Recent Advances in Wave Travel Time Based Methodology for Structural Health Monitoring and Early Earthquake Damage Detection in Buildings

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SUMMARY:

Recent advances in the development of a wave travel time methodology for earthquake damage detection in buildings, for use in rapid assessment of structural health following an earthquake, are reviewed. Its main advantages over the modal methods are the insensitivity to the effects of soil-structure interaction, local nature, and robustness when applied to real structures and strong earthquake response. Three algorithms are reviewed, which identify wave velocity profiles of vertically propagating shear and torsional waves through the building by fitting a layered shear beam/torsional shaft model in observed building earthquake response, and a selection of results of their application to three buildings: Los Angeles 54-story office building (steel), Millikan Library in Pasadena (RC), and Sheman Oaks 12-story office building (RC), damaged by the San Fernando earthquake of 1971. The appropriateness of the model for different types of buildings, and the accuracy of the identification are discussed.

Keywords: earthquake damage detection, structural health monitoring, seismic interferometry, wave travel time

1. INTRODUCTION

The ability to monitor the health of an instrumented structure based on information obtained from the recorded response, detect damage as it occurs, and issue an early warning during or soon after the earthquake (or some other natural or man made disaster), and before physical inspection is possible, has significant potential benefits in reducing loss of life and injuries, in emergency response, and in recovery following the disaster. For example, a timely decision can be made to evacuate an unsafe building, reducing the risk of loss of life and injuries caused by potential collapse of the weakened structure during shaking from aftershocks. The timeliness of such information, even when the damage is obvious or there is no structural damage, is very useful to a building owner, as physical inspection takes time after a disaster, when the demand for inspection is large and the supply of inspection teams is limited. Such monitoring, in principle at least, can detect even nonvisible damage due to an extreme event or structural decay. These potential benefits have been recognized since the beginning of deployment of strong motion instruments in structures in the 1930s, and the development of system identification methods has been pursued by engineers for many decades (Chang et al. 2003). To be practically useful, a structural health monitoring method must be *robust* when applied to real data, sensitive even to smaller and local damage, and accurate. It should neither miss significant damage nor suggest false alarms causing needless evacuation and costly service interruptions. This is particularly important for critical facilities, e.g. hospitals. Satisfying all these requirements has been a challenge. The majority of vibrational methods found in structural health monitoring literature are not robust when applied to real structures and large amplitude response data. Further, the damage sensitive parameters vary due to causes other than damage, e.g. amplitude dependency, nonlinear soil response (via the mechanism of soil-structure interaction), which is the case for the natural frequencies of vibration, and environmental effects (e.g. temperature). The mode shapes are less sensitive to such changes but are more uncertain to estimate (see review in Chang et al. 2003).

This paper reviews the progress, since 14WCEE (Todorovska and Trifunac 2008c), in the development of a wave propagation structural health monitoring method by the authors. The method uses data from vibrational sensors, and is based on 1D wave propagation model of the building, represented as a layered shear beam. It identifies the distribution of shear and torsional wave velocities along the building height, which are directly related to the structural rigidity, and serve as damage sensitive parameters. Their changes during an earthquake are monitored by comparison with values estimated during an initial time window when the response is small. Instead of matching the natural frequencies of vibration, as do most of the system identification methods (Chang et al. 2003). including robust methods (Naeim et al. 2006), which are affected by soil-structure interaction and are sensitive to changes in the soil, this method matches pulses in the impulse response functions, whose shift in time is not affected by soil-structure interaction. This method can be viewed as an intermediate scale method, bridging the gap between the NDT and global vibrational methods. Important advantages of this wave method over the other vibrational methods, which are mostly modal (based on detecting changes in the natural frequencies and mode shapes), are its robustness, insensitivity to the effects of soil-structure interaction, and local nature, achieved with relatively small number of sensors. Applications to three damaged buildings have shown that the identified changes are consistent with the magnitude and spatial distribution of the observed damage (Todorovska and Trifunac 2008ab, Rahmani et al. 2012). The insensitivity to the effects of soil-structure interaction has been demonstrated on analytical models considering foundation horizontal motion (Snieder and Safak 2006), as well as coupled foundation horizontal and rocking motion (Todorovska 2009a). The developments reviewed are three fitting algorithms (one direct and two iterative), and detailed comparative analysis of their spatial resolution and accuracy, in particular for identification using data from a dense array. Other developments include findings on the capabilities and limitations of this method, based on detailed application to different types of buildings (Rahmani and Todorovska 2012bc, Rahmani et al, 2012).

