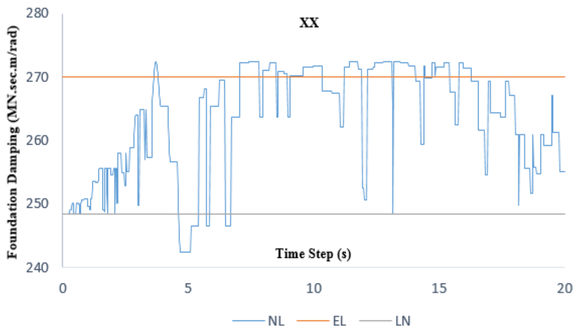
- Direction Z



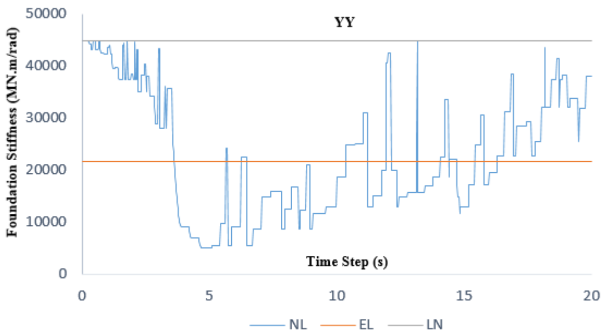


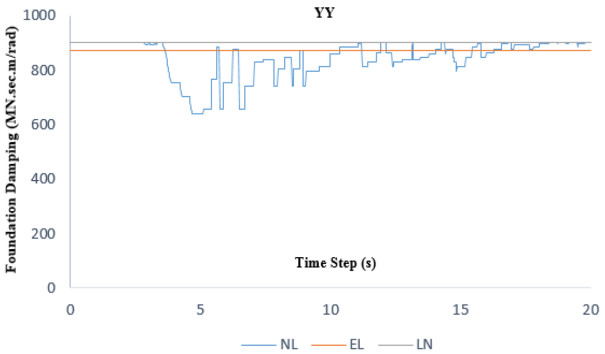
- Direction XX:



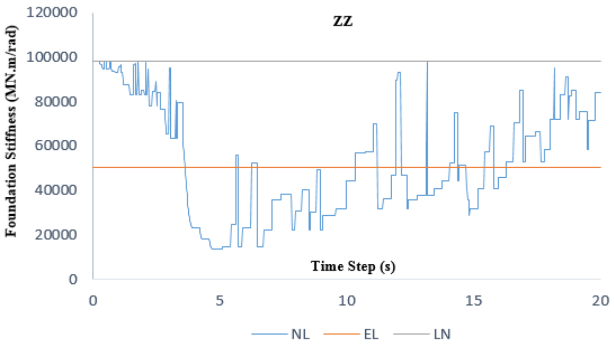


- Direction YY:





- Direction ZZ:



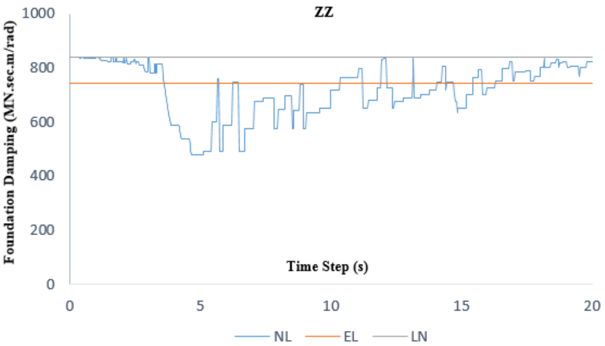
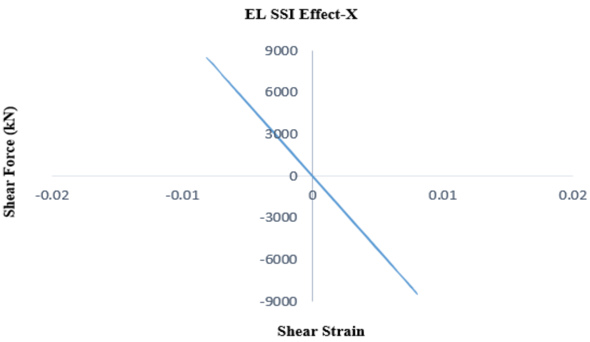
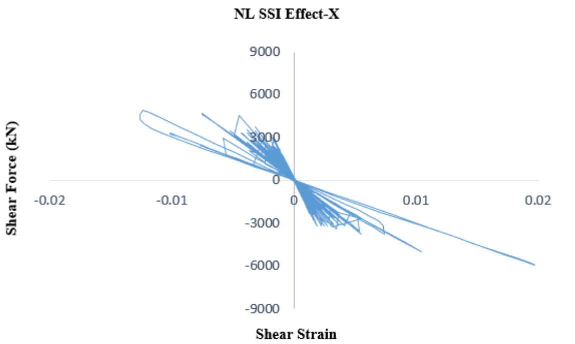
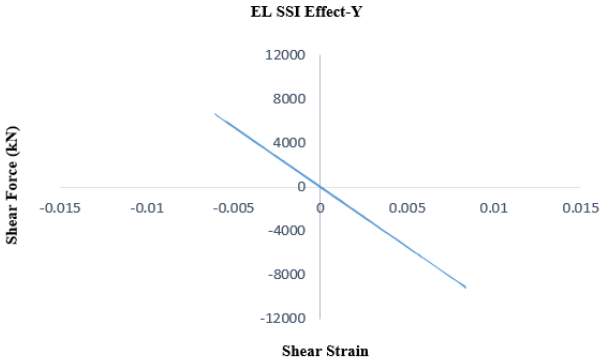


