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Publication Date

2006-04-18

Peer reviewed

FORCED VIBRATION TESTING OF A FOUR STORY RC BUILDING UTILIZING THE NEES@UCLA MOBILE FIELD LABORATORY

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ABSTRACT

The *nees*@UCLA mobile field laboratory was utilized to collect data from forced and ambient vibration testing of a four-story RC building damaged in the 1994 Northridge earthquake. Both low amplitude broadband and moderate amplitude harmonic excitation were applied using a linear shaker and two eccentric mass shakers, respectively, and ambient vibrations were measured before and after each forced vibration test. Accelerations, interstory displacements, and curvature distributions were monitored using accelerometers, LVDTs and concrete strain gauges. Natural frequencies and the associated mode shapes for the first 7 modes were identified. Fundamental frequencies determined from the eccentric mass shaker tests were 70% to 75% of the values determined using ambient vibration data, and 92% to 93% of the values determined using the linear shaker test data. Larger frequency drops were observed in the NS direction of the building, apparently due to damage that was induced during the Northridge earthquake.

Introduction

Field testing of full-scale structures using mechanical shakers plays an important role determining the dynamic properties of structures to improve our ability to model structural systems subjected to earthquakes. Field performance data represents the "ground truth" information against which all analysis procedures, code provisions, and other tests results are calibrated. However, several factors have limited the impact of field testing to date, including (1) the inability of artificial (forced) vibration sources to impart moderate-to-large amplitude vibrations on test structures, (2) the inability of traditional vibration sources to excite structures with inputs that represent realistic simulations of broadband seismic excitation, and (3) practical difficulties associated with deploying a sufficiently large sensor array to assess detailed component behavior.

The University of California, Los Angeles NEES Equipment Site (nees@UCLA) was established to address the aforementioned limitations. The nees@UCLA equipment portfolio includes: (i) large capacity eccentric mass shakers for harmonic excitation; (ii) a linear shaker for broadband realistic seismic structural simulation at low force levels; (iii) sensors for

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monitoring acceleration and/or displacement responses; (iv) wireless field dataloggers that efficiently transfer data from the test structure to the high performance mobile network; and (v) a high performance mobile network that receives and locally stores data at a mobile command center deployed near the test site, transmits and broadcasts selected data via satellite for teleobservation of experiments [Whang et al., 2004].

The advanced field testing capabilities of the nees@UCLA Site were recently deployed on a four-story reinforced concrete building in Sherman Oaks, California. The building suffered significant damage in the 1994 Northridge earthquake and is slated for demolition. Previous studies documented some of the building damage and included analytical studies to assess reasons for the observed damage. The results of these studies were inconclusive, in part due to the considerable uncertainty in modeling the structural system. Consequently, ambient and forced vibration testing was performed to collect field measurement data for evaluating the validity of prior analytical models as well as providing essential data for model calibration. Furthermore, detailed field test datasets of full-scale structures with significant levels of shaking are rare. Therefore, another objective of this study was to generate a truly unique dataset in terms of high-density spatial resolution that will be archived in the NEES data repository. This dataset could be useful for earthquake engineering researchers to improve understanding of the dynamic response of real buildings, and for damage detection studies. In the following, we first document the building condition and provide an overview of the nees@UCLA equipment, and then we describe the test program and present results.

