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SYSTEM IDENTIFICATION AND DAMAGE DETECTION OF R.C. STRUCTURE.

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Abstract. *Evaluation of system parameters of civil engineering structures using system identification techniques has attracted considerable attention in recent years. Recorded dynamic responses of the structures due to ambient or earthquake excitations are utilized for identification of system parameters of the structures. Huge amount of works have been carried out in the field of linear time invariant system identification; but the application in time varying system identification is very limited. In the present study, three numbers of numerically simulated models of 1/5th scaled bare frame two storey R.C. buildings have been considered. Nonlinear time history analyses are carried out to detect the changes of modal parameters of the deteriorating structure. Further, acceleration responses are collected from different floors during base excitation of models. Linear Deterministic-Stochastic Identification (DSI) technique (e.g. N4SID) is then used for the identification of modal parameters (natural frequency). Since, a linear DSI algorithm would not be applicable for structures excited to nonlinear range, hence modal parameters are obtained for short data time windows. These instantaneous modal parameters are found to represent the deterioration of the structure reasonably well. Further, an approach referred to as the sequential non-linear least-square estimation (SNLSE) is used to compare the identified result obtained from N4SID algorithm using short time data window. In this approach, acceleration responses and external excitations are measured, whereas the structural parameters and response state vector are estimated similar to the Extended Kalman Filter (EKF) approach. It is superior to EKF in terms of stability and convergence.*

1 INTRODUCTION

In developing and developed countries, where exists a lot of aged important and costly structures exists has survived various intensities of earthquakes throughout its life span. A concern is raised in the minds civil engineering community about the health of such structures. Whether, any major internal externally invisible damage has occurred or estimating the life of such structures or whether it can survive any other major earthquake or whether it can carry any further load as in the case of bridges. Traditional damage detection techniques for identifying local damages referred to as non-destructive testing (NDT) such as CT scanning and ultrasonic, etc., are taboo these days with the advent of system identification techniques based on vibration data.

But most of the theories upon which structural dynamic analysis is founded rely heavily on the assumption that the dynamic behavior of the structure is linear. Failure to obey the superposition principle implies that the structure is nonlinear. In fact, most practical engineering structures have a certain degree of nonlinearity due to nonlinear dynamic characteristics of structural joints, boundary conditions and material properties. For practical purposes, in many cases, they are regarded as linear structures, whenever the degree of nonlinearity is small and therefore insignificant in the response range of interest, whereby linear model analysis methods can be applied to analyze their dynamic characteristics. Civil structures are designed and exhibit highly non-linear characteristics under severe loads such as strong seismic excitation. Thus, non-linear structural identification (SI) is necessary for civil infrastructure if health monitoring or accurate simulation of response is needed. As per Ewins 0,the identification process for linear multi-degree of freedom (DOF) systems is now mature and the methods, whilst operating in the time or frequency domains and using a variety of intermediate models, almost all yield a final model based upon modal parameters, viz. natural frequency, mode shape, modal damping and modal mass. Earlier attempts for identification of nonlinear systems can be found in the works of Masri and Caughey [2], Zhang et al. [3] based on curve fitting methods, with little success. Kerschen et al.[4] classified the nonlinear system identification in the literature into the following seven categories: time-domain methods, frequency-domain methods, time-frequency methods, methods that by-pass nonlinearity is using linearization, modal methods, black box methods and structural model updating methods.

Overschee and Moor [5] proposed subspace identification approach for linear system and implemented the same in practical application. A combined deterministic-stochastic algorithm namely Numerical Algorithm for Subspace State Space System Identification (N4SID) is a popular algorithm in research community as well as for practicing civil engineers for linear system identification of civil structures. It uses acceleration inputs at various DOF's and ground excitation to identify the modal parameters like eigenvalues, eigenvectors, damping etc.

