Wavelet and HHT Based Identification of Different Levels of Inelastic Action in RC Structures

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SUMMARY:
A fiber-based finite element approach capable of replicating the actual non-linear hysteretic behavior of reinforced concrete structures was used to generate a numerical model of a single column bent bridge. In order to induce different levels of inelastic action into the bridge, the model was subjected to different levels of seismic excitation. The acceleration response of the bridge was analyzed using Wavelet and Hilbert-Huang Transforms to evaluate the efficiency of these techniques to detect yield episodes and frequency sifts in the structure. Eigenvalue analyses were also performed before and after each strong motion to compare with the results obtained from the analysis of the strong motion response. While the evaluated methodologies were able to identify the yield episodes and shifts in the natural period of vibration, an estimation of the level of damage is still uncertain and future research efforts should focus in this aspect.

Keywords: system identification, signal processing, nonlinear modeling, empirical mode decomposition

1. INTRODUCTION

This paper presents a numerical evaluation of the capabilities of Wavelets Transforms (WT) and Hilbert – Huang Transforms (HHT) in the identification of different levels of inelastic action in structures with complex hysteretic response and subjected to nonstationary/nonlinear base excitations. For this purpose, a numerical model of a reinforced concrete single column bent bridge was developed and subjected to different levels of seismic excitation. The model was developed using fiber based elements capable of replicating the actual non-linear hysteretic behavior of reinforced concrete members. The objective of this study is to investigate the capabilities and limitations of WT and HHT to identify the changes undergone in the structure based solely in the analysis of the dynamic acceleration response of the structure to the damaging event, i.e. no finite element models or previous analysis of the pristine structure are employed for baseline comparison purposes.

2. FIBER BASED MODELING OF REINFORCED CONCRETE STRUCTURES

In a fiber-based model, the flexural member is represented by unidirectional fibers and constitutive material relationships are specified to each type of fiber. In reinforced concrete (RC) members, for example, fibers representing the reinforcing steel, cover concrete (unconfined), and core concrete (confined) can be employed (Fig. 1 - left). The bridge model was developed in the *OpenSees* (Open System for Earthquake Engineering Simulation) software framework system (McKenna et al. 2000) using the *BeamWithHinges* element (Scott and Fenves 2006). This element confines the nonlinear constitutive behavior to plastic hinge regions of a specified length while maintaining numerical accuracy and objectivity. The model uses the force-based fiber beam-column element formulation for the hinge region and the section response between hinges is assumed linear elastic (Fig. 1 - right). Confined and unconfined concrete fibers are modeled using the *Concrete02* material, the input data required: maximum compressive strength, strain at maximum strength, crushing strength, and strain at crushing, were calculated as proposed by Mander et al. (1988). Longitudinal steel bars are modeled using the *ReinforcingSteel* material (Mohle and Kunnath 2006), this model accounts for degradation of strength and stiffness due to cycling according to a Coffin and Manson fatigue model through the factors α, C_f, and C_d. The damage strain range constant α is used to relate damage from one strain range to an equivalent damage at another strain range and is constant for a material type. The ductility constant C_f is used to adjust the number of cycles to failure. A higher value of C_f translates to a larger number of cycles to failure. The strength reduction constant C_d controls the amount of degradation per cycle. A larger value for C_d will result in a lower reduction of strength for each cycle. Suggested values by Mohle and Kunnath (2006) for bars with a slenderness (ratio between the bar unsupported length and the bar diameter) of 6 are $\alpha = 0.506$, $C_f = 0.26$, and $C_d = 0.38$. In general, these values are expected to change with the steel type, bar diameter, and the confinement provided to the section. Fig. 2 shows the results obtained for a large scale RC cantilever column (Montejo et al. 2010) subjected to increasing cyclic reversals. In addition to the results obtained using the fiber approach, Fig. 2 also shows the results obtained using more traditional lumped plasticity approaches based on multilinear hysteretic rules. While this type of models have been successful used for years by the structural engineering community for seismic design or assessment purposes, their use for validation of damage detection techniques may not adequate due to the unrealistic abrupt change in stiffness proper of any multi-linear hysteretic model (Velazquez and Montejo 2011).