Building and Damage Survey

The four story reinforced concrete office building, referred to as the Four Seasons Building is located in Sherman Oaks, California. The building was constructed in 1977 and the structural system consists of a perimeter moment resisting frame and an interior post-tensioned slab-column gravity frame with drop panels. Figure 1 shows a plan view of the building.
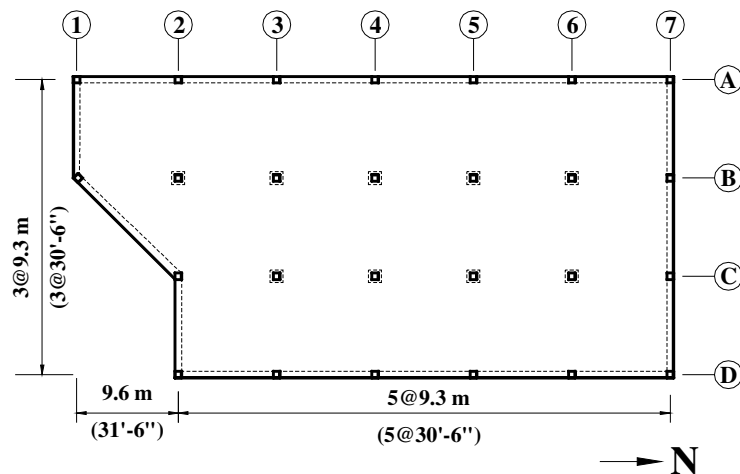

Figure 1. Typical plan of test building

Typical interior gravity frames consist of a 216 mm deep post-tensioned flat slab constructed of 21 MPa lightweight concrete and 610 mm square column with 1219 mm square 190 mm deep drop panel of 28 MPa normal weight concrete. Seven wire post-tensioning strands

having a minimum ultimate tensile strength of 1,862 MPa were used for the unbonded post-tensioned slab. Lateral seismic resistance is provided by a Special Moment Frame (SMF) located around the perimeter of the building constructed of 28 MPa normal weight concrete. The perimeter SMF beams are 610 mm wide by 760 mm deep, except for beams on the 2nd floor level which are 610 mm wide by 1097 mm deep. Typical perimeter columns have 610 mm square section reinforced with 8 #9 to 12 #10 bars and #4 hoops at 102 to 152 mm on center. Reinforcement with specified yield strength of 413.7 MPa is used for longitudinal and transverse reinforcement in all structural members. Additional details and detailed drawings of the structural system, connection details and material properties are provided in Yu [2005].

Previous studies

Several studies were conducted to document the building damage [Sabol, 1994] and to assess reasons for the damage [Aschheim and Moehle, 1995; Hueste and Wight, 1997]. Aschheim and Moehle performed linear and non-linear static pushover analyses using the 50% draft of ATC-33. Results obtained from their analyses were not consistent with the degree and distribution of the observed damage. They cited five possible reasons for the discrepancy: (1) ground motions at the site were more severe than those used; (2) properties of the analytical model differ significantly from those of the actual building; (3) torsional response of the building was dominant; (4) vertical response was significant; and (5) the tentative model used to assess slab-column punching was unconservative. Hueste and Wight [1997] performed both nonlinear static pushover analysis and nonlinear dynamic analyses using ground motions recorded near the building. Analysis results based on the design drawings did not indicate punching problems at the slab-column connections. However, inspection of one of the damaged connections revealed that reinforcement had not been placed as indicated on the drawings, which reduced the effective slab depth and resulted in greater potential for slab-column punching failures. Subsequent static and dynamic analysis results using revised punching capacities reduced due to as-built conditions indicated potential for punching failures at all 1st and 2nd floor slab-column connections.

In summary, although prior analytical studies indicated some potential for damage at the slab column connections, the degree and distribution of the observed damage were substantially greater than could be predicted using the analytical tools available. Therefore, the forced-vibration tests on the building provides a unique opportunity to collect data that could be used to improve and calibrate models that could be used for more detailed studies to assess reasons for building damage.

Damage survey

Detailed visual inspection of the building damage was performed prior to the forced-vibration testing to document the existing structural condition. Damage was particularly severe at slab-column connections due to slab punching shear failure around the perimeter of the drop panels. The most dramatic failures were observed at the south end of the 2nd floor level (column B2) and the north end of the 3rd floor level (column B6, Figure 2(a)). In additional to the slab punching failures, significant joint diagonal cracks, column flexural cracks, and minor concrete spalling at some girders adjacent to the beam-column joint were observed on the perimeter moment frame.