Deterministic-stochastic subspace identification (DSI) technique was utilized successfully by Moaveni and Asgarieh [6]0 an input-output parametric linear system identification method, for characterization of nonlinear dynamic structural system based on time varying amplitude-dependent instantaneous (i.e. based on short-time window) modal parameters. They applied the technique both on analytical and physical hysteretic models and claimed that it performs much better than wavelet based analysis. As N4SID is also a deterministic-stochastic subspace identification method, this is applied in this present work to verify its applicability and feasibility to analytical nonlinear models.

N4SID suffers drawback as it is purely a black-box system identification tool and the results are difficult to interpret by engineers until recently Kim and Lynch [7] in their paper on black-box system identification attempted to provide a detailed explanation on geometrical interpretation of N4SID algorithm to provide civil engineers a deeper appreciation with the algorithm. N4SID is quite robust tool for linear system, few researchers Paulo et al. 08] tried to modify the linear N4SID to bilinear systems by playing with the system matrix. Our civil structures which are nonlinear cannot be generalized to behave in bilinear manner. Still it needs to be explored more.

Since the development of Kalman Filter in 1960 by Rudolf E. Kalman, extensive use of kalman filter for estimate of state variables has been found in literature. The Kalman filter estimate is sometimes denoted the “least mean-square estimate”. The kalman filter algorithm was originally developed for systems assumed to be represented with a linear state-space model. The kalman filter for nonlinear models is denoted the Extended Kalman Filter (EKF). Many researchers used EKF for nonlinear structural system identification with various modifications. Hoshiya and Saito [9]0 applied EKF to the system identification problem of seismic structural systems. A weighted global iteration procedure with an objective function was proposed for stable and convergent estimation, which was incorporated in EKF. Investigation was carried out for MDOF, bilinear hysteretic systems. Yang and Lin [10] carried out on-line identification of non-linear hysteretic structures using an adaptive tracking technique. The hysteretic model was modeled using Bouc-Wen model which was incorporated in the nonlinear term of the EKF. The main drawback was that generalized nonlinearity could not be handled.

In order to remove all the drawbacks of LSE and EKF approaches for the on-line system identification, a new approach referred to as the sequential non-linear least-square estimation (SNLSE) was proposed by Yang and Lin [11]0. Simulation results for both linear and nonlinear structures will be presented to demonstrate the efficiency of the proposed adapted SNLSE in tracking the structural damage. In this approach, acceleration responses and external excitations are measured, whereas the structural parameters and response state vector are estimated similar to the EKF approach. It is superior to EKF in terms of stability and convergence.

With the advent of the recently developed SNLSE technique, an attempt was made in this paper to identify the modal parameters of degrading RCC structure excited to nonlinear range (material nonlinearity) using the SNLSE technique and N4SID technique with short time data window and compare the result with that obtained from exact analysis using “OpenSees” software.

2 DESCRIPTION OF THE LABORATORY SCALED MODELS

Three bare frame models are considered and designated them by Model-1, Model-2 & Model-3. These Models were developed considering a RC prototype with a scale factor of 1/5. Nonlinear dynamic analysis was performed using OpenSees Software.

2.1 Similitude & Scaling

The laws of similitude are followed to arrive at appropriate input motion, dimensions of test models as well as for finalization of additional mass requirement for gravity load simulation. The similitude requirements are derived from dimensional analysis which depends on physical characteristics like length, force, time. Table 1 shows derived scaling relationship of the parameters relevant to the present study.

Parameters	Scale	Prototype
		1/5-scale model
Length	S	5
Mass	S^2	25
Displacement	S	5
Time	$\sqrt{S}$	2.236
Acceleration	S	5

Table 1: Similitude Requirement.

2.2 Geometrical and material properties of the models

The test models have been designated as Model I, Model II and Model III. The descriptions of various geometrical properties of all the test models are furnished in Table 2 and the various material properties are shown in Table 3.

