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IMPROVED TWO-STEP METHOD FOR SEISMIC ANALYSIS OF STRUCTURES

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ABSTRACT

Seismic soil-structure interaction (SSI) analysis requires use of specialty programs such as SASSI2010, which are not suited for general structural analysis for other (non-seismic) load cases and load combinations. Also, due to the computational effort involved, SSI analyses are generally performed using somewhat coarser finite element mesh, whereas static/dynamic analyses for other load cases are typically performed using more refined finite element mesh. Because of this difficulty, the seismic design forces are obtained using selected outputs from SSI analysis as input to a second analysis that is performed using general purpose structural analysis software. This approach is called two-step method, in which SSI analysis is the first step. The second step of analysis is either equivalent static analysis or response spectrum analysis. In the former inertial loads are applied based on subjective interpretation of SASSI2010 results, which ignores their temporal and spatial distribution and therefore leads to conservative results for sliding/overturning stability evaluations and member design forces. In response spectrum analysis, the SSI-generated Foundation Input Motion (FIM) across the foundation expanse is accounted for determining the envelope of foundation in-structure response spectra (ISRS) at selected locations as the input spectrum. While this is an improvement, this approach does not capture the superstructure response to the actual incoherent foundation motion that contains effects such as SSI-induced rotational motion (i.e., rocking and torsion), basemat flexibility, and cross-directional excitation effects. This paper presents an improved two-step method that characterizes the incoherent FIM using combination of spatial mode shapes (i.e., admissible shape functions that collectively approximate the total foundation motion at each node). Results are compared with SASSI2010 to judge the method's accuracy for various response quantities, and sensitivity of results to the number of spatial modes is studied. The results show that the proposed method is quite accurate when a few basemat mode shapes are included in addition to the six rigid body mode shapes (three translations and three rotations due to SSI).

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Seismic soil-structure interaction (SSI) analysis requires use of specialty programs such as SASSI2010, which are not suited for general structural analysis for other (non-seismic) load cases and load combinations. Also, due to the computational effort involved, SSI analyses are generally performed using somewhat coarser finite element mesh, whereas static/dynamic analyses for other load cases are typically performed using more refined finite element mesh. Because of this difficulty, the seismic design forces are obtained using selected outputs from SSI analysis as input to a second analysis that is performed using general purpose structural analysis software. This approach is called two-step method, in which SSI analysis is the first step. The second step of analysis is either equivalent static analysis or response spectrum analysis. In the former inertial loads are applied based on subjective interpretation of SASSI2010 results, which ignores their temporal and spatial distribution and therefore leads to conservative results for sliding/overturning stability evaluations and member design forces. In response spectrum analysis, the SSI-generated Foundation Input Motion (FIM) across the foundation expanse is accounted for determining the envelope of foundation in-structure response spectra (ISRS) at selected locations as the input spectrum. While this is an improvement, this approach does not capture the superstructure response to the actual incoherent foundation motion that contains effects such as SSI-induced rotational motion (i.e., rocking and torsion), basemat flexibility, and cross-directional excitation effects. This paper presents an improved two-step method that characterizes the incoherent FIM using a combination of spatial mode shapes (i.e., admissible shape functions that collectively approximate the total foundation motion at each node). Results are compared with SASSI2010 to judge the method's accuracy for various response quantities, and sensitivity of results to the number of spatial modes is studied. The results show that the proposed method is quite accurate when a few basemat mode shapes are included in addition to the six rigid body mode shapes (three translations and three rotations due to SSI).

Introduction

Seismic soil-structure interaction (SSI) analysis is required for mission-critical facilities (especially safety-related nuclear facilities) in order to determine the earthquake-induced forces for structural design and develop in-structure response spectra (ISRS) for seismic qualification of

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critical equipment. Seismic SSI analysis requires use of specialty programs such as SASSI2010 [1], which are not suited for general structural analysis for other (non-seismic) load cases and load combinations. Also, due to the computational effort involved, SSI analyses are generally performed using somewhat coarser finite element mesh, whereas static/dynamic analyses for other load cases are typically performed using more refined finite element mesh. As such, seismic forces cannot be directly inferred from the SSI analyses unless the analytical models for static and seismic analysis are identical (which requires significant computational resources). Because of this difficulty, the seismic design forces are obtained using selected outputs from SSI analysis as input to a second analysis that is performed using general purpose structural analysis software [2]. Such two-step approach, in which SSI analysis is the first step, enables combination of the seismic demands with those due to other applicable load cases.