Significant diagonal cracks were observed within the beam-column joint regions (Figure 2(b)) along the east and west perimeter frames, primarily at the 3rd floor level between columns

A4 to A6 and columns D4 to D6. Concrete spalling was observed at the middle of the joint, and the diagonal cracks often extended up to the floor slab. Moderate joint cracks were observed at several locations in the east and west perimeter frame except at the 2nd floor level. No joint cracks were noted in the north and south perimeter frames except for column C7 at the roof level. The damage survey also indicated that the majority of the girders in both directions had minor to moderate concrete crushing at the beam - joint interface, and almost all of columns had flexural cracks distributed over the height on all four sides. Overall, the perimeter frame damage patterns indicate that the building experienced considerably more deformations in the N-S direction than the E-W direction.

<table>
<tr><td>(a)</td><td>(b)</td></tr>
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Figure 2.   Earthquake damages of the building, (a) Punching shear failure (Column B6 at the 3rd floor level), and (b) Shear crack on beam-column joint (Column A4 at the 3rd floor level)

nees@UCLA Equipment overview

Equipment, instrumentation, and data acquisition systems integrated into the Network for Earthquake Engineering Simulation (NEES) mobile field laboratory at UCLA (nees@UCLA) were deployed for forced-vibration testing. An overview of the nees@UCLA portfolio is provided in the following subsections. More detailed information can be obtained at the nees@ucla website (nees.ucla.du), or on the www.nees.org site specific database.

Two, large capacity, eccentric mass shakers (ANCO Model MK-15) and a modest-capacity linear shaker were anchored at the roof of the building as vibration sources. A single eccentric mass shaker develops uni-directional harmonic forces with two counter-rotating baskets, with a peak force output of 445 kN. The two baskets contain several compartments that can be filled with steel bricks so as to alter the amplitude of force by increasing the mass-eccentricity. In the present test, two different basket configurations were used as shown in Table 1. The eccentric mass shakers were positioned at the north and south ends of the roof to maximize the torsional excitation that could be produced, while also enabling translational excitation in the N-S and E-W directions.

The nees@UCLA linear shaker system can generate broadband excitations at low force levels to enable efficient assessment of modal properties using system identification techniques.

The linear shaker is driven by a hydraulic actuation system capable of moving a nominal weight of 2.25 ton with a peak force of 66.75 kN through a stroke of ± 38.1 cm. The linear shaker was mounted at a 45° angle form the N-S perimeter frames about 3.3 meters away from the approximate center of the roof (the reference point). This location and orientation were selected to impart approximately equivalent levels of excitation in N-S and E-W translations while also providing torsional excitation.

Table 1. Mass-eccentricity of each basket and limiting frequencies

	Empty basket	Half-full basket
Mass-eccentricity	16786 lb-in	56620 lb-in
Limiting frequency *	5.40 Hz	2.95 Hz

* The frequency at which the shaker achieves the maximum force amplitude for that mass-eccentricity

(a)   (b)

Figure 3. Field shakers, (a) Eccentric mass shaker and (b) linear shaker
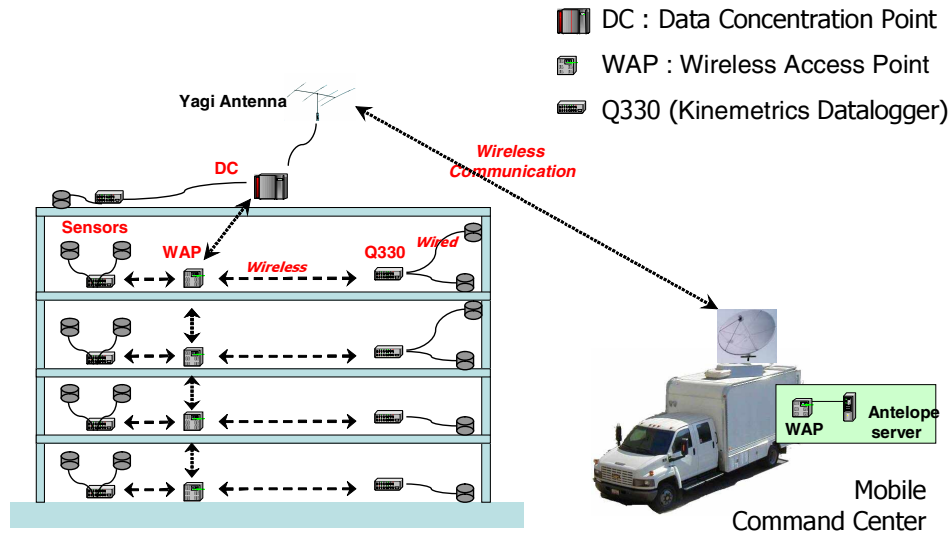

