Improvement of damage assessment in buildings eliminating SSI effects using Signal Deconvolution

J. García-Solano and R. Rodríguez-Rocha
Escuela Superior de Ingeniería y Arquitectura, SEPI-Estructuras, IPN, Mexico

L.R. Fernández-Sola
Universidad Autónoma Metropolitana-Azcapotzalco, Mexico

15 WCEE
LISBOA 2012

SUMMARY:
In this work, damage assessment in buildings is improved applying deconvolution techniques to structural dynamic responses to eliminate Soil-Structure Interaction (SSI) effects. Using transfer functions to suppress the influence of the flexible base, modal information becomes more precise, thus, resulting on enhanced damage identification results. The Baseline Stiffness Method is applied to determine damage (loss of stiffness); location and magnitude. This method allows assessing damage in buildings without baseline modal information (undamaged state) using, solely, the approximated lateral stiffness of the first storey and the acceleration records of the damaged system. Controlled damage cases were simulated decreasing storey stiffness values of shear-beam building models. Results demonstrate the efficiency of the proposed methodology to identify damage in buildings without baseline modal information.

Keywords: Damage, SSI, buildings, deconvolution.

1. INTRODUCTION

Over time, buildings may suffer damage due to use, lack of maintenance, and mainly because of large-scale natural events such as earthquakes. During seismic events, damage in a structure may be such that will lead to collapse and consequently to economic and human losses. It is therefore vital to identify the extent of damage as a preventive decision tool for habitability and/or reinforcement. The extent of damage will depend, among other factors, on the type of soil in which the structure is supported, explicitly Soil-Structure Interaction (SSI). This effect produces an increment on the fundamental period of vibration and on the damping of the building, thus SSI must be considered in damage assessment.

Methods to identify and measure damage, also known as Structural Health Monitoring methods have the main purpose of providing information about the structural state of the system without damaging the structure. These methods are based on the premise that a change in the system, as the damage, is manifested as a change in the dynamic response of the structure. Therefore, these algorithms use, conventionally, acceleration records of undamaged and damaged states.

This confronts us with a very common problem, which is the unavailability of the dynamic response of structures in a healthy state. In order to solve this problem the Baseline Stiffness Method (Rodríguez et al., 2010) is presented in this work. This method determines location and magnitude of damage utilizing solely dynamic responses from the damaged structure and the approximated lateral stiffness of the first story without damage. Since the BSM considers a fixed base, dynamic response of the structure must not include soil dynamics.

Applying deconvolution techniques to structural dynamic responses to eliminate Soil-Structure Interaction (SSI) effects the structural properties are isolated. Using transfer functions (TF) to suppress the influence of the flexible base the modal information becomes more precise, thus, resulting on enhanced damage identification results.

Simulated damage cases in two shear-beam buildings were studied. Results demonstrate that the proposed methodology improves damage assessment, identifying the elements simulated as damaged and not reporting false elements as occur when not deconvoluted signals are used.

2. SOIL-STRUCTURE INTERACTION (SSI) MODEL

The structure is modeled as a shear-beam with flexible base according to Fernández and Avilés (2008).

Soil flexibility under earthquake excitation can be accounted in two parts: the site effects, and the SSI. Site effects produce modification of the seismic movement due to geotechnical properties of the superficial layers of soil. On the other hand, SSI effects depend of the difference between the structure and soil stiffness. In order to consider site effects, one-dimensional wave propagation in stratified media model was utilized, which for the case of depth clay deposits (Mexico City i.e.) is well addressed.

Using the transference function proposed by Wolf (1985) for a homogeneous layer the movement on the surface when a wave is started in a point of a base rock, after traveling through a soft media, is obtained.

To consider SSI it is possible to decompose it in two different topics (Whitman and Bielak, 1980). The first relates to an input movement modification, because of the presence of the foundation. The rigid foundation will experience some average horizontal displacement and a rocking component. This rigid-body motion will result in accelerations, which will vary over the height of the structure; this geometry averaging of the seismic input motion is known as the kinematic interaction (Wolf, 1985).. In this work, an approximate solution was used developed by Kausel et al. (1978).