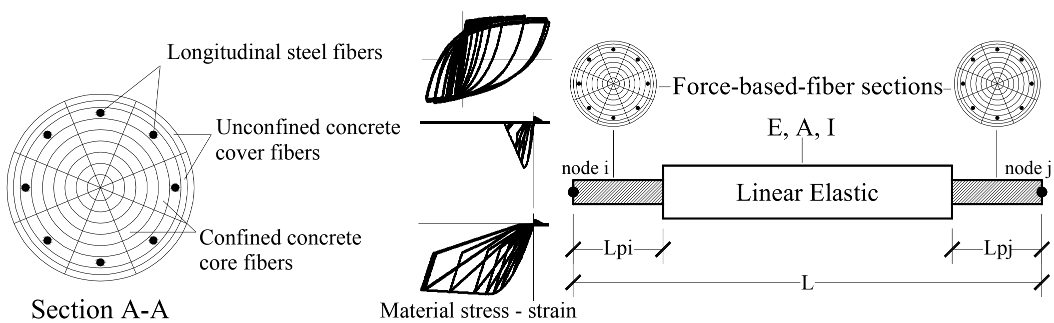

Figure 1. Fiber-based model using the BeamWithHinges (Scott and Fenves 2006) element in OpenSees

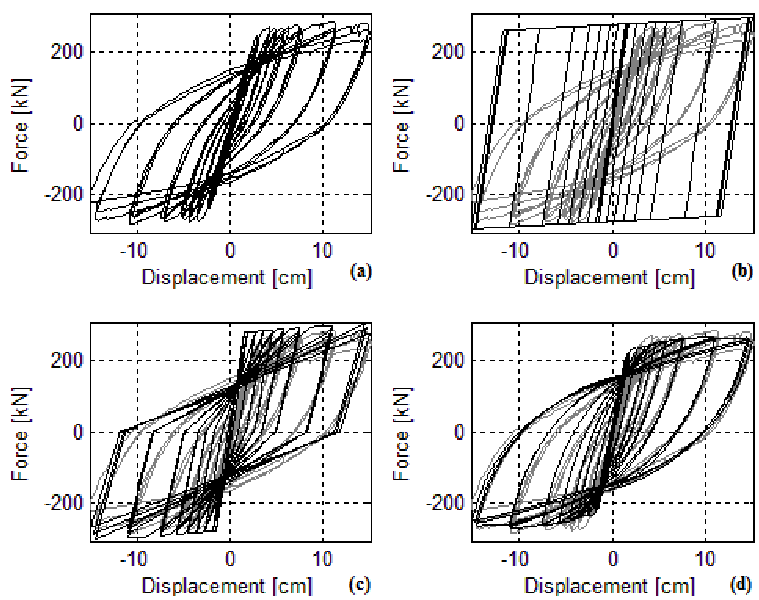

Figure 2. Hysteretic force-displacement response: (a) experimental test, (b) bilinear model, (c) multilinear "Takeda like" model and (d) fiber-based model

3. BRIDGE MODEL DESCRIPTION

The geometry and material properties of the bridge model (Figs. 3 and 4) were partially based on the Collector-Distributor 36 of the Santa Monica (I10) Freeway which was damaged in the 1994 Northridge earthquake. Cyclic and monotonic pushover analyses were used to track levels of strain and formation of plastic hinges to determine the structure yield displacement and force. Fig. 5 shows the results obtained, the estimated first yield force was 1624 kN and the displacement at ductility 1 was 0.033 m. These values will be used later to keep track of the yield episodes and ductility levels reached by the structure during the dynamic analyses. Once the lateral hysteretic response of the structure is characterized, an Incremental Dynamic Analysis IDA (Vamvatsikos and Cornell 2002) is performed to induce different levels of inelastic action in the structure. The elastic damping in all the dynamic simulations is represented by 0.5% tangent stiffness proportional damping (Priestley et al. 2007), that is, the damping coefficient is proportional to the instantaneous value of the stiffness and it is updated whenever the stiffness changes. The seismic input for the IDA was an acceleration record form the 1989 Loma Prieta earthquake registered at Treasure Island Station (Fig. 6). The results from the IDA are summarized in Fig. 7 (left). For the first record with peak ground acceleration PGA=0.05g the structure remained on the "elastic range" (meaning that strain in the reinforcing steel did not reached yield), the calculated displacement ductility was 0.36. For the last record (PGA=0.36g) the structure reached displacement ductility 5.81. Eigenvalue analyses were performed to the pristine structure and after the application of each strong motion, the estimated natural frequencies of vibration are displayed in Fig. 7 (right). These values will be used later to validate the results obtained using WT and HHT.