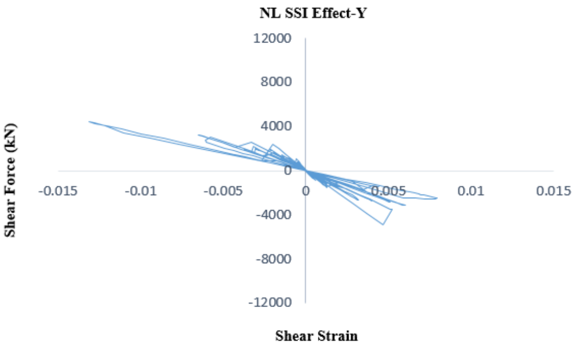
Figure 19. Linear, Equivalent-linear, and Nonlinear Foundation Stiffness

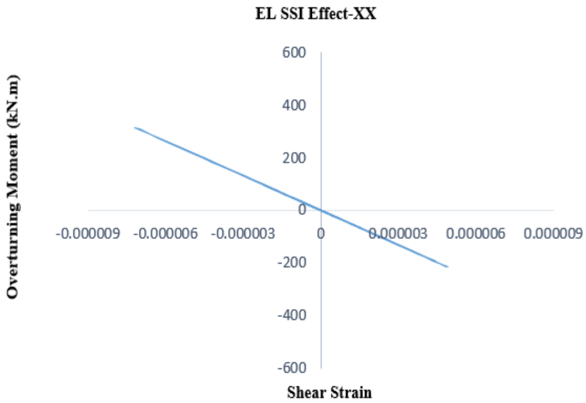
Hysteretic Curve of Foundation-Soil System