The second step of analysis is generally done as an equivalent static analysis, and is performed by applying inertial loads based on the results of the SASSI2010 analysis. The use of inertial loads ignores the temporal and spatial distribution of the inertia forces during time-history of seismic response, and is based on subjective interpretation of the SSI results. Thus, the resulting static analysis generates very conservative results for purposes of sliding/overturning stability evaluations and member design. Alternatively, the second step can be performed using response spectrum analysis by considering the Foundation Input Motion (FIM) results from SSI analysis. The actual FIM is incoherent across the foundation expanse owing to SSI-induced rotational motion effects (i.e., rocking and torsion), basemat flexibility, and the cross-directional excitation effects. However, for simplicity, the FIM spectra (horizontal and vertical) are selected as the envelope of several ISRSs across the foundation expanse. While this is an improvement, the assumption of enveloped coherent FIM does not truly capture the superstructure response to the foundation motion due to SSI-induced rotations and other effects.

This paper presents an improved two-step method that characterizes the incoherent FIM using a combination of spatial “mode shapes” (i.e., admissible shape functions that collectively approximate the total foundation motion at each node). This improvement eliminates the conservatism associated with a simplistic equivalent static analysis while avoiding the pitfalls of a response spectrum analysis that fails to properly account for the FIM incoherence. The featured methodology defines the foundation motion of the structure in terms of a limited number of spatial foundation mode shapes, whose time-histories are obtained from the SASSI2010-generated total displacement time-histories at select foundation nodes and are applied to the superstructure as concurrent coherent motions in the second analysis step. At minimum, the spatial mode shapes should include the SSI induced rigid body modes (three translation modes, two rocking modes, and one torsional mode). Additionally, out-of-plane mode shapes of the basemat and embedded walls (which depend on their rigidity) may be added if deemed important. In the second step of the seismic analysis, fixed-base coherent seismic analysis is performed for each of the constituent foundation mode shapes using the time-history analysis method using a general purpose structural analysis software such as SAP2000 [3], and the total seismic response of the structure is obtained as the algebraic sum of responses.

The accuracy of the proposed method and the number of foundation mode shapes necessary are investigated for a sample labyrinthine shear wall structure (representative of a nuclear facility) supported on soil subgrades. It is noted that conventional SSI analysis (e.g., using SASSI2010) is performed using equivalent linear stiffness and viscous damping properties. As such, the current two-step techniques are not suitable for addressing any nonlinear behavior at the soil-structure interface and at the superstructure level. On the other hand, the proposed two-step approach could be used to perform a nonlinear analysis in the second step provided that the nature and degree of nonlinear behavior has only a small effect on the FIM. Examples where such nonlinear analysis may be desirable are: displacement dependent hysteretic behavior of major shear walls (including response to beyond-design-basis seismic motion, minor episodes of sliding (stick-slip) at the soil-foundation interface, and small amounts of foundation uplift (foundation rocking involving small amplitudes and small loss of contact area). Further examples and discussion of such effects and their application are provided in [4] and [5]. This paper only considers linear response behavior in order to test the proposed method's effectiveness. Nonlinear response behavior will be studied in future if good accuracy is demonstrated for linear behavior.

Description of Proposed Two-Step Methodology

The goal of the methodology presented here is to adequately capture the interface motion of the structure to facilitate a two-step structural analysis. As discussed before, the complete interface motion (herein referred to as the foundation motion) is calculated in the 1st step of the analysis (typically a SSI analysis). For the 2nd step, the complete foundation motion is decomposed into a set of spatial modes and their corresponding time-histories. The second step of the analysis is then carried out by concurrent application of the spatial modes and their corresponding time-histories.

Let $R(t,x,y,z)$ represent the complete foundation displacement response, where, t refers to time and $x, y,$ and z refer to the coordinates of each node on the foundation. Then, R is decomposed into N modes as shown in Eq. 1, where $\varphi_i(x,y,z)$ is the spatial mode i and $d_i(t)$ is its corresponding time-history.

$$R(t,x,y,z) = \sum_{i=1}^{N} d_i(t) \cdot \varphi_i(x,y,z) \quad (1)$$

In general, the foundation modes may be arbitrarily selected. However, to achieve reasonable accuracy with the least number of modes as possible, the first 6 modes are selected as three rigid body translations and three rigid body rotations of the foundation. Additional modes may be arbitrarily selected as long as they constitute independent and admissible deformation shapes for the foundation (i.e., satisfy continuity and boundary conditions for the foundation). Following the methodology described below, the additional modes will always increase the accuracy of the results (barring numerical instability in the analysis) and impertinent modes will be made irrelevant by corresponding insignificant time-histories. Note that even though the foundation response is presented as superposition of different mode shapes, the methodology is not limited to linear applications since the only superposition done is on the forcing function and the transient analysis is performed concurrently for all mode shapes.

