Effect of Soil-Structure-Interaction on Identified Modal Parameters and Damage Localization

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SUMMARY:
This study focuses on understanding the effect of nonlinear seismic soil-structure-interaction (NLSSI) on identified modal parameters and damage localization. A numerical model of a four-storey building resting on isolated shallow foundations has been developed for this purpose. A Winkler-based model has been used to simulate nonlinear SSI. The building has been modelled alternatively using elastic and nonlinear degrading elements to study the effect of nonlinear SSI on identification of structural damages. A series of nonlinear time history analyses has been performed with varying intensity ground motions that are padded before and after with a low intensity band-limited white-noise to simulate the ambient conditions. The logic behind choosing the ground motions of various intensity levels is to investigate the response of the structure with nonlinear foundation. Finally, the effect of SSI on damage detection and localization has been investigated by studying the change in pattern of modal properties.

Keywords: structural damage detection, system identification technique, nonlinear soil-structure interaction

1. INTRODUCTION

In current practice, if a structure appears to be damaged, re-occupation is not allowed until safety evaluation is performed by means of visual inspections and subsequent recommendations. This process is experience-based, highly time consuming and expensive. In order to understand how vibration-based system identification methodologies can be used to develop a faster and reliable method for post-disaster safety evaluation, several buildings, bridges and other structures have been instrumented around the world to record and archive real-time vibration data. The research emphasis in this area can be observed by a large number of articles and conference papers published in recent years. However, contrary to laboratory experiments involving structural models, it seems that the detection of damage in a structure during a earthquake using the identified modal parameters is a complex procedure. This is because changes in the identified modal parameters may not be the direct consequence of structural damage. In fact, in a few recent studies, it has been shown that the measured instantaneous (apparent) period of a few structures varied significantly before, during, and after an earthquake even though there was no visible damage to these structures. In these cases, the changes in modal properties due to earthquakes have been attributed to various factors including behavior of nonstructural components such as partition walls and nonlinear behavior of underlying soil.

Vibration-based structural damage detection has drawn considerable attention in recent years due to its nondestructive nature. Since the last few decades, several methodologies have been developed for this purpose, which may overcome the difficulties associated with a visual inspection in terms of downtime and reliability of evaluation. Brincker et al. (2001) proposed a frequency domain method for structural modal identification of output-only systems, using cross-power-spectral-density (CPSD) of the structural responses. Caicedo et al. (2004) demonstrated the efficacy of natural excitation technique (NExT) and eigensystem realization algorithm (ERA) by using simulated data generated from the popular IASC-ASCE benchmark structure. Lu and Gao (2005) proposed an efficient time-domain technique based on residual error of autoregressive prediction model for structural damage localization. Zhang et al. (2010) developed a probabilistic damage detection approach for output-only structures with parametric uncertainties.

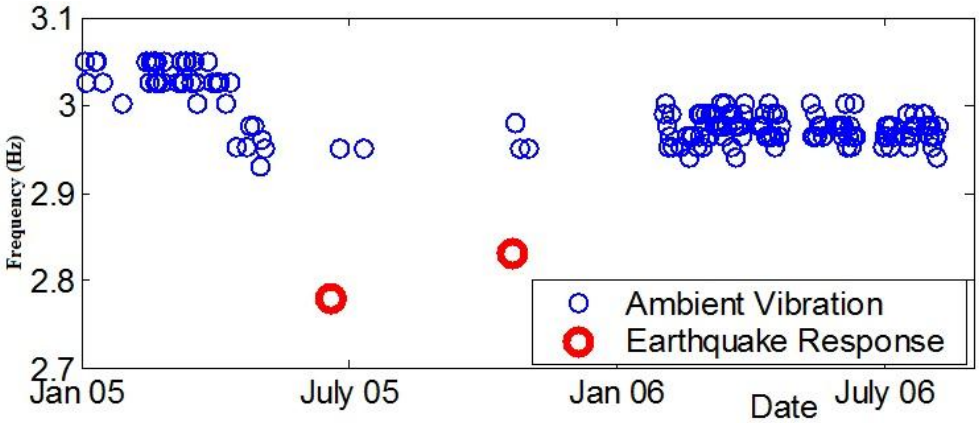

Figure 1.1. Fundamental frequency (Hz) of Cal-IT building in the longitudinal direction