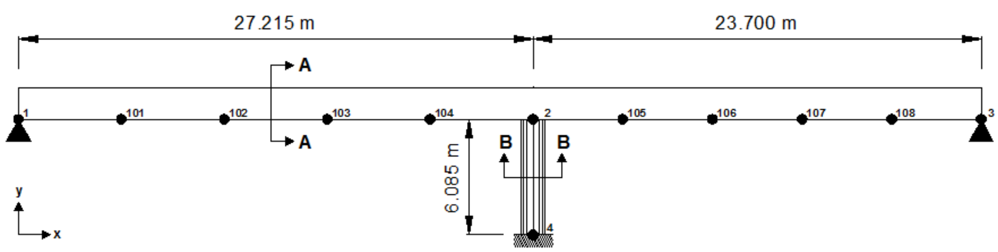

Figure 3. Simplified bridge model

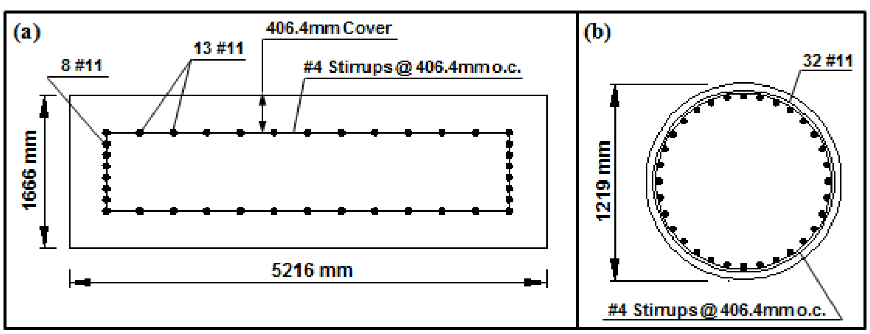

Figure 4. (a) Deck reinforcement details (b) Pier reinforcement details

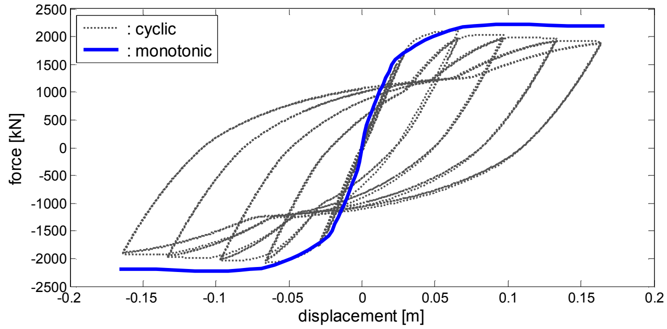

Figure 5. Monotonic and cyclic pushover results

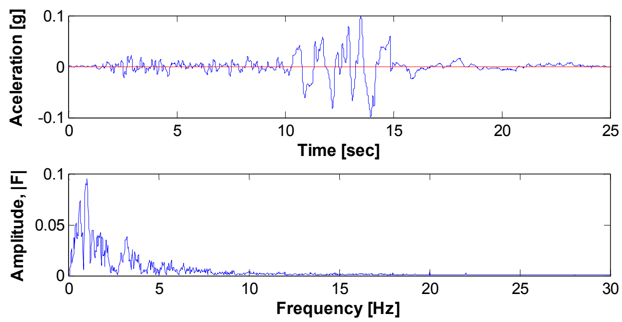

Figure 6. Loma Prieta earthquake and its Fourier Spectrum

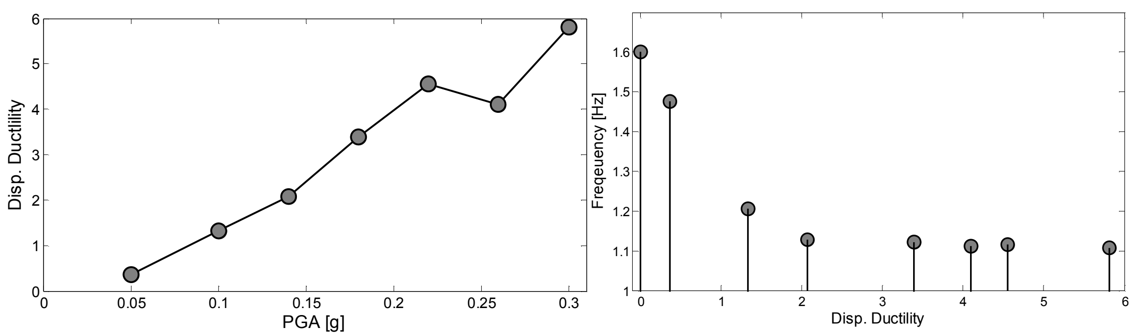

Figure 7. IDA curve (left) and change in the structure frequency of vibration as the ductility demand increases (right)

4. DAMAGE IDENTIFICATION USING SIGNAL PROCESSING TECHNIQUES

Based on the results of the numerical model we now explore the possibility of identifying damage (in the form of inelastic action) from the direct examination of the nonlinear-nonstationary characteristics of the structure dynamic response, eliminating in this way the dependency on large and detailed FE models or on the prior knowledge of the undamaged structure vibration characteristics. Analysis of the registered structural accelerations will be performed using two signal processing tools that allow for simultaneous time frequency examination, i.e. Hilbert-Huang transforms (Huang et al. 1998) and Wavelets Transforms (Mallat 1989). The occurrence of damage is associated with any changes in the vibration parameters (instantaneous frequency) or with the occurrence of singularities in the high frequency response. Details on the mathematical background and implementation of these techniques are presented in a companion paper (Montejo et al. 2012).