Once the modes are selected, the complete foundation motion from the 1st step of the analysis is sampled at finite (say $n>N$) "reference" nodes. At each time instant ($t = t_j$), the enforcement of Eq. 1 at each reference node provides a system of n equations and N unknowns. The unknowns are the values of the displacement time-histories at $t = t_j$. This system of equations can be solved using a least squares approximation at each time instant. The mathematical representation of this process is shown in Eqs. 2 through 5.

$$\mathrm{R}_{\mathrm{n}\times 1} = \Phi_{\mathrm{n}\times \mathrm{N}} \cdot \mathrm{D}_{\mathrm{N}\times 1} \tag{2}$$

$$\Phi_{\mathrm{n}\times \mathrm{N}} = \begin{bmatrix}\varphi_1 & \varphi_2 & \cdots & \varphi_{\mathrm{N}}\end{bmatrix} \tag{3}$$

$$\mathrm{D} = \begin{bmatrix}\mathrm{d}_1(\mathrm{t_j}) & \mathrm{d}_2(\mathrm{t_j}) & \cdots & \mathrm{d}_{\mathrm{N}}(\mathrm{t_j})\end{bmatrix}^{\mathrm{T}} \tag{4}$$

$$\mathrm{D} = \left(\Phi^{\mathrm{T}} \cdot \Phi\right)^{-1} \cdot \Phi^{\mathrm{T}} \cdot \mathrm{R} \tag{5}$$

where, $\varphi_1 \;\; \varphi_2 \;\; \cdots \;\; \varphi_N$ are the mode vectors for modes 1 through N, and $\mathrm{d}_1(t_j) \;\; \mathrm{d}_2(t_j) \;\; \cdots \;\; \mathrm{d}_N(t_j)$ are their corresponding time-histories at $t = t_j$.

The above calculation will be repeated at each time instant to obtain modal time-histories $\mathrm{d}_1(t)$, $\mathrm{d}_2(t)$, …, $\mathrm{d}_{\mathrm{N}}(t)$ corresponding to the N considered modes. Note that the $N \times n$ matrix $(\Phi^T \cdot \Phi)^{-1} \cdot \Phi^T$ is time-independent and will only need to be computed once.

Criteria for Assessment of Proposed Methodology

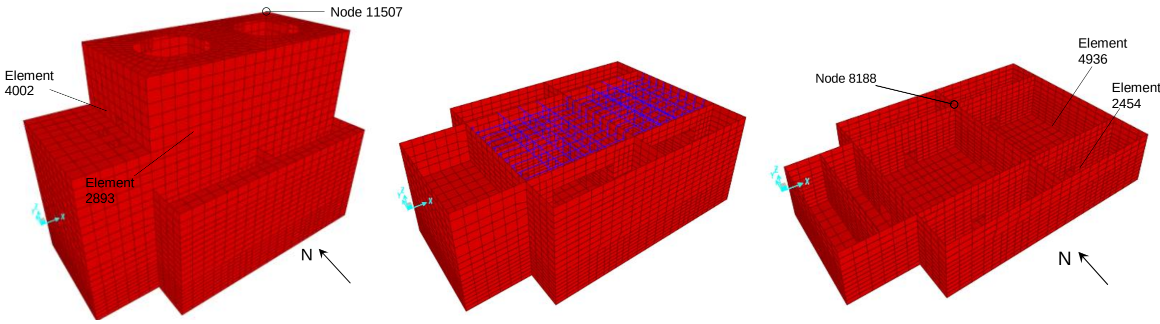
As described in the above section, the proposed method considers a certain number of significant spatial modes and determines their amplitudes by examining the displacement time-histories at selected reference nodes on the foundation. Subsequently, time-histories for the various spatial modes are generated at all foundation nodes, including the reference nodes. Therefore, an obvious point of comparison is that, for the reference nodes, the SASSI2010 generated displacement time-histories and the sum of time-histories of constituent spatial modes should match closely. Additional assessment criteria for the proposed method are based on the following comparisons:

1. ISRSs at various locations on the foundation (to help assure that the FIM frequency content is accurately preserved),
2. In-plane shear forces in finite elements used to model major shear walls (to help assure that the design of shear wall is performed for correct level of demand), and
3. ISRSs at select superstructure nodes (to help assure that seismic qualification of the floor or wall supported safety-critical system or equipment/component can be performed for accurate input motion).

A test problem, which is representative of small to medium large labyrinthine nuclear facility, was devised to study the accuracy of the proposed method. The test problem is described next.