2. METHODOLOGY

The building is modeled as a layered shear beam (Fig. 1a), described by the floor heights h, shear modulus μ and mass density ρ and shear wave velocity $V = \sqrt{\mu/\rho}$, which serves as the damage sensitive parameter. For horizontal base excitation, the building model is mathematically identical to that of a horizontally layered half-space representing near surface geology (Fig. 1b). Nevertheless, geophysical identification methods are not directly applicable to buildings due to physical differences. One is that buildings are slender structures, with foundation rocking DOF, and soil-structure interaction via foundation rocking, and another one is that they also deform in bending, to a degree depending on the structure. The former implies that the structural damping cannot be identified by impulse response analysis (Todorovska 2009a, Todorovska and Rahmani 2012), and the latter implies some degree of dispersive wave propagation, which is not captured by the model. Another difference is the demand for high accuracy in application to structural health monitoring.



Fig. 1 The model. a) Layered shear-beam representing a building. b) Layered half-space.

We developed three fitting algorithms of model impulse responses (computed from the propagator in the time domain; Haskell 1953, Gilbert and Backus 1966, Trampert et al. 1993, Todorovska and Rahmani 2012) in observed impulse responses (computed from regularized transfer-function; Snieder and Şafak 2006). The impulse response functions are computed for virtual source at roof, which are much simpler than those for virtual source at base, because reflections from the roof are suppressed, and there is no constructive interference of direct and reflected waves, either from the external or from the internal boundaries. Herein, the three algorithms are summarized briefly; for details see Todorovska and Rahmani (2012) and Rahmani and Todorovska (2012a). The identification method belongs to the class of seismic interferometry or as virtual source methods (Snieder at al. 2006).

Direct Algorithm: It computes the wave velocity directly from the time shifts of the direct pulses, as $V = h/\tau$, where τ is the difference between pulse arrival time at the top and bottom of the layer. It is based on the ray theory interpretation of the impulse response functions, according to which the travel time from base to roof is sum of the travel time through the different layers.

Nonlinear LSQ fit Algorithm: It fits model impulse response functions as waveforms, within time windows containing the main lobes of the direct pulses, by minimizing in the least squares sense, the sum of the error between the model and observed impulse responses at all levels where motion was recorded. The Levenberg-Marquardt method is used, which is a fixed regressor and small residual nonlinear model (Seberg and Wild 2003). This means that the parameters are assumed not to change in the time window analyzed, and that it requires close initial values to insure convergence. We use as initial values the parameters identified by the direct algorithm. This wave propagation algorithm is not limited by the assumptions of ray theory, such as smoothness in the variation of material properties, and accounts for pulse deformation. It also uses information from the pulse amplitudes which are primarily governed by the impedance contrast (i.e. distribution of wave velocities).

Time Shift Matching Algorithm: It is similar to the direct algorithm, and based on ray theory, but is iterative and gives more accurate results. The wave velocities are determined recursively from top to bottom by iteratively matching the time shifts of the transmitted pulses.

RESULTS

We critically examined the identification method and three algorithms, from point of view of their appropriateness and limitations in application to buildings, accuracy, and sensitivity to light damage. Figures 2-12 show selected results for three buildings: (1) Los Angeles 54-story steel office building (NS, EW and torsional responses) excited by Northridge earthquake of 1994 (M_1 =6.4, R=32 km), (2) Millikan Library 9-story RC building in Pasadena, California (NS, EW and torsional responses) excited by Yorba Linda, earthquake of 2002 (M=4.8, R=40 km), and Bank of California building (NS and EW response) excited by San Fernando earthquake of 1971 (M=6.6, R=27 km). Only the third building was damaged by the earthquake (Blume and Assoc. 1973). Figs 2, 6 and 11 show photos of the buildings, and sketches of vertical cross-sections and floor layouts, in which the location of the sensors is marked. Figs 3 and 7 show results of the identification for the first two buildings, in tabular form for the fit of an equivalent uniform shear beam, and, in graphical form, for the more detailed multi-layer models. Figs 4 and 9 show the agreement of the model and observed impulse responses, which demonstrates the goodness of the fit. Figs 5 (left) and 8 (right) compare the model (fixed-base) and observed (system) transfer-functions for the same two buildings. Disagreement is expected for the frequency of the first mode, which is affected by soil-structure interaction, while agreement for the higher modes demonstrates the goodness of the model for the particular structure. In Fig. 5 (right), the model fixed-base mode shapes are also plotted for the first five modes of the 54-story building. Fig 8 (left) shows the agreement of the pulse arrival times at different levels in Millikan library obtained from models fitted by the different algorithms, as another demonstration of the goodness of the fit, and opportunity to rank the different fitting algorithms. Fig. 10 shows observed impulse responses for EW response of Millikan library in two frequency bands, 0-7.5 Hz and 7-15 Hz, the difference of which is an evidence of significant wave dispersion. Finally, Fig. 12 shows results for the identified shear wave velocities of the Bank of California building in moving time windows, for different window size.