The HHT approach failed to identify the evolution of the natural frequency of vibration in all cases, mainly due to a serious mode mixing problem in the IMFs extracted. An ensemble of EMD results from different noise added signals (EEMD, Wu and Huang 2009) was also explored obtaining no significant improvements. Slightly better results were obtained when the signal was filtered previous to the application of the HHT. A 10^{th} order Butterworth filter was used to remove frequency contents above 2.2 Hz. The first 3 IMFs extracted from the filtered acceleration response of the structure to the earthquake with PGA=0.18g are displayed in Fig. 8. Instant frequencies results are displayed in Fig. 9 in the form of a Hilbert spectrum. It is seen that the resolution at which the frequencies of vibration are estimated and the presence of spurious modes does not allow a robust identification of the changes occurring in the system. Similar results were obtained for all the other cases. The occurrence of singularities in the high frequency response was explored by post-processing the first IMF of the unfiltered signal with an intermittency check (Yang et al. 2004, Xu and Chen 2004). In the intermittency check, data from the first IMF having frequencies lower than a specified intermittency frequency is removed by a straightforward counting process. The intermittency frequency should be smaller than the frequency of the discontinuity but larger than the highest structural frequency. The results obtained after applying an intermittency check at 50 Hz are displayed in Fig. 10 (right). Fig. 10 (left) shows the base shear history, the horizontal lines represent the force require for yield, each time the shear history cross an of these lines a yield episode (excursion into the inelastic range) have occurred. It is seen that with exception of the first case (PGA=0.05) where several spurious spikes appear, the spikes emerging from the post-processed first IMF can be related to yield episodes in the column.

Wavelet analyses were performed in the low and high frequency ranges using the Continuous Wavelet Transform (CWT) along with the modified Morlet Wavelet (Montejo et al. 2012). The results obtained in the low frequency range are displayed in Figs. 11 in the form of wavelet maps (left) and extracted ridges (right), the dotted horizontal lines represent the initial and final frequencies of vibration according to the eigenvalue analyses performed. It is seen that the evolution of the structure frequency of vibration is successfully identified in all cases. The results for the high frequency analyses are displayed in Fig. 12 (right), this figure shows only the wavelets coefficients that correspond to an 80Hz frequency. It is seen that the larger spikes appearing are all related to yield episodes. While spikes also appear in the first case where the structure remained in the “elastic range,” the magnitude of such spikes is negligible when compared to the magnitude of the spikes appearing in the cases where the structure was subjected to excursions into the inelastic range. The capabilities of the Discrete Wavelet Transform DWT (Mallat 1989) for high frequency singularities detection were also explored. In the DWT, a signal can be represented by its approximations and details. The approximations are the high-scale, low-frequency components of the signal. The details are the low-scale, high-frequency components. Fig. 13 shows the absolute values of the detail functions obtained via DWT with the Bior6.8 wavelet basis. As for the CWT analysis, the spikes in these plots relate well with the yield episodes in the column.

To verify if the amplitude of the spikes appearing in the high frequency analysis can be related to the level of inelastic demand in the structure, the maximum peak amplitude was identified in each time history and plotted against the maximum ductility demand reached in each case. The results obtained are displayed in Figs. 14, it is seen that most of the peaks amplitude follow the same pattern. However, the peaks amplitudes seem to saturate as the ductility level goes beyond 4.

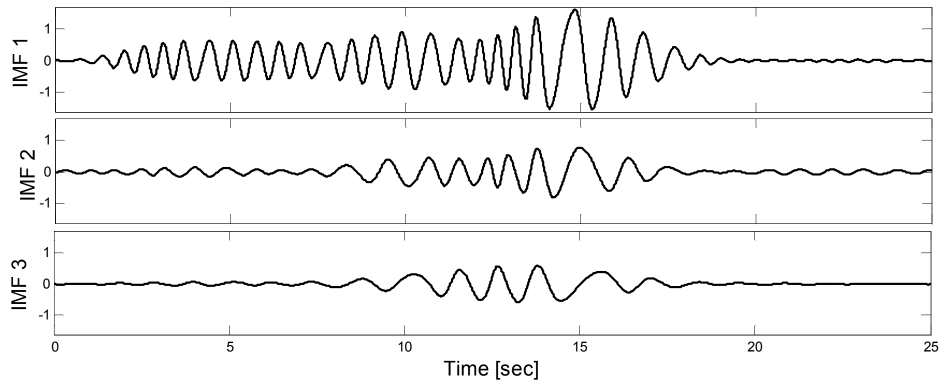

Figure 8. First 3 IMF obtained from the filtered acceleration response of the structure to the earthquake with PGA=0.18g.