To highlight the effect of soil-structure-interaction (SSI) on structural health monitoring results, the study by Soyoz (2007) is of particular interest. This study deals with the CAL-IT2 building at the University of California, Irvine, which is supported by isolated square and strip shallow foundations. This building was fully instrumented with sensors that includes free-field and foundation level accelerometers for long term health monitoring. Fig. 1.1 shows the fundamental frequency of the building before, during (instantaneous) and after earthquakes. This long-term identification study shows that during the earthquakes the lower frequencies dropped by a considerable amount. However, in most cases the frequencies returned nearly to its pre-event values thereby indicating minor or no damage to the structure. It was also noted that other factors such as behavior of nonstructural components and nonlinear SSI may have played a role in this frequency reduction during the earthquakes as no noticeable structural damage was observed after these events.

Another study conducted by Kohler *et al.* (2005) on UCLA Factor building has also shown the change in instantaneous frequency during an earthquake. This steel-frame building was instrumented to record seismic and ambient responses. Fig. 1.2 shows the change in instantaneous frequencies in two orthogonal directions during an earthquake versus the relative time with respect to the arrival of the earthquake. The possible reason behind this sudden drop in the frequencies as pointed by Kohler *et al.* was the effect of nonlinear SSI among other factors such as softening of the soil strata (the temporal as well as spatial alteration in soil parameters) beneath the building during tremor. The effect of SSI could not be ascertained from the measured responses as the data were collected only above the third story and no free-field and bore hole data were available.

Todorovska and Trifunac (2007) also recognized the SSI effect on modal properties from the seismic response of a six storey reinforced concrete building (ICS building, El Centro, California). During the 1979 Imperial Valley earthquake, the fundamental frequency in either direction of this building dropped by more than 25%. Todorovska and Trifunac (2008) identified the SSI to be the most likely reason behind the drop in the instantaneous frequency during an earthquake and justified by a simulation study in ETABS. In another study on a seven-story reinforced-concrete hotel building in Van Nuys, California, Trifunac *et al.* (2001a) demonstrated that the apparent frequency during various intensity ground motions may drop by up to 64% of its pre-event value only to regain nearly its pre-event value after the event. It was concluded that a strong nonlinear SSI was the reason behind such a drop of frequency during the earthquakes. This drop in frequency may also be observed even in case of low intensity shaking due to various causes such as presence of gap elements and softening of soil strata (Trifunac *et al.*, 2001b).