Figure 4. A schematic drawing of the Kinemetrics DAQ system

A dense array of sensors was deployed prior to the test. A total of 16 triaxial and 27 uniaxial force-balance accelerometers (Kinemetrics ES-T and ES-U, respectively) were mounted on floor slabs, 26 linear variable displacement transducers (Trans-Tek, Series 240 DC LVDT) were mounted on beams and between floor slabs, and 96 strain gauges (TML, PL-60-11-5L) were affixed to slab and column surfaces to allow determination of accelerations, interstory drift, and beam, slab, and column curvature distributions. The data acquisition was performed using

two separate systems: a Kinemetrics/Antelop system and a National Instrument/Labview system. Data from both systems were GPS time-stamped to ensure data compatibility. Figure 4 shows a schematic of the Kinemetrics data acquisition system and data flow through the system.

Test Program

The test program consisted of a series of forced vibration tests using the linear shaker and eccentric mass shakers, respectively, as well as collection of ambient vibration data. The linear shaker was used to generate broadband excitations which are useful to assess modal properties of the building from acceleration measurements by time domain system identification techniques. During relatively high-amplitude sinusoidal vibrations obtained by synchronized operation of two large capacity eccentric mass shakers (EMS), interstory displacements were obtained from both displacement measurements by LVDTs and the differences between story accelerations divided by the square of circular frequency. Curvature distributions of slabs and columns during the EMS tests were investigated using the measured concrete strains. Ambient vibrations were measured before and after each test to investigate potential drifts of modal properties resulting from the forced vibrations.

Linear shaker and ambient vibration tests

A whitenoise-type broadband excitation was applied to the building using the linear shaker installed at the roof level. The whitenoise-type input function was obtained by bandpass filtering (fifth-order Butterworth filter) random numbers to cut out the frequency content below 0.5 Hz and above 10 Hz. The acceleration of the moving mass of the linear shaker was measured by a uniaxial accelerometer to estimate the input force to the structure. The acceleration responses measured at each floor were transformed to three components of story accelerations (i.e. N-S, E-W translational and torsional components) with respect to a reference point, which is chosen as the midway point between column B4 and C4. The natural frequencies and the damping ratios from each dataset were obtained by system identification analyses, described later in this paper.

Eccentric mass shaker tests

In EMS tests, the two MK-15 eccentric mass shakers were synchronized to provide N-S, E-W, and torsional excitation at a total of 153 frequency steps. Tests were performed at two force amplitudes (i.e., empty and half-full baskets). Each of the tests began by ramping up the shaker rotation to the lowest target frequency of 0.5 Hz, and gradually increasing the target frequency until the limiting frequency at which the maximum force capacity of the shakers was achieved. The testing duration at each target frequency was maintained for a minimum of 60 seconds to obtain the steady state response. Before proceeding to data interpretation, acceleration measurements at the ground floor were investigated to assess the importance of rocking motion of the building associated with soil-structure interaction. Distributions of vertical acceleration revealed that rocking motion was not significant [Yu, 2005], which is consistent with expectations for this relatively flexible moment frame building supported on caissons [Stewart et al, 1999]. Thus, the effect of rocking motion was ignored in the analysis of test data.