Particulars	Model I	Model II	Model III
Floor Panel Size (mm)	1000 x 1000	1000 x 1000	1000 x 1000
Slab Thickness (mm)	40	40	40
Column Height (mm) at GS	660	490	660
Column Height (mm) at FS	660	660	660
Column size (mm) at GS	75 x 75	75 x 75	100 x 100
Column size (mm)at FS	75 x 75	75 x 75	75 x 75
Beam size (mm)	60 x 90	60 x 90	60 x 90

GS: Ground Storey; FS: First Storey

Table 2:Geometrical Properties

2.3 Compensating Mass for Scale Test Models

The case of scale test model, the geometry of the structure is decided in accordance to the scale factor. However, the mass of the prototype structure does not get proportionately reduced as per similitude requirement. Hence, additional masses need to be added to maintain requirement of scaling relationship for gravity load similitude as mentioned in Table 1.The mass of the prototype building have been determined as 75088 kg considering unit weight of concrete as 2435 kg/m3. The total mass of the test model is 600 kg. The mass required for fulfilling similitude requirement is3000 kg. Hence, an additional mass of 2400 (=3000-600) kg is required to be uniformly distributed over two floors of the model.

Particulars	Model 1	Model 2	Model 3
Concrete			
Density (Kg/m^3)	2435	2435	2435
Compressive Strength(N/mm^2)	15	15	15
Reinforcing Steel : Longitudinal Bars			
Type of Steel	Fe 250	Fe 250	Fe 250
Diameter of bars (mm)	6	6	6
Number of bars	4	4	4
Young’s Modulus (N/mm^2)	2.00E+005	2.00E+005	2.00E+005
Reinforcing Steel : Stirrups			
Spacing of stirrups (mm)	45	45	45
Diameter of bars (mm)	3	3	3
Clear cover			
Beams (mm)	15	15	15
First storey columns (mm)	15	15	15
Ground storey columns (mm)	15	15	20

Table 3: Material Properties

2.4 Details of Ground Motion

Two different earthquake ground motions have been considered for subjecting excitation to the analytical test models. Earthquake records with different peak accelerations, displacements and durations have been selected for use in analytical experiment. These were recorded during El Centro (1940): Comp – 180 & Victoria (1980): Comp - CPE045 earthquakes. The descriptions of relevant characteristics of the earthquake records are presented in Table 4. For nonlinear identification, spectrum compatible time histories are used.

Earthquake components	Peak ground acceleration (g)	Frequency Range (rad/sec)
El Centro (1940): Comp - 180	0.32	0-65
Victoria (1980): Comp - CPE045	0.62	0-160

Table 4: Characteristics of the selected Earthquake records.

2.5 Scaled time Histories

As mentioned in the Section-2.1, the sampling rate of earthquake records used for shake table excitation need to be scaled through a factor $1/\sqrt{S}$, where S denotes scale factor. The

Table 5 represents original and scaled sampling rate of earthquake records used for Time History Analysis.

Earthquake components	Original Time Interval (Sec)	Scaled Time Interval (Sec)
El Centro (1940): Comp - 180	0.02	0.008944
Victoria (1980): Comp - CPE045	0.01	0.004472

Table 5: Time interval of earthquake records used for Time History analysis.

3 MODELLING CONSIDERATION

3.1 The Analytical Model

The System for Earthquake Engineering Simulation (OpenSees) software Mazzoni et al. [12]0 was used to make an analytical model of the structure for performing nonlinear dynamic analysis. The numerical model is shown in Figure 1.