Description of Test Problem and Analysis Cases

The selected structure, shown in Fig. 1(a), is a representative safety-related nuclear facilities building with a footprint of about 160 ft by 105 ft founded on a 6 ft mat foundation at El. 0 ft and comprises of reinforced concrete walls and slabs ranging in thickness from 2 to 3 ft, with the majority of walls being 3 ft thick and the majority of slabs of 2 ft thickness. Four major shear walls provide lateral load resistance in the long (East-West (X)) direction of the structure while three major shear walls provide resistance in the short (North-South (Y)) direction, as shown in the cross-sectional view in Fig. 1(c). The structure consists of two main floors extending on the entire footprint at El. 33 ft and El. 62 ft, and a third floor occupying the middle 63 ft of the North-South direction and the full width in the other direction at El. 80 ft. Steel beams form a platform between El. 46 ft and El. 62 ft as shown in the middle portion of the structure in Fig. 1(b). The roof is at El. 114 ft. The finite element (FE) model of the structure is developed using thick shell elements for the mat foundation, walls, and slabs. The main structural characteristics for the structure are summarized in Table 1. The fundamental natural frequencies of the structure in the horizontal directions and their corresponding mass participation factors (MPF) are presented in Table 2.



(a) Full Model (b) First Floor (EL 31.25 ft) (c) Basemat

Figure 1. Finite element model of selected structure – 3D views.

Table 1. Structure and model summary.

Footprint	160 ft x 105 ft
Height	114 ft
Foundation Thickness	6 ft
Wall and Slab Thicknesses	2 to 3 ft
Total Weight	76800 kips
Foundation Area	15500 ft^2
Average Soil Pressure	4.95 kip/ft^2
Concrete Elastic Modulus	3950 ksi

Table 2. Major fixed-base modal frequencies.

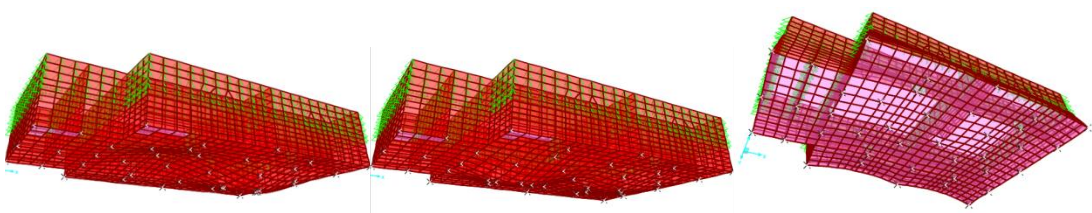
Direction	1st Mode		2nd Mode		3rd Mode	
	Frequency	MPF	Frequency	MPF	Frequency	MPF
East-West (X)	6.0 Hz	32%	8.4 Hz	17%	10.5 Hz	8%
North-South (Y)	5.3 Hz	18%	6.3 Hz	18%	9.5 Hz	26%

The structure is situated on a soil site with V_{S30}, shear wave velocity in the top 30 m of the supporting soil media, of 1205 ft/sec. The SSI analysis of the example structure is carried out using SASSI2010, which explicitly includes the FE model of the structure as well as the subsurface in a linear frequency domain analysis and uses the free-field seismic ground motions and strain-compatible soil profiles for the considered site as input. The SSI analysis is preceded by a site response analysis, which is carried out to obtain the free-field seismic ground motion at foundation elevation and strain-compatible soil properties at the building site. Since focus of this paper is on the two-step methodology following an SSI analysis, the details of the site response analysis and SSI analysis for the example structure are not presented.

As described in the methodology section, the foundation motion of the structure is described in terms of rigid body modes and additional foundation spatial modes. For the example structure, the results obtained from the following three cases are presented:

(1) Three-mode case, including the translational rigid body modes of the foundation,
(2) Six-mode case, including the three translational and the three rotational rigid body modes of the foundation, and
(3) Nine-mode case, including the six rigid body modes and three additional foundation modes as discussed below.

As discussed before, the additional foundation spatial mode shapes are not unique. Any independent and admissible deformation shape (i.e., not violating the continuity or boundary conditions for the structure) may be used as a spatial mode. For the example structure, three additional spatial modes are obtained by simply restraining the structural nodes above the mid-height of the 1st floor walls and obtaining the modal deformations for the unrestrained foundation. These three additional modes are presented in Fig. 2Figure 2.



(a) Mode 7 (b) Mode 8 (c) Mode 9

Figure 2. Additional foundation spatial modes for nine-mode case.

As an example, the displacement time-histories corresponding to the nine-mode case obtained following the methodology described before are presented in Fig. 3.