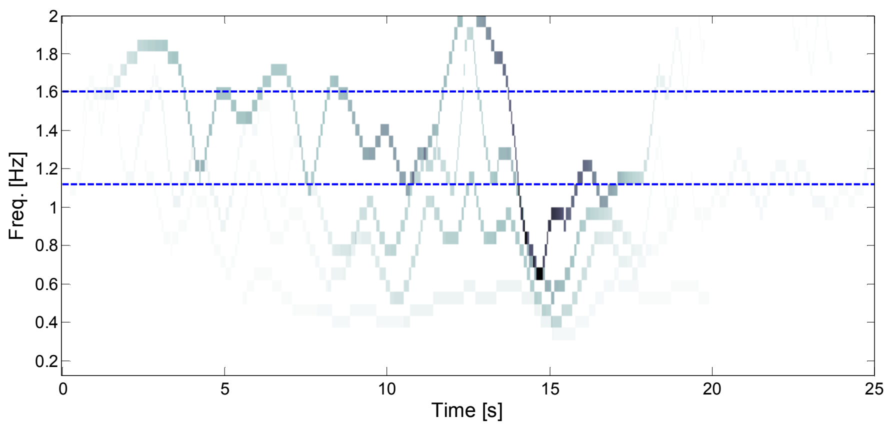

Figure 9. Hilbert spectrum for the filtered acceleration response of the structure to the earthquake with PGA=0.18g.

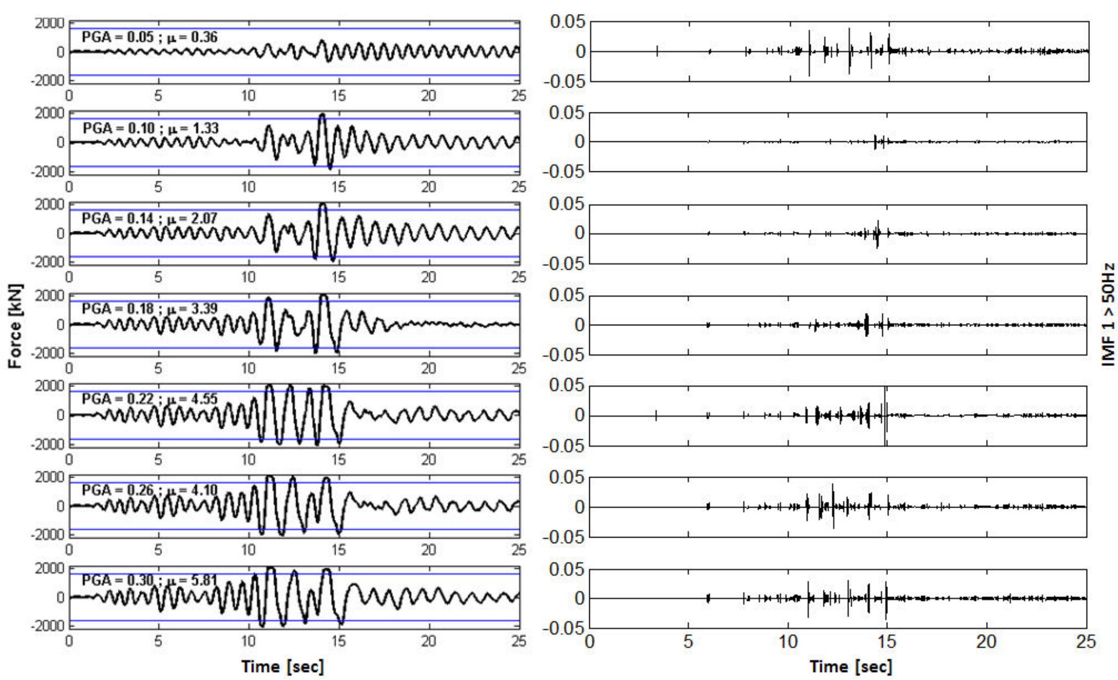

Figure 10. Left: Shear force time history, right: IMF 1 with intermittency check at 50 Hz