Los Angeles 54-story office building (CSMIP 24629)



Fig. 2 Los Angeles 54-story office building (CSMIP 24629): photo, vertical cross-section, and typical floor layouts (redrawn from <u>www.strongmotioncenter.org</u>).



Equivalent uniform model fit

Fig. 3 Los Angeles 54-story office building: identification results for the NS (top), EW (middle) and torsional (bottom) responses. The table shows the equivalent uniform model properties, where Q and ζ are the apparent quality factor and apparent damping ratio. The four columns of plots show: (1) the observed pulse arrival time at the layer boundaries, (2) the assumed mass density profile, (3) the identified wave velocity profile during Northridge, 1994 earthquake, and (4) the observed average layer peak drift.



Los Angeles 54-story office building (CSMIP 24629)

Fig. 4 Los Angeles 54-sory Office Bldg: agreement of observed and model impulse response functions.



Fig. 5 Los Angeles 54-sory Office Bldg: agreement of observed (apparent) and model (fixed-base) transfer functions. The model NS and EW fixed-base mode shapes are also shown.

Millikan Library (NSMIP 5407)



Fig. 6 Millikan library: photo (left, courtesy of M. Trifunac), vertical cross-section (center), typical floor layout (right-top), and sensor locations at basement (right-bottom), (redrawn from Snieder and Şafak 2006).



Equivalent uniform model fit

Fig. 7 Millikan library: identified velocity profiles during Yorba Linda, 2002, earthquake. The table on the top shows the equivalent uniform model properties, where Q and ζ are the apparent quality factor and apparent damping ratio. The plots show detailed model fits by different algorithms. Top: NS response (0-15 Hz).
Middle: EW response (0-7.7.5 Hz, left; 7-15 Hz, right). Bottom: torsional response (0-25 Hz). The algorithms used are: LSQ = the nonlinear least squares fit (waveform fit, based on wave propagation theory), TSM = iterative the time shift matching, and Direct = the direct algorithms (based on ray theory).



Fig. 8 Millikan Library: agreement of observed and model pulse arrival times (left) and of observed (apparent) and model (fixed-base) transfer functions (right) for fits using different algorithms.

For Los Angeles 54-story steel building, the results show that, in the frequency band 0-1.7 Hs for NS and EW motions and 0-3.5 Hz for torsion, which include the first five modes of vibration, the structural response is essentially nondispersive and predominantly in shear. Therefore, a layered shear beam/torsional shaft model is appropriate in these frequency intervals. The same is true for the NS and torsional responses of the 9-story RC Millikan library for frequencies up to about 15 Hz, which contain most of the energy of the response. For the EW response, this is true for frequencies only up to about 7.5 Hz, and dispersive behavior was detected for higher frequencies (Rahmani and Todorovska 2012a, Fig. 10). For both buildings, the structural fixed base frequencies were also identified from the transfer function of the fitted model. For Millikan library, the identified fundamental fixed-base frequency differs significantly from the observed apparent frequency, especially for the NS response, due to soil-structure interaction, which agrees qualitatively with other studies. The difference is 42% for the NS, 33% for the EW and 31% for the torsional response. For the 54-story steel building, the difference is much smaller, in agreement with smaller degree of frequency shift for flexible structures (13% for NS, 5% for EW, and 6.5% for torsional responses). This difference is much smaller (more than three times) for the higher modes, which indicates very good fit and appropriateness of the simple nondispersive model for this building (Rahmani and Todorovska 2012b). Study of five different earthquakes, none of which caused damage, showed small variability of the identifies wave velocities, with sample coefficient of variation less than about 2% for

the equivalent uniform model wave velocities, and less than about 3.5% for the individual layers (Rahmani and Todorovska 2012b,c).