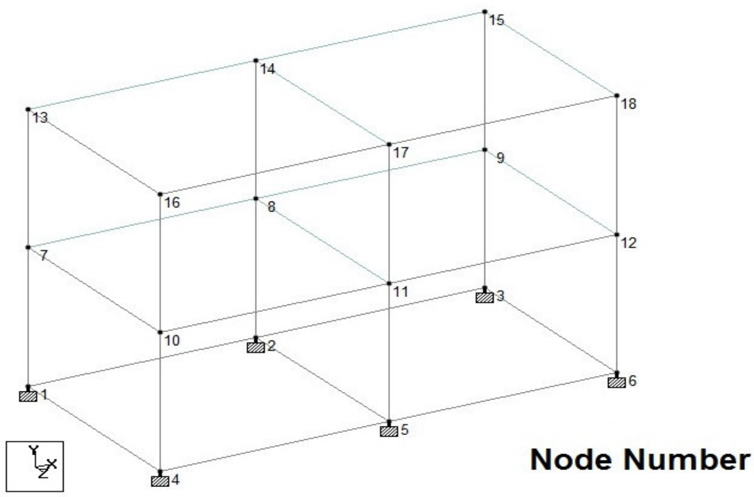

Figure 1: The Analytical Model

All the 6 numbers of support nodes are constrained in all DOF i.e. fixed support. Concentrated plasticity with elastic interior (beam with hinges) elements has been considered for beam and column elements. Plastic hinge length for column considered as 0.113m from beam column joint and for beam it is 0.130m.

3.2 Element Section Type

Fiber Section has been considered. Reinforcement detail of column section has been shown in Figure 2.

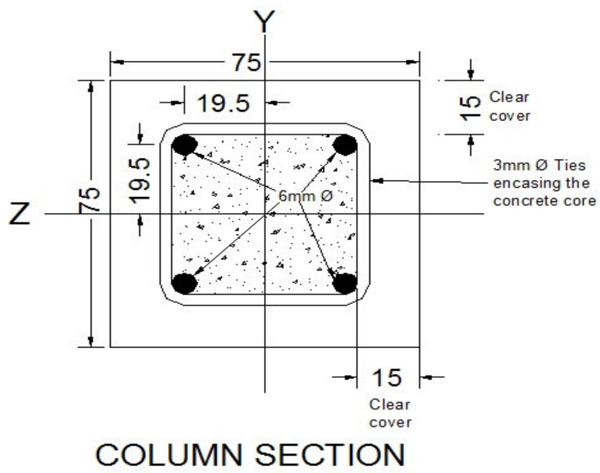

Figure 2: Typical Reinforcement Details of Column Section.

3.3 Material behavior for concrete or the concrete model for concrete fibers.

OpenSees " Concrete02" material –Linear Tension softening is used here. This is an uniaxial concrete material object with tensile strength and linear tension softening. It uses mander model Mander et al. [13]0 for confined inner core strength and unconfined strength for cover.

3.4 Material behaviour for Steel or the Steel model for Reinforcement fibers.

OpenSees "Steel02" material, Giuffre-Menegotto-Pinto Model with Isotropic Strain Hardening has been considered. This is an uni-axial Steel material object. Isotropic hardening has been ignored in the present case.

4 COMPARATIVE STUDY OF THE CHANGE OF MODAL PARAMETERS OF PROGRESSIVELY DETERIOTING STRUCTURE.

4.1 RCC Prototype Model-1

Total of 15 data window taken with each data window containing 100 points i.e. 15x100=1500 time step x 0.00447=9.387sec of Victoria (1980): Comp - CPE045 (Spectrum Compatible) time-history for lab-scaled analytical model no-1. The scale factor for amplitude of excitation was selected such that the structure is taken into nonlinear range. The scale factor selected was 0.204g. It is expected that the modal frequencies of the structure should degrade as the behavior of the structure changes from linear to nonlinear. This is evident from the results as obtained in Table 6. The results of System Identification using N4SID, SNLSE and exact analysis by OpenSees software are presented in Figure 3.