On the other hand the inertial loads applied to the structure will lead to an overturning moment and a transverse shear acting at the base (Wolf, 1985). This effect is known as inertial interaction and it's controlled by the stiffness ratio between the structure and soil. Inertial interaction was modeled through the soil impedance functions (frequency-dependent stiffness and damping of soil-foundation). Based on the analogy with an elementary oscillator, the dynamic stiffness of the foundation for any mode of vibration is usually expressed by a complex function dependent of the frequency excitation (Gazetas, 1983).

The static stiffness for horizontal translation modes, rocking and coupling for circular foundations buried in a uniform layer with rigid base, were approximated by the expressions of Gazetas (1991), Sieffert and Cevaer (1992).

In the same way, the stiffness and damping coefficients for horizontal translation modes, rocking and coupling for circular foundations buried in a uniform layer with rigid base, were computed using formulas presented by Gazetas (1991), Sieffert and Cevaer (1992).

The structure was modeled as a shear-beam system of N Degrees of Freedom (DoF). Considering DoF corresponding to translation and rocking of the base, an $N+2$ DoF system is established, as shown in Fig. 2.1. As the impedance functions (springs and dampers support) depend of the frequency excitation it was convenient to use the method of complex frequency response and Fourier analysis (Chopra, 1995) to determine structural response.

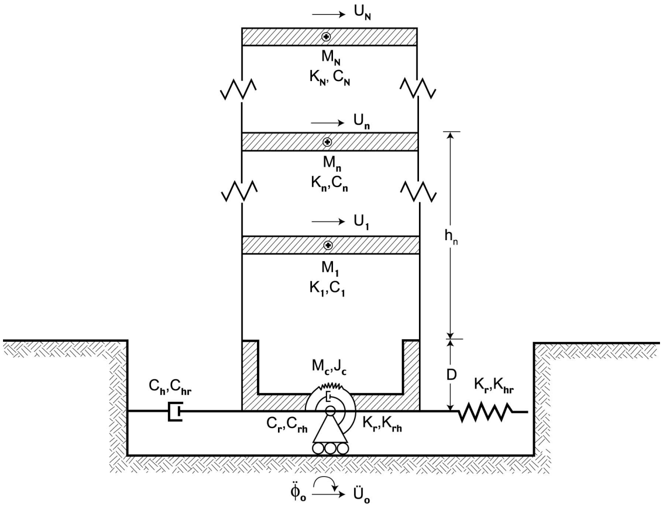

Figure 2.1. Soil-Structure Interaction model (Fernández and Avilés, 2008)

3. FREQUENCY DOMAIN DECOMPOSITION (FDD) METHOD

The FDD method was used to extract modal parameters from structural response (accelerations). According to Brincker et al. (2000) the q x q power spectral density matrix of the response must be obtained, where q is the number of responses. One way to determine this matrix is:

$$\left[\hat{G}yy(f)\right] = \left[\bar{Y}(f)\right]^T \left[Y(f)\right] \tag{3.1}$$

This matrix operates at discrete frequencies $f = f_p$ where p is a discrete series for each frequency in the domain. $\left[Y(f)\right]$ is the transformed response from time to frequency domain for each frequency value f. Eqn. (3.2) expresses Eqn. (3.1) as a Singular Value Decomposition, SVD:

$$\left[\hat{G}yy(f)\right] = \left[U_p\right]\left[S_p\right]\left[U_p\right]^T \tag{3.2}$$

where $[U_4]$ is a matrix containing singular vectors $\{u_{pk}\}$. [Sp] is a diagonal matrix containing singular values spk. These singular values can be plotted on the frequency domain and peak values may be observed which correspond to the natural frequencies of the system. A mode shape associated to each extracted frequency can be determined as well through SVD.