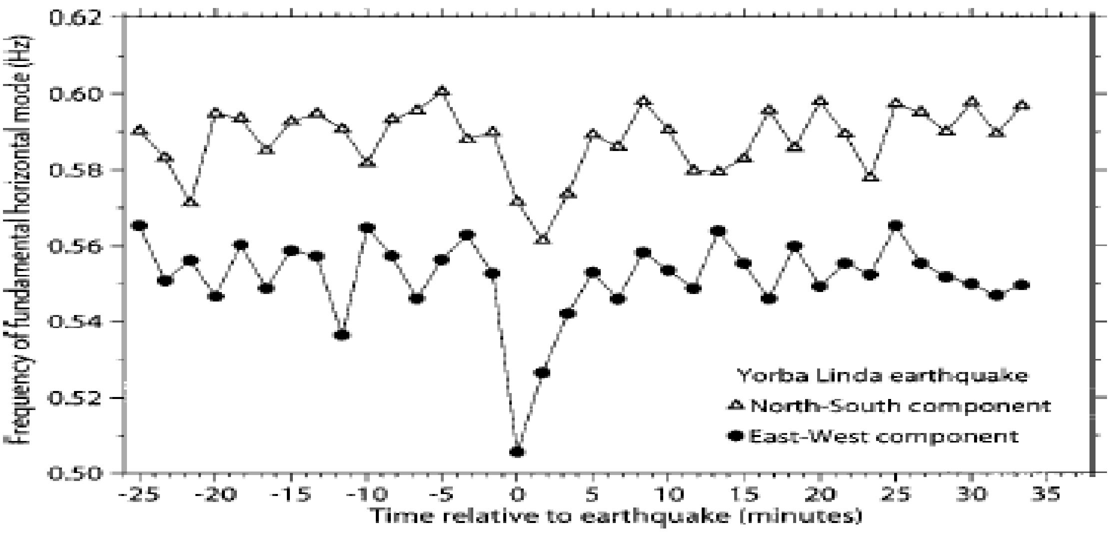

Figure 1.2 Frequency change with relative time of earthquake [Kohler, Davis and Safak(2005)]

Although it is now widely recognized that SSI plays a major role on measured frequency (apparent), many researchers continue to ignore the effect of SSI while focusing on frequency-based damage detection. For example, in a recent study, Shiradhonkar and Shrikhande (2011) used finite element model updating method for seismic damage detection of a fixed base moment-resisting frame considering two real strong ground motions. However, such studies are only be useful for real structures when the effect of nonlinear SSI is also included in modelling.

In this study, a numerical model of a four-storey building resting on fixed, elastic and nonlinear foundations is analyzed under a few ground motions of varying hazard levels. The effect of SSI on damage detection and localization has been investigated by studying modal properties. The aim of this study is to investigate the combined effect of structural damage and foundation nonlinearity under severe (2% exceedance in 50 years) ground motions. For this purpose, the first few modal frequencies are studied under damaged and undamaged conditions in presence of nonlinear soil-structure-interactions (NLSSI) to distinguish the influence of structural damage from that of NLSSI. A brief details of structure, foundation and numerical modelling is described in the following sections.

2. NUMERICAL STUDIES

2.1. Model Description

In this study, a three-bay four-storey steel moment-resisting frame building is considered. The building was previously designed by Santa-Ana and Miranda (2000) and also considered in Ray-Chaudhuri and Villaverde (2008). The building was designed following a weak-beam strong-column philosophy. A representative numerical model of this building is developed using OpenSees version 2.3.1 (2011). To study the effect of SSI, three different foundation conditions, namely, fixed base, elastic base and nonlinear base are considered here. For modelling the flexible base conditions, the Beams-on-Nonlinear-Winkler-Foundation (BNWF) approach is used (Raychowdhury and Hutchinson, 2009, Raychowdhury, 2011).

Figure 2.1 shows the schematic diagram of the two-dimensional numerical model along with nonlinear soil springs, which was developed in the framework of OpenSees. Structural elements were modelled using nonlinear beam-column elements with 3% kinematic hardening. A uniform seismic mass distribution along the height was considered with typical floor mass of 16000 kg. These masses were lumped at nodal points of each floor in such a way that the mass of interior nodes are twice the mass of the exterior nodes to mimic a realistic mass distribution scenario. The details of the steel sections used in modelling the structure are given in Table 2.1. One may notice from this table that the interior columns are much stiffer than that of the exterior columns. An eigenvalue analysis was performed under the fixed base condition and the first four frequencies are found to be 1.28 Hz, 4.39 Hz, 8.77 Hz and 13.19 Hz, which show a good agreement with Ray-Chaudhuri and Villaverde (2008).