Data window	Time (sec)		Frequency (Rad/Sec)					
			N4SID		SNLSE		OpenSees (Exact)	
	From	To	Mode-I	Mode-2	Mode-I	Mode-2	Mode-I	Mode-2
1	0.00	0.45	54.92	134.59	53.69	143.34	44.62	117.54
2	0.45	0.89	54.60	120.24	43.20	113.71	42.35	108.48
3	0.90	1.34	42.63	101.77	46.00	118.34	43.58	113.70
4	1.35	1.79	45.54	109.61	44.76	114.67	43.96	115.43
5	1.79	2.24	40.61	98.23	44.74	114.51	39.67	104.36
6	2.24	2.68	39.54	100.37	42.84	110.03	36.86	100.16
7	2.69	3.13	45.21	107.61	40.26	104.47	37.00	102.26
8	3.13	3.58	42.29	105.07	39.54	102.66	35.87	95.59
9	3.58	4.02	41.31	106.47	38.62	101.05	36.86	95.92
10	4.03	4.47	40.63	113.68	38.36	100.46	36.23	97.35
11	4.47	4.92	39.22	102.23	37.95	99.62	37.01	97.86
12	4.92	5.36	38.28	105.07	37.85	99.28	35.70	99.33
13	5.37	5.81	39.23	110.46	37.77	99.03	36.06	100.07
14	5.82	6.26	40.37	105.67	37.72	98.84	37.01	96.76
15	6.26	6.71	42.96	105.07	37.68	98.70	37.47	96.53
16	6.71	7.15	43.76	104.50	37.66	98.65	38.22	97.27
17	7.16	7.60	41.43	97.85	37.65	98.61	37.73	98.20
18	7.60	8.05	38.78	88.29	37.64	98.56	37.95	96.84
19	8.05	8.49	42.47	96.93	37.63	98.50	38.03	96.77
20	8.50	8.94	40.61	108.05	37.62	98.48	38.22	97.06
21	8.94	9.39	40.47	101.23	37.62	98.47	38.20	97.16
22	9.39	9.83	39.77	92.24	37.62	98.46	37.85	97.01
23	9.84	10.28	41.09	95.45	37.62	98.46	38.59	97.51
24	10.29	10.73	37.77	79.12	37.62	98.44	38.40	97.17

Table 6: Comparative study of N4SID, SNLSE output with that of exact modal parameters given by OpenSees software (base excitation scale factor 2). (Model-1)

Figure 4 shows the variation of natural frequencies of first and second mode of vibration of Model-1 subjected to Victoria (1980): Comp - CPE045 (Spectrum Compatible) time-history but with magnified base excitation factor=4. To study the performance of SNLSE algorithm in comparison to short time data window technique of N4SID, all the three models were also subjected to a different base excitation El Centro (1940): Comp – 180 (Spectrum compatible) earthquakes and the results obtained are shown in Figure 5.

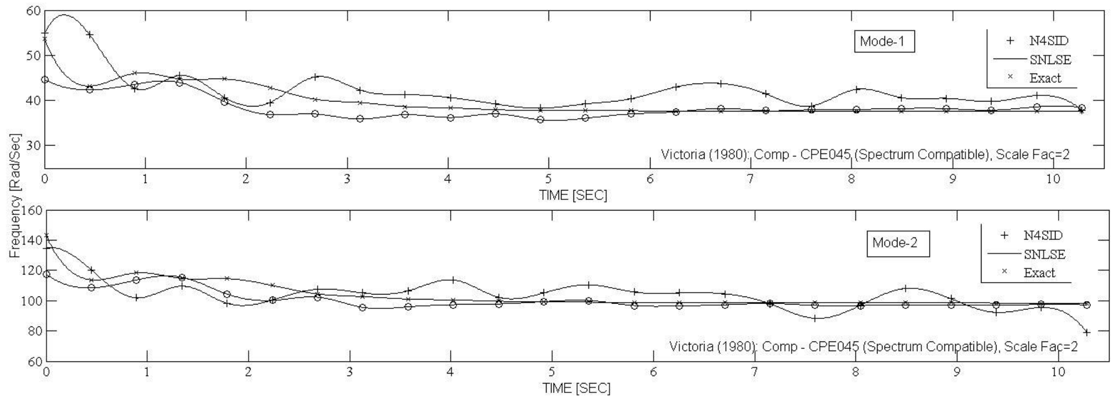

Figure 3: Instantaneous natural frequencies of first two vibration modes of prototype RCC model-1 with scale factor for base excitation=2 of Victoria(1980) Comp-CPE045(Spectrum Compatible), identified using N4SID, SNLSE.