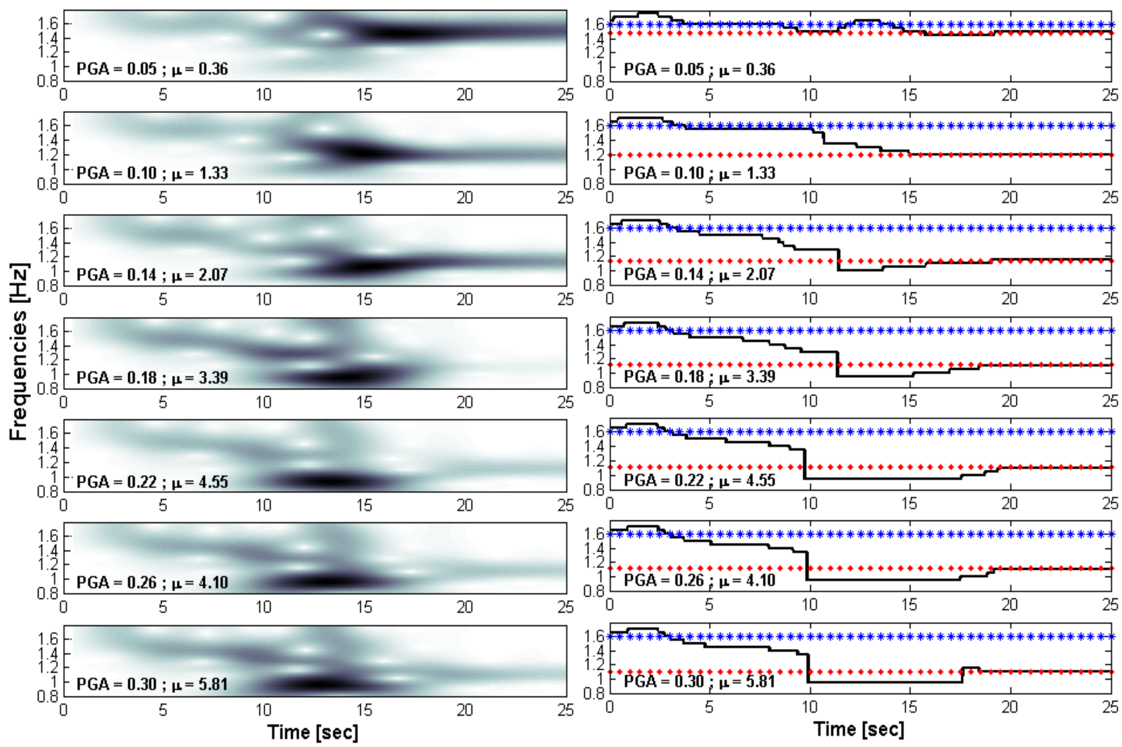

Figure 11. Left: Wavelet maps, right: extracted ridges (instant frequencies)