In the 2nd step, the displacement mode shapes and the corresponding time histories, developed based on the methodology described previously, are utilized in a 'fixed-base' time history analysis using SAP2000. This analysis is notionally called 'fixed-base', although displacement values are imposed at the foundation nodes, and may be performed with any commercial software that allows imposing loading patterns that are function of space and time. Since the foundation motion is sampled at a set of finite number *n* of reference foundation nodes, the time history analysis is performed by applying the loading patterns at these sampled nodes only, with each loading pattern defined for the number of modes considered and varying in space and time. The combined loading function is defined as the superposition of the multiple loading patterns. As an example, for the nine-mode case, the analysis is performed for a combined load function that consists of 9 loading patterns. Six of these loading patterns represent the rigid body translation and rotation modes (unit displacements and rotations) and their corresponding time histories, and the remaining three loading patterns represent the additional foundation mode shapes with their corresponding time histories. As described in Table 2, the dominant frequencies of the structure in the horizontal directions are between approximately 5 Hz to 10 Hz. Using this information, the mass and stiffness proportional Raleigh damping parameters are computed and assigned to the SAP2000 model, to ensure that the SAP2000 model is compatible with the SASSI2010 model, which incorporates structural damping directly in the frequency domain analysis.

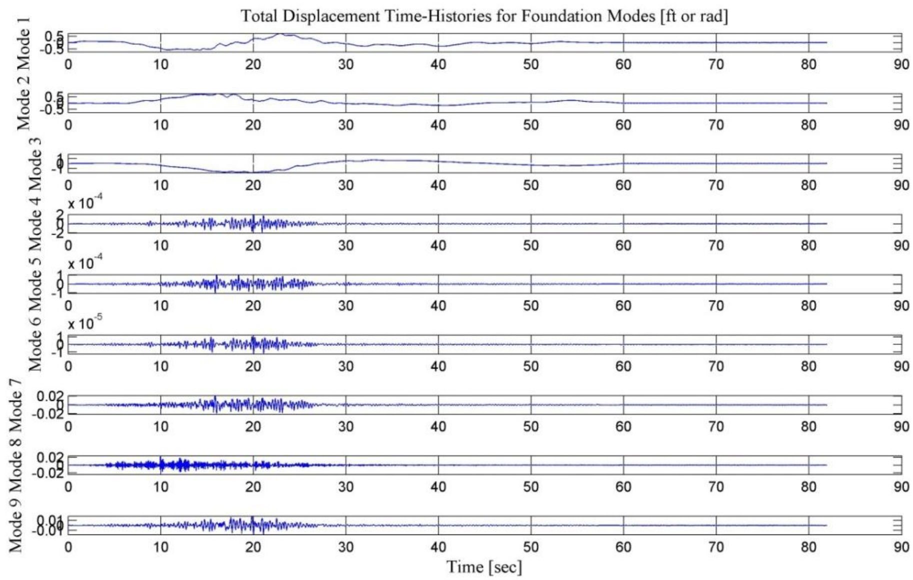
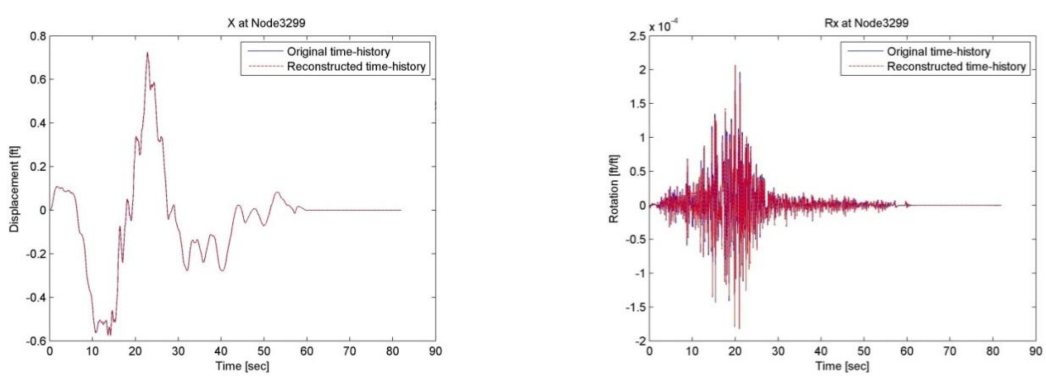


Figure 3. Time-histories corresponding to nine-mode case.