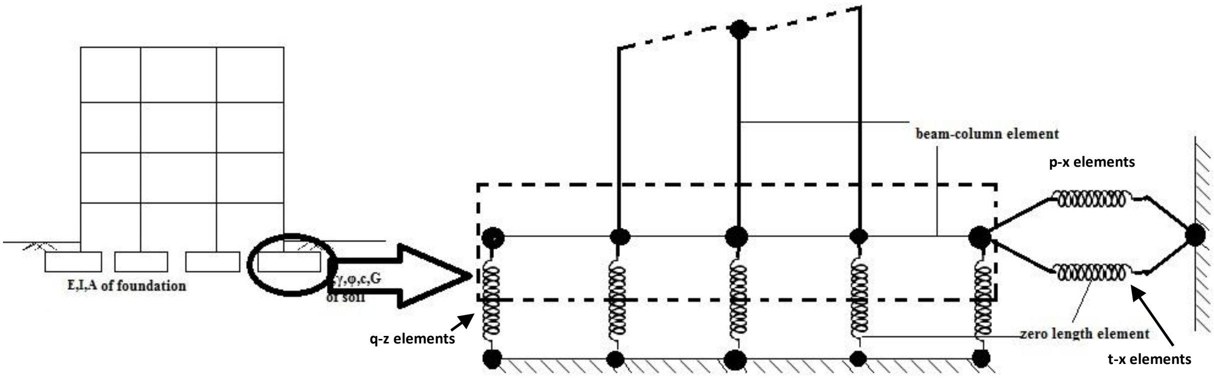

Figure 2.1 Soil-structure system with Nonlinear-Winkler-foundation Model

Table 2.1 Steel structural sections used in modelling (Santa-Ana and Miranda, 2000)

Storey	External Column	Internal Column	Beam
1	W14x109	W14x176	W24x104
2	W14x109	W14x176	W24x104
3	W14x90	W14x145	W24x94
4	W14x90	W14x145	W24x94

2.2. Foundation Description

Three different foundation conditions (fixed based, elastic and nonlinear foundation), adopted from Raychowdhury (2011), are considered in this study (see Figure 2.2). Assuming a factor of safety 4, the interior and exterior footing sizes are determined as $2m \times 2m$ and $1.56m \times 1.56m$, respectively. The parameters used throughout the analysis are as follows: $c = 5\text{kP}a$, $\Phi = 36$; $\gamma = 18\text{kP}a$ and $G = 50\ \text{MP}a$. Mathematical modelling of these foundations is done according to the calibrated BNWF model using the *ShallowFoundationGen* command in OpenSees (Raychowdhury and Hutchinson, 2009).

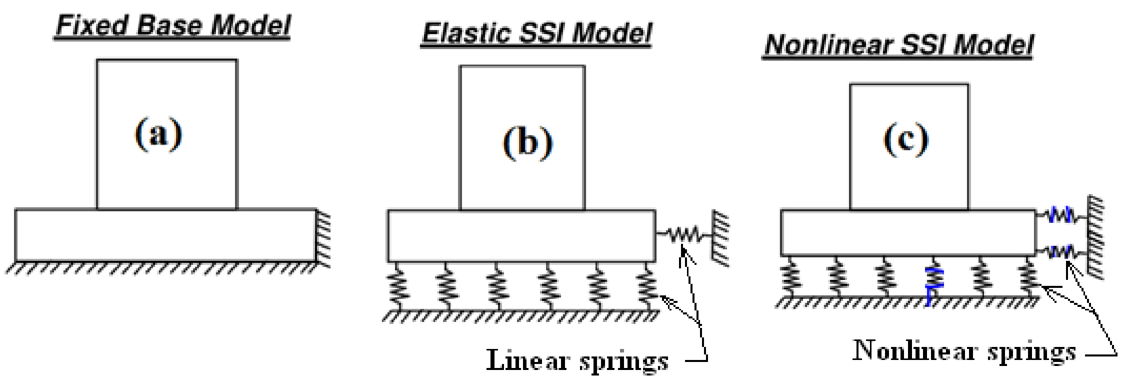

Figure 2.2 Schematic Diagram of shallow isolated footing [Raychowdhury (2011)]

3. DAMAGE ASSESSMENT

Typically, a damage assessment involves two sets of data. One set deals with the information of undamaged state, sometime known as the reference state, and the other set involves the state for which the condition is to be assessed. Usually the second set of data refers to the information collected after any hazardous event such as an earthquake, which may cause damage to a structure. Finally, the information extracted from the data after such an event is compared with the reference state to detect any possible damage.