The Bank of California building suffered only *light* structural and nonstructural damage during the 1971 San Fernando earthquake, which was its first exposure to strong shaking. Yet, the moving window analysis shows *significant* permanent decrease of shear wave velocity, well beyond the uncertainty of the estimation, between Gnd and 6th floors of about 30% for NS and 37% for EW responses, and between 7th floor and roof of about 25% (Rahmani et al. 2012), indicating that this interferometric method *is* sensitive to light damage in RC buildings.

Millikan Library (NSMIP 5407)



Fig. 9 Millikan Library: agreement of observed (Yorba Linda, 2002) and model (LSQ fit, $\zeta = 2\%$) impuse responses for NS (0-15 Hz), EW (0-7.5 Hz), and torsional (0-25 Hz) responses.



Fig. 10 Millikan Library: evidence of dispersion in the observed EW response (Yorba Linda, 2002). Impulse responses are shown in different frequency bands. Left: 0-15 Hz (solid line) and 0-7.5 Hz (dashed line). Center: 0-7.5 Hz. Right: 7.5 – 15 Hz.

Comparison of the performance of the three fitting algorithms showed that the nonlinear LSQ fit of pulses as waveforms (i.e. the wave algorithm) gives most accurate estimation, especially for identification of detailed models from dense arrays (Rahmani and Todorovska 2012a). Over carefully chosen frequency band, it converged almost always, even in shorter moving windows (8-12 s). The scatter in the results for different window length (Fig. 12) is due to the Heisenberg-Gabor uncertainty principle (Gabor 1946), and cannot be avoided in windowed analyses. This scatter can be used as a measure of the uncertainty of the estimated wave velocities (Rahmani et al. 2012). The direct algorithm (ray algorithm), gives good initial estimates for the nonlinear LSQ algorithm (wave algorithm), but should not be used *per se* for identification of detailed models (e.g. one floor per layer), because of poor accuracy. Also it should not be used for identification of models with slabs, for which it produces biased estimates, due to wave scattering from the slabs, ignored in ray theory.

Sherman Oaks 12-story Office Bldg. (NSMIP 0466)



Fig. 11 Sherman Oaks 12-story Office Bldg.: photo (courtesy of M.Trifunac), and sketches of floor plans and side view showing the location of the strong motion instruments (redrawn form Bloome and Assoc. (1973).



Fig. 12 Sherman Oaks 12-story Office Bldg.: results of identified layer shear wave velocities, by the nonlinear LSQ fit algorithm (0-6 Hz), in moving time windows, for different window length (8s, 10 s and 12 s).

CONCLUSIONS

It is concluded that the presented interferometric identification method, involving matching of pulses in impulse responses and representation of buildings as layered shear beams, is valid for both steel and RC buildings up to some critical frequency, and is sensitive to light damage in RC buildings. Further, it is not sensitive to the effects of soils-structure interaction, and identifies the true fixed-base properties of the structure, in contrast to methods that match the modal frequencies (e.g. Naeim et al. 2006). It can be used: (1) to track changes in the structural integrity, and also (2) to simulate the building response and its performance during an earthquake, complementing performance based methods for rapid post earthquake assessment of buildings in programs such as the San Francisco Building Occupancy Resumption Program and similar future programs (City and County of San Francisco 2006, Naeim et al. 2006). Our analyses of Imperial County Services building, Van Nuys hotel building, Millikan library, Los Angeles 54-story steel building, Bank of California building, and Borik-2 building provide specific quantitative knowledge about the changes of wave velocities associated with damage, and about their variability due to factors other than damage (Todorovska and Trifunac 2008ab, Todorovska 2009b, Trifunac et al. 2010, Rahmani and Todorovska 2012abc, Rahmani et al. 2012). They showed that steel structures exhibit smaller variability due to factors other than damage than the RC buildings, for which the history of prior exposure to strong shaking is important. Analysis of more such case studies will help expand and refine such knowledge for different types of structures.

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