Normalized displacement curves were constructed from the peak values of measured accelerations divided by the square of circular frequencies and the input force for each test

frequency. The input forcing function from two shakers was estimated from the mass-eccentricity of baskets and the recorded pulse marker signals. The three components of story acceleration at the reference point of each floor were calculated from the measured accelerations. Computed story accelerations for each frequency step were filtered with a narrow bandpass filter (fifth order Butterworth filter with cutoff frequencies of 50% and 150% of the excitation frequency) to obtain a single vibration frequency. The peak amplitudes of the story accelerations were determined from the steady state part of the filtered signal. The amplitude and phase spectra of normalized displacement curves obtained from the EMS tests (empty basket) and the transfer function curves from the linear test are shown in Figure 5. No significant discrepancy in fundamental frequencies was observed between the empty basket test and half-full basket test.

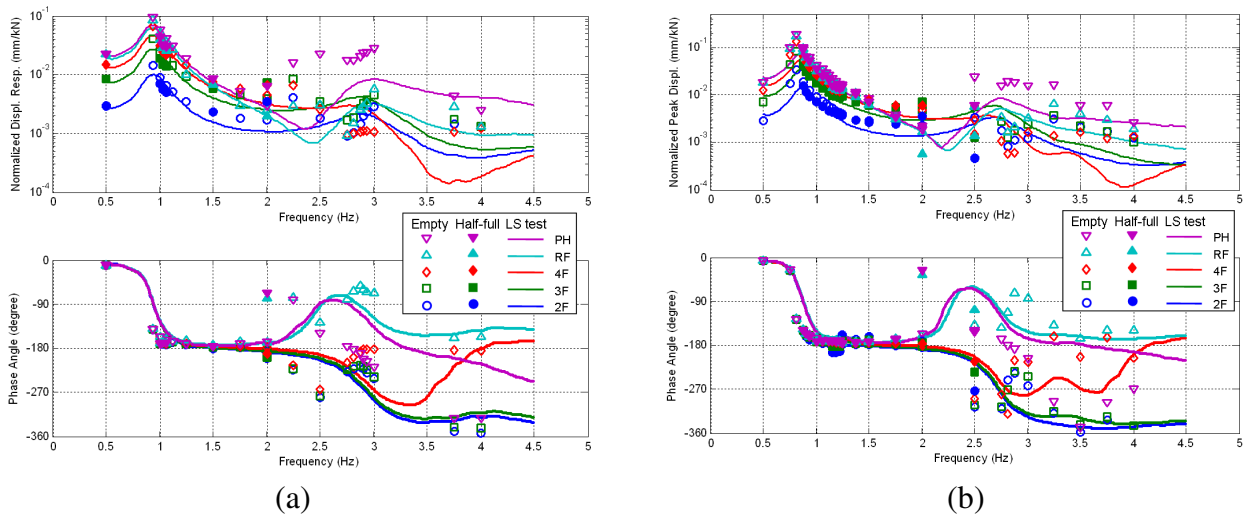(a)   (b)

Figure 5 Amplitude and phase plot of normalized displacement curves by EMS test and transfer function from the linear shaker test. (a) N-S and (b) and E-W Translation

System identification

Natural frequencies and damping ratios identified by system identification analyses are presented in Table 2 and Table 3. For the linear shaker and ambient vibration data, a time-domain system identification method, Numerical Algorithm for Subspace State Space System Identification (N4SID) was adopted. The fundamental frequencies for the EMS test were estimated by fitting the normalized displacement curves to SDOF transfer function curves. The curve-fitting was performed using the normalized displacement points around -90° phase angle to find fundamental frequencies in each direction.

The first six or seven modes could be identified by linear shaker test and ambient vibration test data. The frequencies identified with the data from the EMS tests and ambient vibrations were, on average, 8% lower and 14 to 35% higher than those obtained using the linear shaker test data, respectively. Since there was no change in building mass during the tests, these differences are attributed to changes in stiffness properties either due to the contribution of nonstructural elements and/or stiffness degradation of structural members. The contribution of non-structural elements was thought to be relatively minor since most of the drywall partitions were already separated from neighboring structural members due to earthquake damage, and no in-filled or exterior brick veneer walls existed. Consequently, stiffness degradation of the

structural members may be the more likely cause of the frequency drop.