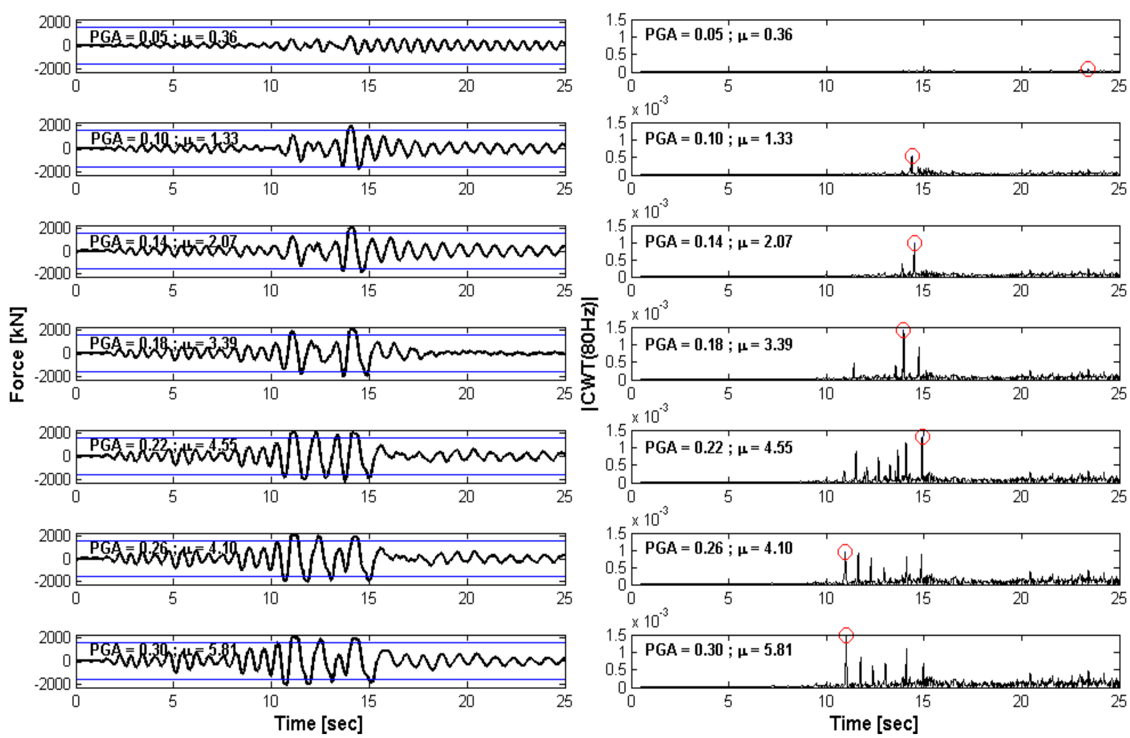

Figure 12. Left: Shear force time history, right: wavelet coefficients at 80 Hz

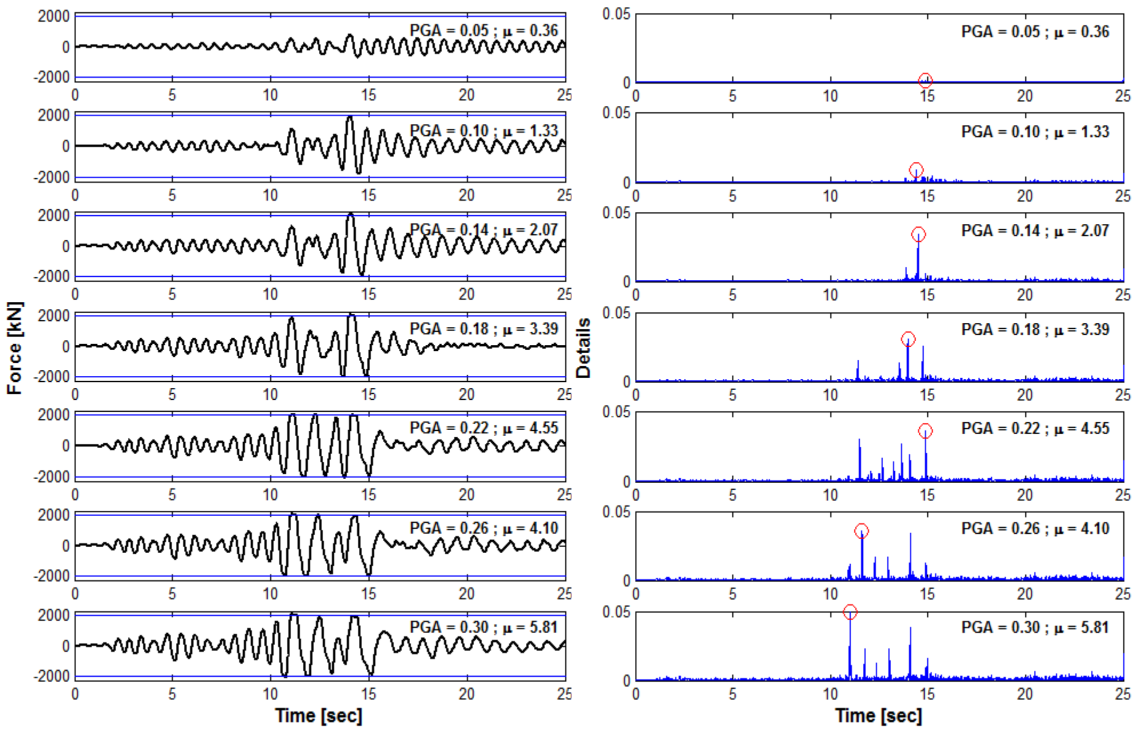

Figure 13. Left: Shear force time history, right: detail function via DWT

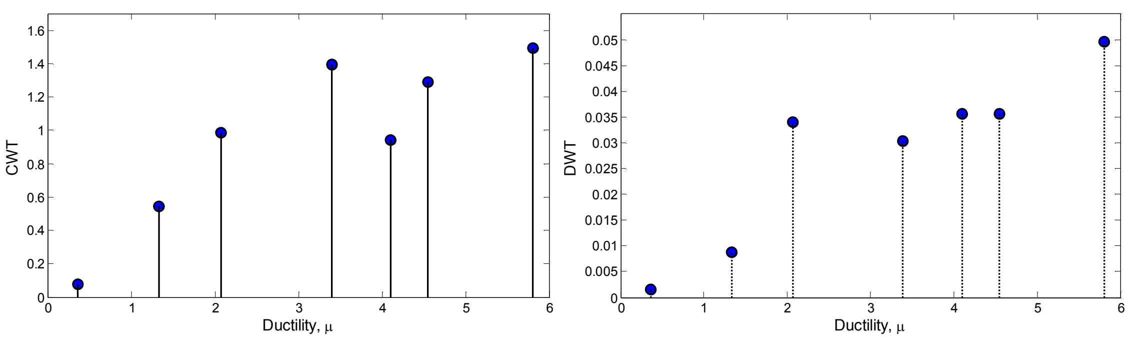

Figure 14. High frequency analysis maximum peak amplitude as function of the maximum ductility demand in each case. Left: CWT results, right: DWT results.