In this study, a series of nonlinear time history analyses has been performed with ground motions of different hazard levels and padded before and after with a low intensity band-limited white-noise data to simulate ambient vibration conditions. To avoid any possible interference between the white-noise and the actual ground motions, a considerable time gap is ensured between the ambient and seismic excitations. The gap after a seismic event is particularly important in order to die down the seismically excited motions by structural damping. Earthquake motions considered for numerical simulations are those from the FEMA/SAC motions: LA47 with 50% probability of exceedance in 50 years and LA36, LA39 and LA40 with 2% probability of exceedance in 50 years. (http://nisee.berkeley.edu/data/strong_motion/sacsteel/ground_motions.html).

In this study, the damage detection and localization are performed based only on the ambient response data, which generally is the case for most of the civil engineering applications. One of the damage detection algorithms used in this study involves the use of natural excitation technique coupled with eigensystem realization algorithm (NExT-ERA) for modal parameters estimation from an ambient response data.

4. RESULT AND DISCUSSION

To study the effect of soil-structure interaction on the modal properties of a structure before and after an earthquake, time history analyses of the building with different base conditions are performed using the aforementioned padded (before and after) ground motions. Structural responses under different footing conditions are then recorded and some typical results are shown for various base conditions and hazard levels.

4.1. Effect of SSI on Identified without Structural Damage

Table 4.1 shows the modal frequencies of the structure before and after an event along with the corresponding percentage drops in frequency for the structure with nonlinear base. Note also that no permanent drop in frequency is observed for the structure with linear base. This is expected as no stiffness degradation was assumed for material modelling of the structural elements. Thus, it is quite clear that the permanent drop in frequency in case of nonlinear base is due to the nonlinear behaviour of the foundation. It may also be noted that the amount of drop in frequency can be related to the hazard level of ground motions implying that different intensity motions cause varied level of change in frequencies.

Table 4.1 Identified frequencies before and after a mild (50% in 50 years) ground motion (LA 47)

NLSSI				Elastic				Fixed			
Mode	**f_{before}**	**f_{after}**	**f_{drop}**	**Mode**	**f_{before}**	**f_{after}**	**f_{drop}**	**Mode**	**f_{before}**	**f_{after}**	**f_{drop}**
I	1.28	1.27	0.92	**I**	1.27	1.27	0.00	**I**	1.45	1.45	0.00
II	4.39	4.39	0.06	**II**	4.38	4.38	0.00	**II**	4.66	4.66	0.00
III	8.77	8.76	0.08	**III**	8.76	8.76	0.00	**III**	8.96	8.96	0.00
IV	13.19	13.19	0.01	**IV**	13.18	13.18	0.00	**IV**	13.31	13.31	0.00
V	21.44	21.44	0.00	**V**	19.52	19.52	0.00	**V**	21.45	21.45	0.00

f_{before} : pre-event frequency, f_{after} : post-event frequency, f_{drop} : frequency drop

For a mild ground motion with 50% probability of exceedence in 50 years (LA47), the drop in the fundamental frequency in case of NLSSI is as little as 1%, whereas in cases of severe (LA40 with 2%

in 50 years) ground motions, the reduction in the fundamental frequency is about 10% to 16%. This trend in drop of frequency with change in hazard level is also observed for higher modes as well. One can notice however that for a given hazard level motion, the amount of drop in frequency reduces as the mode number increases. This trend was also observed in Ray-Chaudhuri (2007). Thus, based on this trend one can infer that the drop in frequencies is the result of change of bottom story stiffness. Since, there was no permanent drop of structural material stiffness, it is concluded that there is a change of stiffness of supporting soil due to earthquake motions. In fact, from the literature on BNWF, it can be found that due to permanent deformations such as vertical settlement and sliding, there is permanent drop in stiffness of the soil-foundation system.