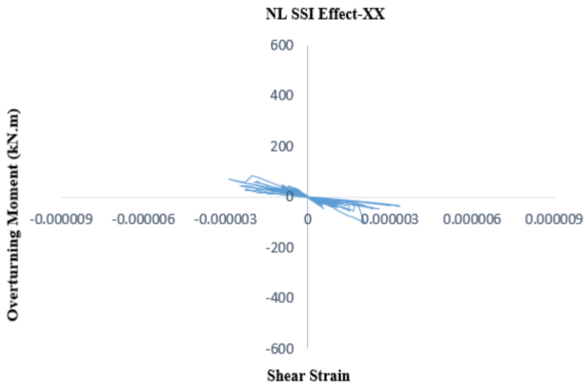


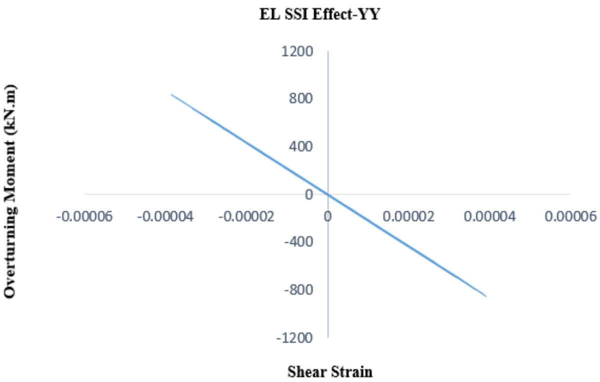


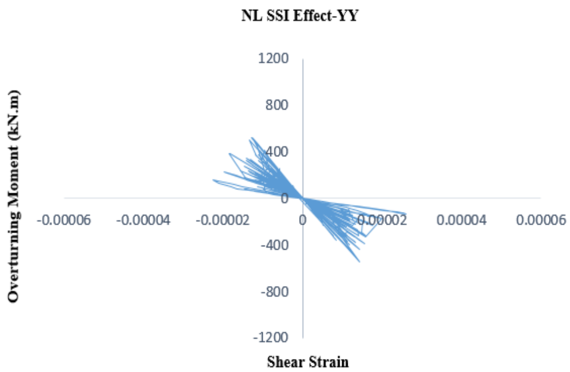












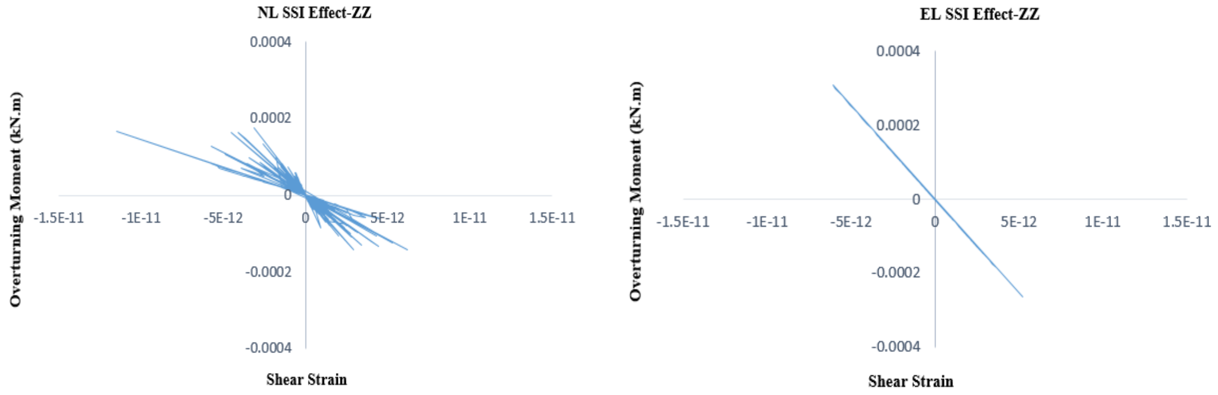


Figure 17 Hysteretic curve of foundation-soil system under both analytical models

Seismic Response of RC frame Structure

In this section, the seismic response of frame structure under both equivalent-linear and nonlinear SSI effect were presented under linear response of base-shear, overturning-moment acceleration, and relative displacement in TD based on Eq. (7), as shown from Fig. 17 to 20, respectively.

- Base-Shear:

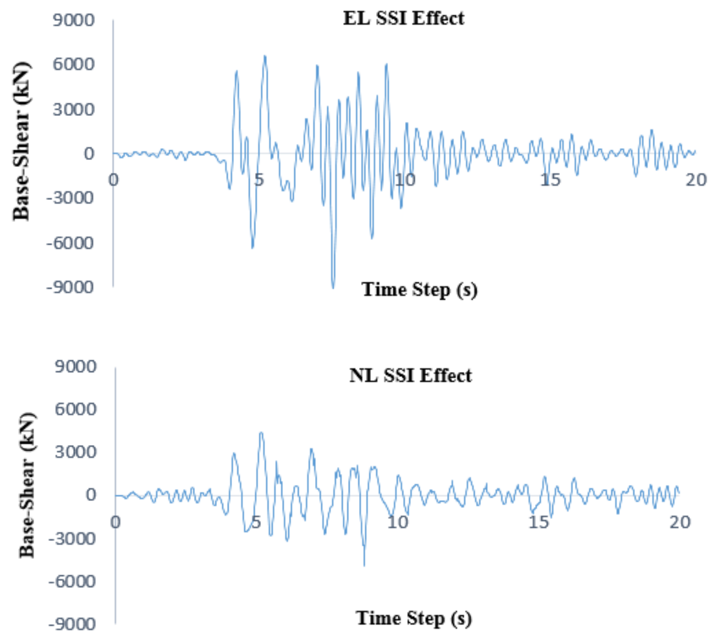
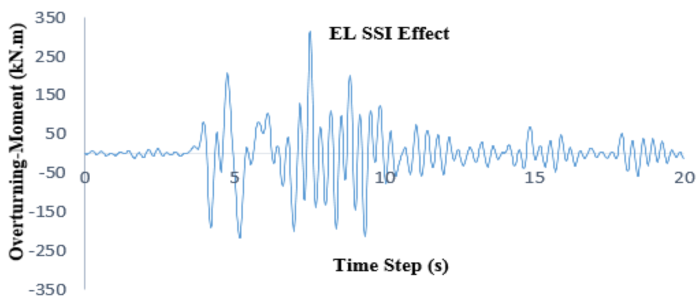


Figure 17. Base-Shear response under equivalent-linear and nonlinear SSI effect

- Overturning-Moment:



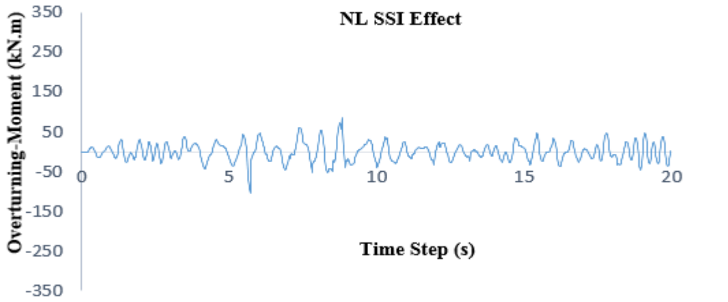


Figure 18. Overturning-Moment under equivalent-linear and nonlinear SSI effect

- Acceleration in each floor:

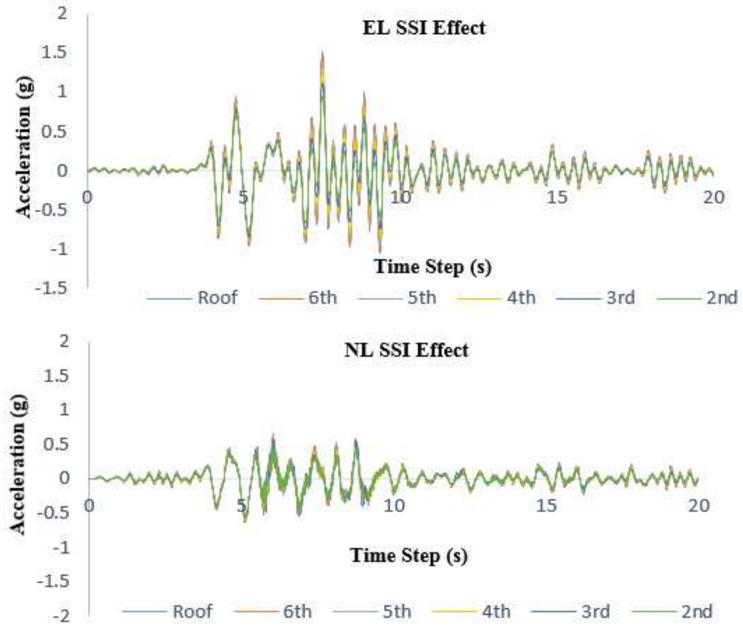


Figure 19. Acceleration under equivalent-linear and nonlinear SSI effect

- Relative Displacement:

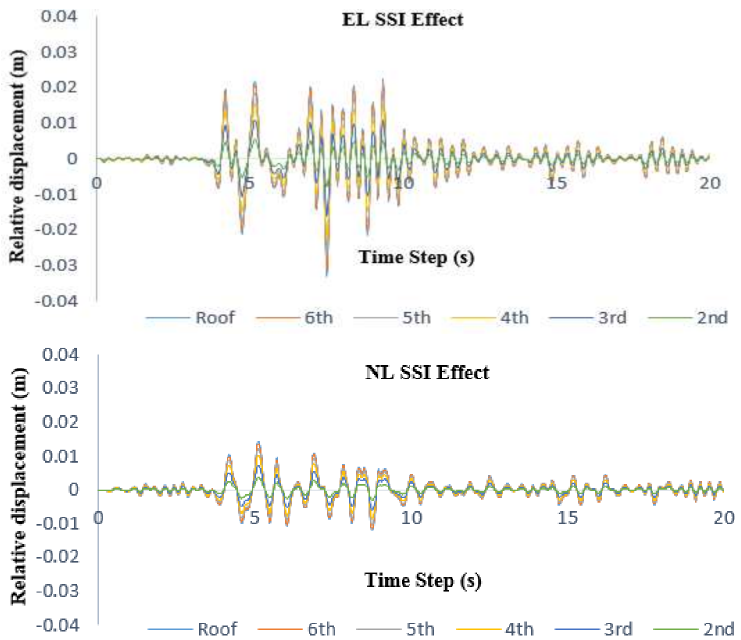


Figure 20. Relative displacement under equivalent-linear and nonlinear SSI effect

Based on the response results above, it was showed that the responses of structure under equivalent-linear SSI effect were larger than the responses under nonlinear SSI effect using substructure approach. These discrepancies showed about overestimated responses of using existing analytical model of SSI problem compared to the actual response of structure under earthquake disaster.

Conclusions

In this paper, the analytical model considering nonlinear response of soil material and motion in TD was proposed. The nonlinear response of motion in TD showed a good agreement with linear response for a few seconds from starting point and with equivalent-linear analysis in FD for the last several seconds. This agreement confirmed about the validation of proposed analytical model.

Furthermore, the seismic response of structure under existing and proposed analytical model were conducted under linear response of base-shear, overturning-moment, acceleration, and relative displacement. The output results showed that the structural response under existing model were larger than the responses under proposed model. These discrepancies showed about the overestimated results of using existing analytical model compared the actual response of structure under earthquake disaster.

In conclusion, the proposed analytical model considering nonlinear SSI effect using substructure approach on the structural response under earthquake loading would be a good candidate for SSI problem and showed about the adequateness of this approach compared to the actual response of structure.

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