Table 2. Natural frequencies from system identification

Identified Natural Frequency (Hz)

	1st	2nd	3rd	4th	5th	6th	7th
Ambient 0702,(a)	1.09	1.25	1.55	3.23	3.63	4.16	5.38
EMS, (b)	0.81	0.87	1.1~1.2				
Ambient 0719, (c)	1.06	1.20	1.50	3.11	3.51	3.99	
LS 0728, (d)	0.87	0.94	1.25	2.73	2.91	3.43	
LS 0802, (e)	0.88	0.94	1.26	2.73	2.94	3.44	4.54
Ambient 0803 (f)	1.06	1.21	1.49	3.11	3.48	3.96	
Normalized to LS 0802, (e)							
(a)/(e)	124%	133%	123%	118%	124%	121%	119%
(b)/(e)	92%	93%	87% ~ 95%				
(c)/(e)	120%	128%	119%	114%	119%	116%	
(d)/(e)	99%	100%	100%	100%	99%	100%	
(f)/(e)	120%	129%	119%	114%	118%	115%	
Normalized to the 1st frequencies							
Ambient 0702,(a)	100%	115%	142%	295%	332%	380%	491%
EMS, (b)	100%	108%	135% ~ 148%				
LS 0802, (e)	100%	107%	143%	310%	334%	391%	516%

Previous researchers have observed the dependence of the natural frequencies on the amplitude of vibration. Foutch [1978] reported that the fundamental frequencies decreased from 5 to 7 % based on an increase of 15 fold in the excitation force. Trifunac [1972] also found that the natural frequencies obtained by forced vibration tests were about 4% lower that those by the ambient vibration. In the current test, the peak values of the story accelerations around the fundamental frequencies, obtained by filtering them with a very narrow bandpass filter (fifth order Butterworth filter with cutoff frequencies of 95% and 105% of the fundamental frequencies in each test), are 5×10^{-5}g, 0.002g to 0.0015g, and 0.007g to 0.009g for the ambient, linear shaker tests, and EMS tests, respectively. Figure 6 shows the relationship between the two quantities. The decrease in fundamental frequencies appears to be proportional to the exponential of the (filtered) vibration amplitude increase.