Discussion of Results

As described earlier, it is expected that the displacement time histories from SASSI2010 should match the displacement time histories reconstructed using Eq. 2 corresponding to the selected number of modes. Fig. 4 shows this comparison of one translation and one rotation time history for one of the foundation reference nodes for the nine-mode case. As expected, the reconstructed time histories match closely with the original foundation motion from SASSI2010; similar comparisons are observed for other degrees of freedom, with better match for higher number of modes considered in the analysis.



(a) Translation X (b) Rotation X

Figure 4. Comparison of displacement time-histories at a foundation reference node.

The analysis results from SASSI2010 and SAP2000 are post-processed to compute the ISRS at selected set of foundation and superstructure nodes and the stress resultants comprising of forces and moments for a selected set of elements located on main shear walls in both directions. Figs. 5 to 7 depict the comparison of X, Y, and Z ISRS at different elevations using the acceleration response spectrum directly from SASSI2010 and using the two-step method for the three-mode, six-mode, and nine-mode cases. The ISRS comparison is presented only up to 15 Hz, since this was used as the cut-off frequency in the SSI analysis.

It is evident from Figs. 5 to 7 that the proposed two-step method can be used to accurately determine the ISRS in the structure by including appropriate foundation modes. At the foundation level, the number of modes considered in the 2^{nd} step analysis does not have any significant impact on the ISRS in the horizontal directions, but the ISRS in the vertical direction are significantly improved by including the additional foundation modes that incorporate the foundation flexibility. The accuracy of ISRS in the horizontal directions at superstructure locations is significantly improved for the six-mode and nine-mode cases, with the nine-mode case also able to closely represent the high-frequency behavior of the structure.

Table 3 provides a comparison of the stress resultants for the major shear wall elements in both directions. The absolute maximum forces and moments from the two-step method implemented in SAP2000 match reasonably well with the corresponding element resultants obtained from SASSI2010 and the accuracy of the results is improved by including a larger number of modes.

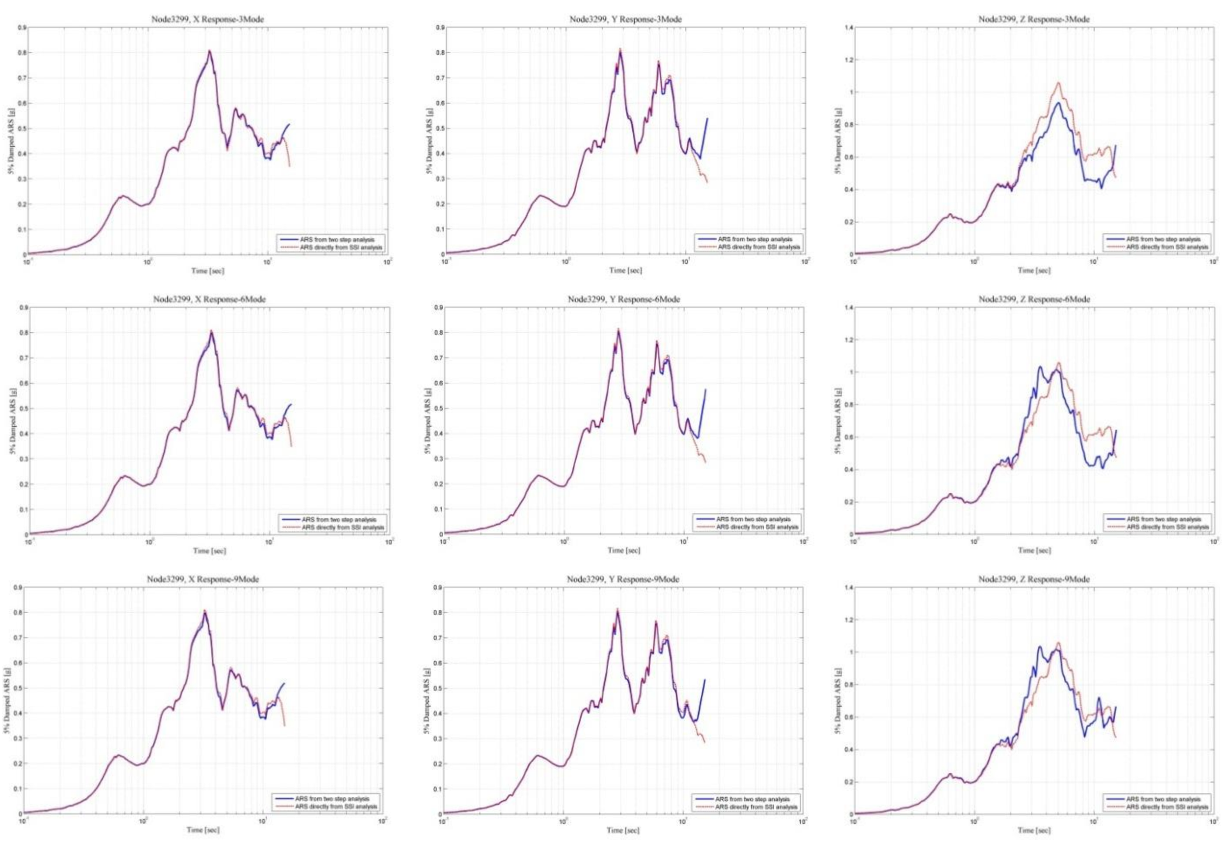


Figure 5. ISRS comparison at foundation level.