Table 4.2 Identified frequencies for structure with NLSSI

LA36				LA39				LA40			
Mode	**f_{before}**	**f_{after}**	**f_{drop}**	**Mode**	**f_{before}**	**f_{after}**	**f_{drop}**	**Mode**	**f_{before}**	**f_{after}**	**f_{drop}**
I	1.28	1.06	16.86	**I**	1.28	1.20	6.51	**I**	1.28	1.20	6.16
II	4.39	4.26	3.05	**II**	4.39	4.32	1.57	**II**	4.39	4.36	0.67
III	8.77	8.65	1.38	**III**	8.77	8.71	0.65	**III**	8.77	8.73	0.51
IV	13.19	13.15	0.34	**IV**	13.19	13.17	0.21	**IV**	13.19	13.18	0.10
V	21.44	21.44	0.02	**V**	21.44	21.44	0.01	**V**	21.44	21.44	0.01

In order study the drop in modal frequencies due to NLSSI for higher intensity of ground motions, three different ground motions of 2% in 50 years hazard levels are considered. Table 4.2 provides the first few identified frequencies along with their percentage drop due to NLSSI for three different ground motions LA36, LA39 and LA40, all with 2% in 50 years hazard level. It can be observed from this table that the fundamental frequency is more influenced by NLSSI in comparison to its higher frequencies for the same reason mentioned earlier as per the study by Ray-Chaudhuri (2007). Comparing Tables 4.1 with Table 4.2 one can notice that higher intensity motions produce more drop in natural frequencies. The highest drop observed in this case is about 17% for the fundamental mode that with a foundation with vertical factor of safety of 4.

Table 4.3 Identified frequencies for damaged structure with NLSSI

LA36				LA39				LA40			
Mode	**f_{before}**	**f_{after}**	**f_{drop}**	**Mode**	**f_{before}**	**f_{after}**	**f_{drop}**	**Mode**	**f_{before}**	**f_{after}**	**f_{drop}**
I	1.28	1.01	21.32	**I**	1.28	1.10	14.47	**I**	1.28	1.09	15.06
II	4.39	3.97	9.58	**II**	4.39	4.04	7.99	**II**	4.39	4.01	8.64
III	8.77	8.06	8.13	**III**	8.77	8.10	7.61	**III**	8.77	8.09	7.71
IV	13.19	12.16	7.85	**IV**	13.19	12.18	7.67	**IV**	13.19	12.17	7.76
V	21.44	21.07	1.74	**V**	21.44	21.07	1.73	**V**	21.44	21.07	1.73

4.2. Effect of SSI on Identified Frequencies in Presence of Structural Damage

To find the combined effect of NLSSI and structural damage on identified modal frequencies, three ground motions used earlier (2% in 50 years - LA36, LA39 and LA40) are considered here. To simulate structural damage under these events, a 15% reduction in the 3rd story stiffness is introduced during time history analysis. Table 4.3 provides the identified frequencies before and after such earthquake events. One can observe from this table that the drop in the fundamental frequency is more than that of the higher mode frequencies. However, the trend of drop of frequencies due to NLSSI (frequency drop reduces with increase in mode number) as observed earlier is somewhat different in these cases. In fact, it can be noted that the frequency drops corresponding to 2nd, 3rd and 4th are somewhat similar and in some cases, the drop in the 4th mode is more than that of the 3rd mode. In addition, the percentage drop in frequencies of the 2nd, 3rd and 4th mode is significant (more than 7%). This is because of presence of damage in the third story level in addition to the effect of NLSSI. By comparing the drop in frequencies corresponding to the 2nd, 3rd and 4th in Tables 4.2 and 4.3, one can observe that the increase in drop in frequencies of these modes due to structural damage is almost same for all ground motions. In other words, if the effect of NLSSI is isolated from the frequency drop in Table 4.3, it is found that the effect due to structural damage is independent of ground motions.

4.3. Mode Shape Curvature to Differentiate NLSSI and Structural Damage - Localization

Mode shape curvature is a good measure for damage localization and a change in mode shape curvature can be used as a damage indicator (Panikkaveettil, 2012). Thus, in this study, using the ambient response, the mode shapes are calculated and the mode shape curvature are determined.