5. CONCLUSIONS

Wavelet Transforms based approaches were capable of identifying the change in the natural frequency and yield episodes of a reinforced concrete bridge model subjected to seismic excitations. HHT approaches failed to identify the change in natural frequency and partially detected the yield episodes. Notice however, that the efficiency of signal processing damage detection techniques can be largely influenced by the frequency content of the excitation and the presence of noise in the registered structure response (Montejo 2011, Velazquez and Montejo 2011) and such aspects were not analyzed in this work. While the evaluated methodologies were able to identify the damage instants, an estimation of the level of damage is still uncertain and future research effort should focus in this aspect. Despite the aforementioned challenges, signal processing based methodologies show great potential for on-line structural health monitoring of civil infrastructure. If both, high and low frequencies analyses are used; one can get a more clear idea of the changes undergone by the system being analyzed (Fig. 15). The results of the eigenvalues analyses before and after the structure is subjected to the base accelerations (Fig. 7 – right) suggest that the change in the frequencies of vibration may not be a good indicator of the level of damage (or inelastic action). Damage identification methodologies based solely in the observed frequency shift before and after a damaging event may largely underestimate the actual level of damage in the structure.

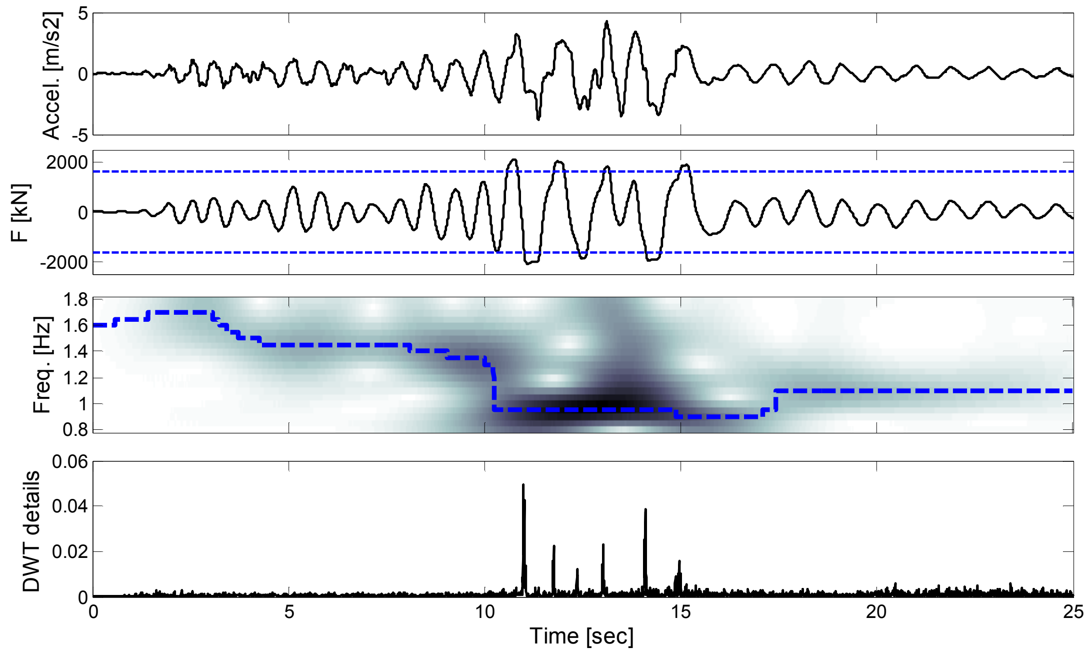

Figure 14. From top to bottom: Accelerations at column top, base shear, frequency variations (via CWT) and detection of yield episodes (via DWT)

ACKNOWLEDGEMENT

This research was performed under an appointment to the U.S. Department of Homeland Security (DHS) Summer Research Team Program for Minority Serving Institutions, administered by the Oak Ridge Institute for Science and Education (ORISE) through an interagency agreement between the U.S. Department of Energy (DOE) and DHS. ORISE is managed by Oak Ridge Associated Universities (ORAU) under DOE contract number DE-AC05-06OR23100. All opinions expressed in this paper are the author's and do not necessarily reflect the policies and views of DHS, DOE or ORAU/ORISE. Additional support was received from the Puerto Rico Strong Motion Program at the Civil Engineering Department of the University of Puerto Rico at Mayaguez. This support is gratefully acknowledged.

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