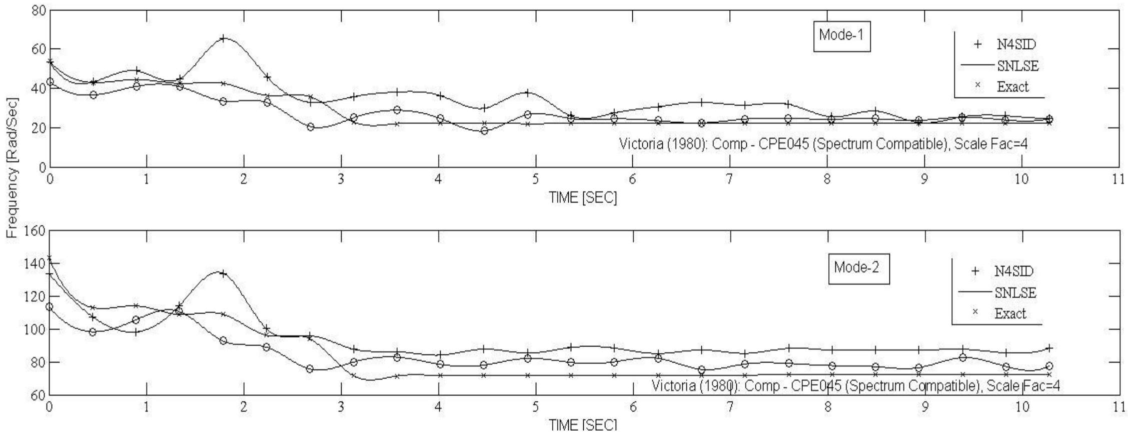

Figure 4: Instantaneous natural frequencies of first two vibration modes of prototype RCC model-1 with scale factor for base excitation=2 of Victoria(1980) Comp-CPE045(Spectrum Compatible), identified using N4SID, SNLSE

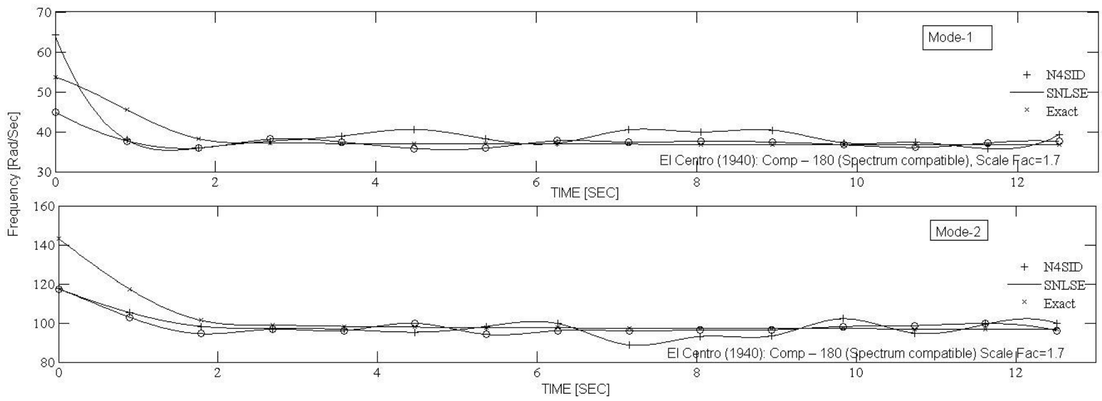

Figure 5: Instantaneous natural frequencies of first two vibration modes of prototype RCC model-1 with scale factor for base excitation=1.7 El Centro (1940): Comp – 180 (Spectrum compatible), identified using N4SID, SNLSE

4.2 RCC Prototype Model-2

Similar to Model-1, Model 2 is also subjected to Victoria (1980): Comp - CPE045 (Spectrum Compatible) with Scale Factor=2 i.e.0.204g and also El Centro (1940): Comp – 180 (Spectrum compatible) Scale Factor=1.7, earthquakes and the results obtained are shown in Figure 6 and Figure 7 respectively.