To study how the change in mode shape curvature can be used to characterize damage, it is assumed that a 15% reduction in the 3rd floor stiffness occurs during an earthquake event. Three base conditions, fixed, elastic and nonlinear are also considered to understand the effect of SSI on mode shape curvature. Figure 4.1 depicts the change in the 1st mode shape curvatures for the structure with fixed, elastic and nonlinear base conditions. The curvature values were calculated from the mode shapes using central difference method. It can be observed from the plots of three base conditions that between the 2nd and 3rd floors, the curvature change shows a clear sign change. This implies the presence of a structural damage between the 2nd and 3rd floors. It can also be noted that there is a clear change of sign for curvature change between the ground and first floors for the NLSSI case, implying permanent change of base stiffness. A change of sign at the base for the fixed and elastic base condition is also observed (which is insignificant with respect to that associate with the structural damage) due to the curvature calculations at the boundary. It can however be noted that a damage in first story stiffness may be difficult to isolate from that of a structural damage.

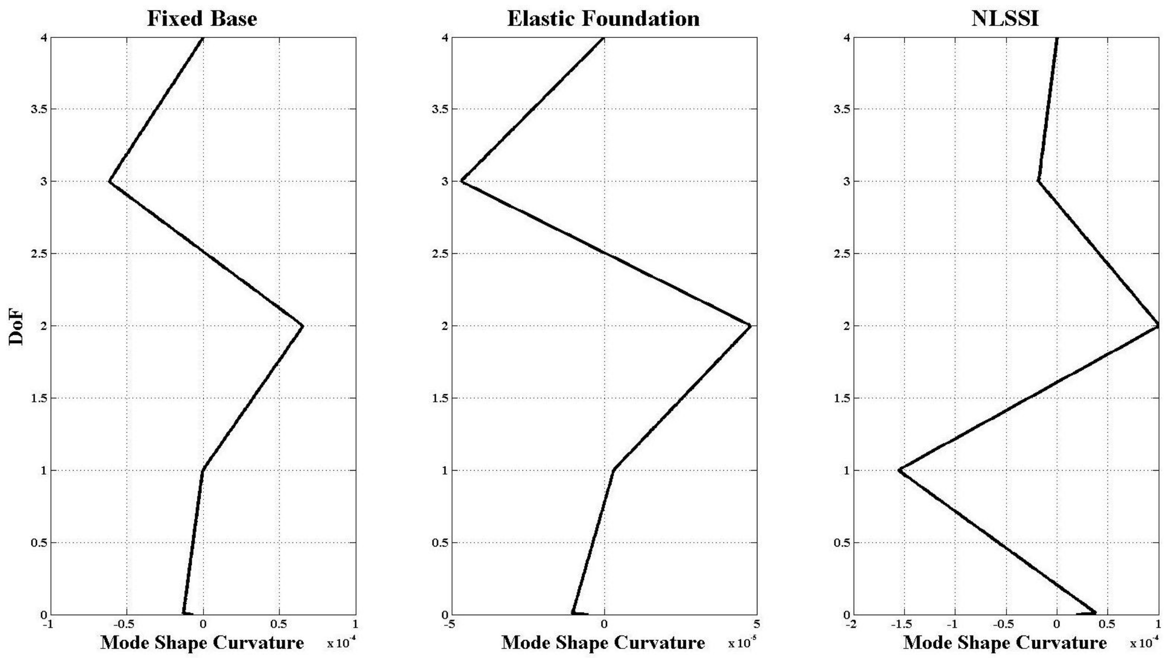

Figure 4.1 SSI Effect in Damage Localization (LA40)

5. CONCLUSION

This study focuses on isolating the effect of SSI from structural damage. Modal identification-based techniques are applied for a four-storey three-bay steel moment resisting frame structure with fixed, elastic and nonlinear foundations. The permanent changes in modal frequencies are then studied through ambient vibration response. The results show that the fundamental frequency drops as high as 16% after an event for a structure resting on nonlinear foundations. It has also been found that the amount of drop in frequency due to NLSSI reduces as the mode number increases. The method based on the change in mode shape curvature for localizing damage was found to be successful in detecting and localizing damage under all foundation conditions considered in the study. However, it is imperative that the effect of NLSSI and a damage in the 1st story may not be distinguishable. This is because of NLSSI producing changes in the boundary conditions resulting in the 1st story stiffness degradation.

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