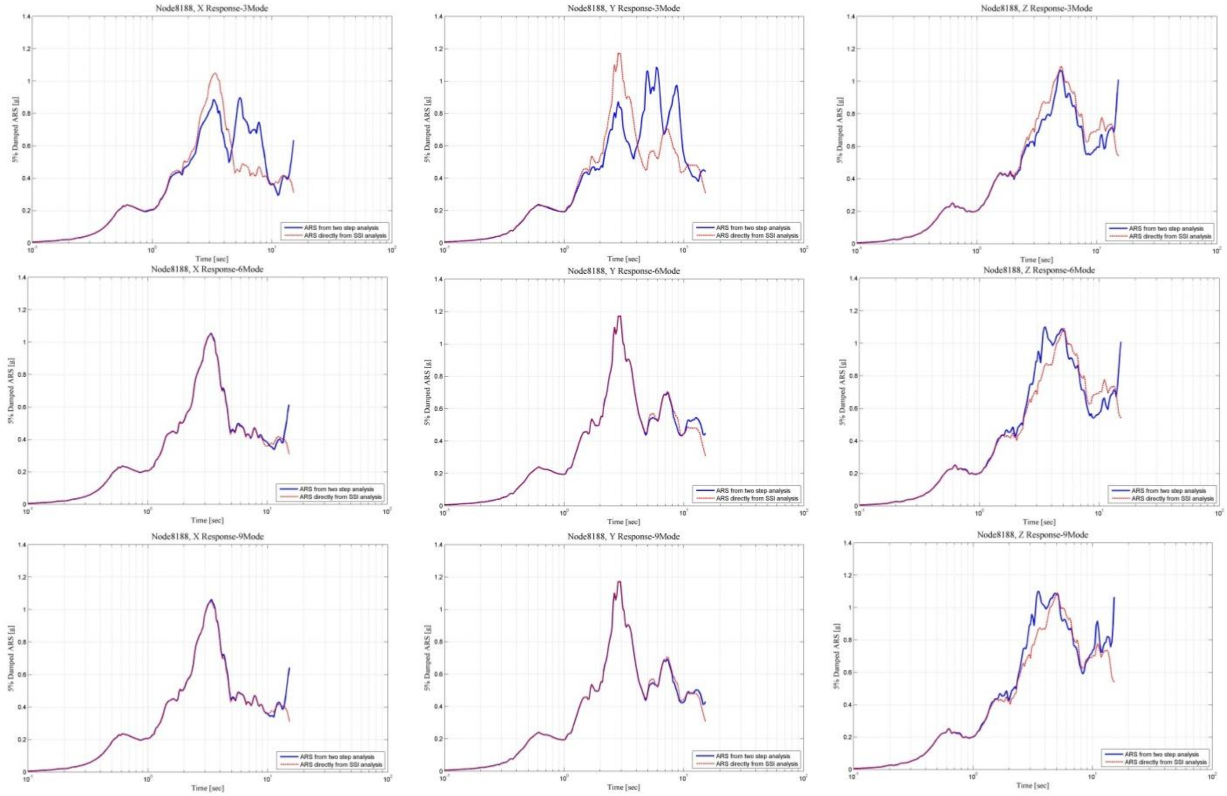


Figure 6. ISRS comparison at El. 33 ft.