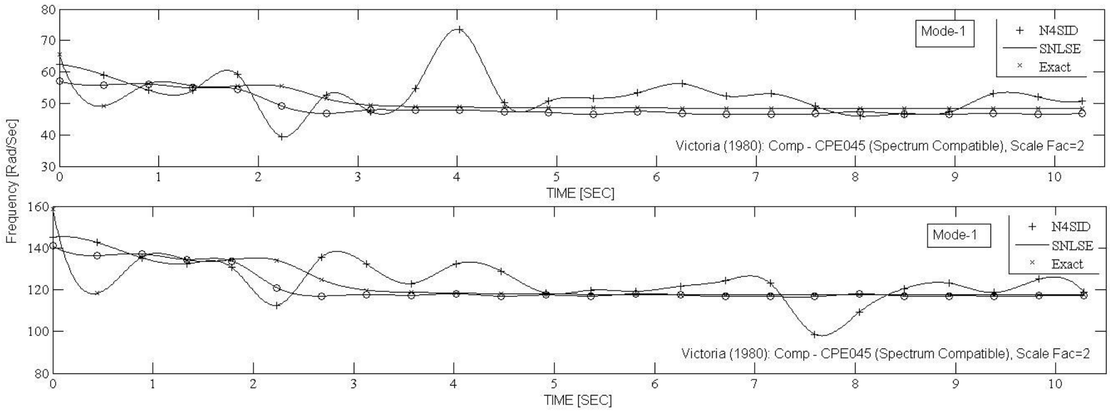

Figure 6: Instantaneous natural frequencies of first two vibration modes of prototype RCC model-2 with scale factor for base excitation=2 of Victoria(1980) Comp-CPE045(Spectrum Compatible), identified using N4SID, SNLSE

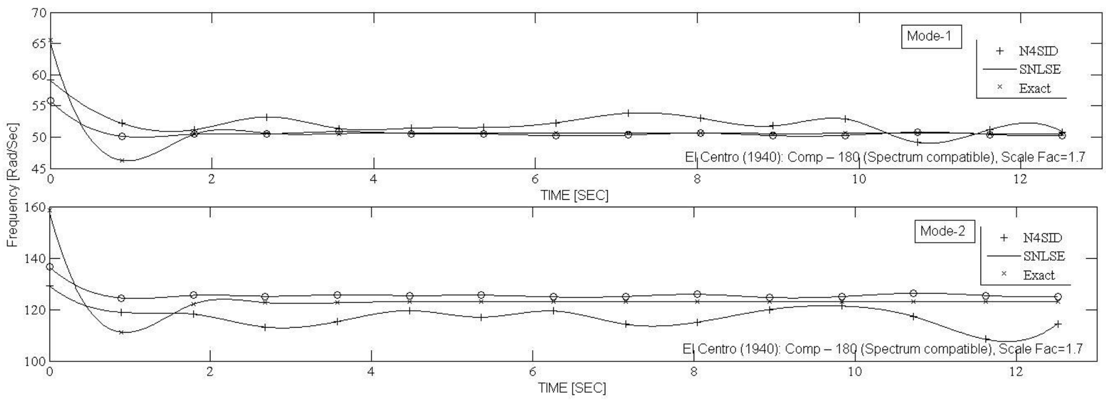

Figure 7: Instantaneous natural frequencies of first two vibration modes of prototype RCC model-2 with scale factor for base excitation=1.7 El Centro (1940): Comp – 180 (Spectrum compatible), identified using N4SID, SNLSE.

4.3 RCC Prototype Model-3

Model-3 is also treated in the same way as model-2 and the results as obtained are shown in Figure 8 & Figure 9.