Table 2 also shows the ratios of identified natural frequencies to the lowest fundamental frequency. Frequency ratios for the forced vibration and ambient vibration tests follow different trends; the 1st N-S frequency is 7 % to 8 % higher than the 1st E-W frequency for the forced-vibration tests (EMS test and linear shaker tests), and 13% to 14% higher for the ambient vibration data. Again, considering that the mass of the building is relatively fixed, the change in the frequency indicates that stiffness degradation occurs for larger amplitude vibrations, and is more severe in the structural members contributing to stiffness in the N-S direction. One possible reason for the noted discrepancy in frequency ratios could be the pre-existing condition of the building, which was damaged in the Northridge earthquake. With the increase of vibration amplitude, the degradation of the (secant) stiffness tends to be larger in members with more residual inelastic deformations.
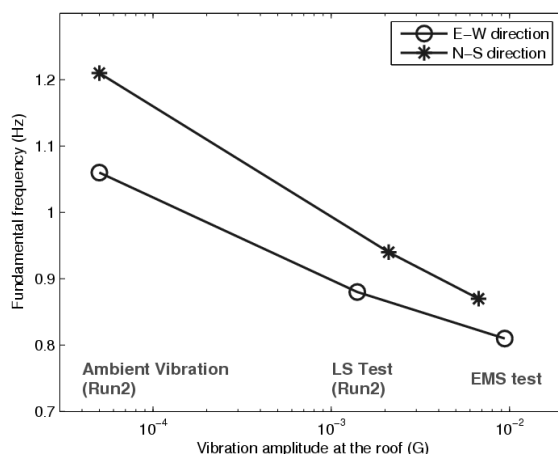
Figure 6. Change of natural frequencies with vibration amplitude
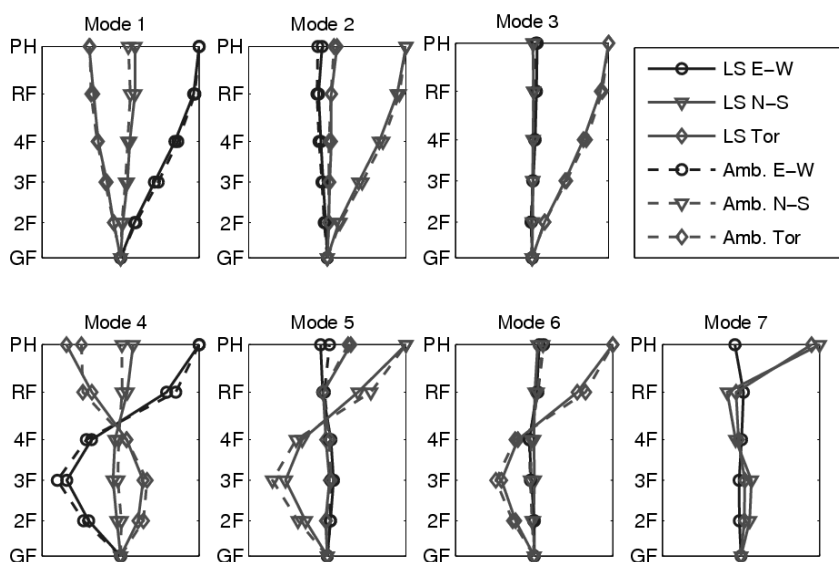

Figure 7. Mode shapes (Linear Shaker test Run2 and Ambient vibration Run2)

As shown in row (a) and (f) of Table 2, identified frequencies for ambient vibration measurements collected after the half-full basket EMS test showed a 3% to 4% reduction in frequencies relatively to the ambient vibration results identified prior to the test. Permanent stiffness reduction in structural members, exterior cladding at typical stories or foundation/soil supporting the building may be the possible reason. Further research is needed to clarify this phenomenon. Damping ratios were obtained as approximately 5 to 8% from linear shaker tests, and as 2 to 4% from ambient vibrations. Mode shapes determined by linear shaker test (run2) and ambient vibration test (run2) are shown in Figure 7. The first six mode shapes correspond to the first and second modes in the order of transverse (E-W), longitudinal (N-S), and torsional direction, respectively. The mode shapes do not change as much with the type of testing (i.e. vibration amplitude) as do the natural frequencies. Curvature distributions of slabs and columns obtained from strain measurements were studied, however, are not presented here due to space limitations.

Conclusions

Data were collected from a series of forced vibration tests and from ambient vibrations of a four-story reinforced concrete building damaged by the Northridge earthquake using the nees@UCLA mobile field laboratory. Two eccentric mass shakers and a linear inertia shaker were used as vibration sources. Global frequency response of the test building and detailed behavior of structural components were monitored with a dense instrumentation array using 75 accelerometer channels, 26 displacement transducers, and 96 concrete strain gauges. From the acceleration measurement, drift of modal properties of the building was investigated using system identification techniques. Approximately the first 6 frequencies and mode shapes for the building could be identified using the test results. From the ambient vibration data collected before and after the eccentric mass shaker test, a drop of about 3% in the natural frequencies was observed. Fundamental frequencies during eccentric mass shaker test were 70% to 75% of those by ambient vibration data, and 92% to 93% of those by linear shaker test. Collected vibration data and modal properties can form a basis of the analytical model predicting the response of the building. Future studies will involve assessing reasons and the degree of building damage that resulted from the Northridge earthquake using site specific ground motions and nonlinear modeling approaches.

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