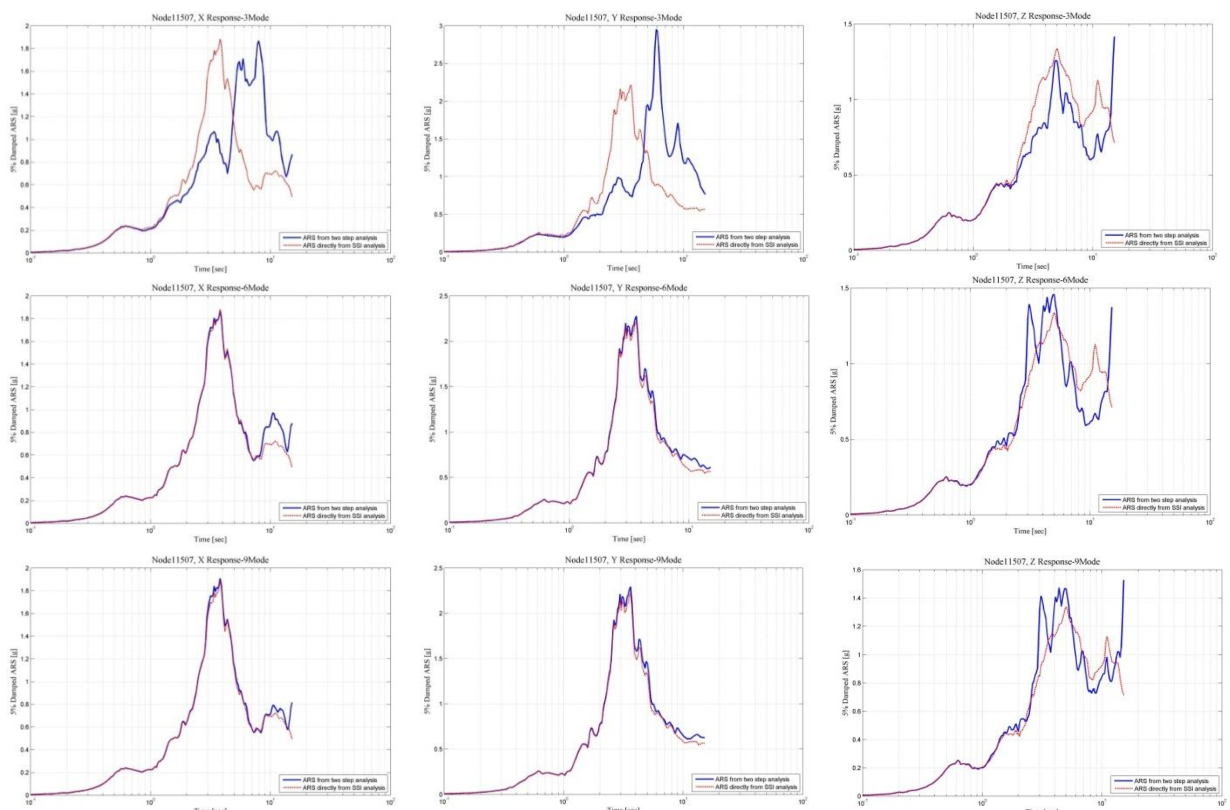


Figure 7. ISRS comparison at roof level.

Table 3. Comparison of in-plane shear (kip/ft) resultants for main shear walls.

Structural Member [Element #]	SSI	Two-Step Method		
		Three-Mode	Six-Mode	Nine-Mode
N-S Wall at Foundation Level [4936]	46.71	39.98	43.24	51.98
E-W Wall at Foundation Level [2454]	40.88	58.45	49.97	52.38
N-S Wall at El. 81 ft [4002]	20.28	30.14	23.34	22.93
E-W Wall at El. 81 ft [2893]	18.66	17.62	17.78	17.67

Summary and Conclusions

A new two-step method for seismic analysis of mission-critical structures has been presented. In the first step, a conventional SSI analysis is performed using a program such as SASSI2010. The first step analysis may involve a somewhat coarse finite element mesh that is deemed refined enough from seismic analysis standpoint. The displacement time-histories at selected foundation nodes are then used to develop time-histories for the individual spatial mode shapes that are deemed to make up the total motion. Increasing number of mode shapes are used to gain accuracy. Using the spatial mode shapes, the incoherent foundation motion is modeled as a sum of various time-histories of coherent spatial mode shapes.

The results show that the spatial mode shapes due to foundation flexibility can be important when the basemat is not rigid. Comparison of stress resultants in major shear walls and superstructure ISRSs indicated that accurate results are obtained when just three additional basemat mode shapes are considered in addition to the six rigid body mode shapes (associated with three translations and three rotations due to SSI effects). It is expected that the improved two-step method could be used to study certain kinds of nonlinear response in the second step provided that the nature and degree of nonlinearity is such that the FIM obtained from the first step (equivalent linear) SASSI2010 analysis remains largely unchanged. Example of nonlinear response that could be studied are: minor and momentary episodes of foundation uplift (loss of contact) due to seismic overturning action, small episodes of stick-slip due to sliding at the soil-foundation interface, and nonlinear (hysteretic) in-plane shear behavior of major shear wall elements. These will be the subject of a future study by the authors.

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