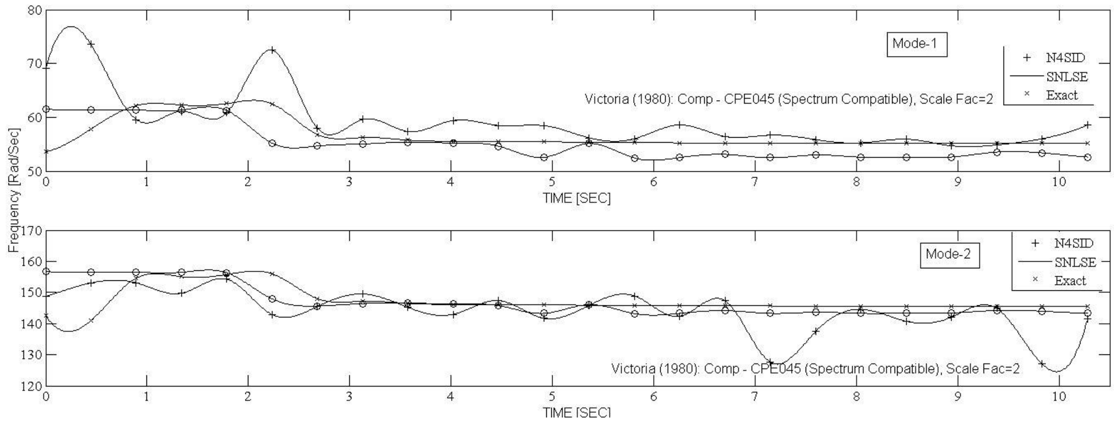

Figure 8: Instantaneous natural frequencies of first two vibration modes of prototype RCC model-3 with scale factor for base excitation=2 of Victoria(1980) Comp-CPE045(Spectrum Compatible), identified using N4SID, SNLSE

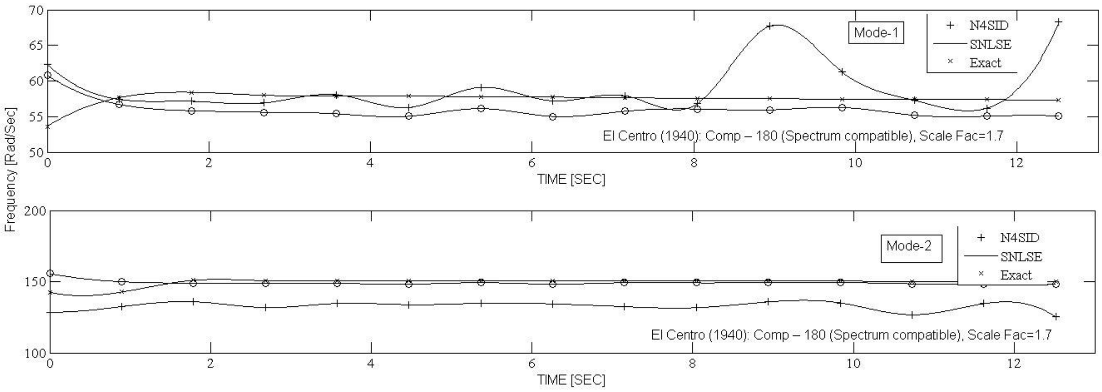

Figure 9: Instantaneous natural frequencies of first two vibration modes of prototype RCC model-3 with scale factor for base excitation=1.7 El Centro (1940): Comp – 180 (Spectrum compatible), identified using N4SID, SNLSE.

5 CONCLUSIONS

In this study, performance of the deterministic-stochastic subspace identification (N4SID) method for instantaneous modal analysis of nonlinear dynamic systems is evaluated based on numerical data. In the numerical study, system identification results of nonlinear MDOF with 2-DOF systems obtained from N4SID are compared to those from Sequential Nonlinear least square estimate (SNLSE) method and the exact values from finite element analysis using "OpenSees" software. Accuracy of system identification results are investigated due to variability of two input factors: input excitation, level of input excitation and structural geometry used in the identification.

- The response nonlinearity and its intensity can be tracked through identified instantaneous natural frequencies as well as SNLSE technique.
- Modal parameters obtained using SNLSE are consistently more accurate than those obtained using N4SID using short time data window.

- The results obtained from N4SID are more sensitive to type of structure; deviation from exact value depends from structure to structure.
- Instantaneous damage tracking using SNLSE is more accurate than N4SID.

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