4. BASELINE STIFFNESS METHOD (BSM)

The BSM was applied to detect damage in buildings without baseline modal information (undamaged state). For a damaged plane frame of s number of floors and i mode shapes and performing signal processing techniques, natural frequencies ϖ and their corresponding mode shapes $[\phi]$ can be computed. Lateral stiffness and mass matrix, $[\overline{K}]$ and $[\overline{M}]$ respectively, are unknown with dimensions $s \times s$. On the other hand, it is possible to compute a vector $\{u\}$ of ratios k_i/m_i (Barroso and Rodríguez, 2004) with dimensions $2s-1\times 1$:

$$\{u\} = \left\{ \left(\frac{k_1}{m_1}\right) \quad \left(\frac{k_2}{m_1}\right) \quad \left(\frac{k_2}{m_2}\right) \quad \cdots \quad \left(\frac{k_i}{m_i}\right) \quad \left(\frac{k_{i+1}}{m_i}\right) \quad \cdots \quad \left(\frac{k_s}{m_s}\right) \right\}^T \tag{4.1}$$

This vector $\{u\}$ is computed utilizing modal parameters from the damaged structure and the first story approximated lateral stiffness k_1 assuming a shear-beam behavior. It is well known this assumption is valid for limited real cases, however, this is proposed just as an initial condition and the flexural effect will be included later on. In this sense, k_1 can be determined as:

$$k_1 = \sum \frac{12EI_1}{h_1^3} \tag{4.2}$$

Substituting k_1 into Eqn. (4.1), some parameters p_i are obtained using back substitution. Once all k_1 are known, the lateral stiffness matrix of the structure without damage $\left[\overline{K}\right]$ can be determined. In order to compute m_1 use back substitution instead of using k_1. These m_i are used to obtain the mass matrix of the structure $\left[\overline{M}\right]$. The former approach was applied to buildings without shear-beam behavior and it was observed that an approximated mass matrix $\left[\overline{Ma}\right]$ is obtained, which differs in magnitude to $\left[\overline{M}\right]$. The difference is null if k_1 is k_1/c, where c is a coefficient that adjusts shear to flexural behavior and it was found to correspond to the greatest eigenvalue of $\left[\overline{M}\right]\left[\overline{Ma}\right]^{-1}$. Thus, when the adjustment by k_1/c, for structures without shear-beam behavior is performed, the BSM provides its undamaged state $\left[\overline{K}\right]$. Simultaneously, a mathematical model of the structure is created considering connectivity and geometry of its structural elements and a unit elasticity modulus. Thus, approximated stiffness matrices $\left[ka_i\right]$ for each element are obtained. The global approximated stiffness matrix of the structure is:

$$\left[Ka\right] = \sum \left[ka_i\right] \tag{4.3}$$

According to Escobar et al. (2005), $\left[Ka\right]$ can be condensed to obtain $\left[\overline{Ka}\right]$ using a transformation matrix $\left[T\right]$ as:

$$\left[\overline{Ka}\right] = \left[T\right]^T \left[Ka\right]\left[T\right] \tag{4.4}$$

For a shear-beam building, $\left[\overline{K}\right]$ and Eqn. (4.4) just differ on material properties, specifically, on the magnitude of the elasticity modulus that can be represented using the matrix $\left[P\right]$ as $\left[\overline{K}\right] = \left[P\right]\left[\overline{Ka}\right]$. Solving $\left[P\right]$ from last equation yields:

$$\left[P\right] = \left[\overline{K}\right]\left[\overline{Ka}\right]^{-1} \tag{4.5}$$

On the other hand, stiffness matrices for each structural element of the undamaged state of the structure are computed as $\left[k_i\right] = P\left[ka_i\right]$; where P is a scalar that adjusts the material properties of the structure from the proposed model. This scalar is obtained as the average of the eigenvalues of matrix $\left[P\right]$, given in Eqn. (4.5). Eigenvalue computations are performed because are useful to obtain characteristic scalar values of a matrix, in this case $\left[P\right]$. It was found that the average of these eigenvalues is precisely P. Once the undamaged state of the structure, represented by $\left[k_i\right]$, is identified and condensed, it is compared against the stiffness matrix of the damaged structure $\left[\overline{Kd}\right]$ using the

Damage Submatrices Method (DSM, Rodríguez et al., 2009). This method is applied to locate and determine magnitude of damage, in terms of loss of stiffness, in percentage, at every structural element. According to Baruch and Bar Itzhack (1978), $[\bar{K}d]$ can be computed from measured modal information. Thus, the condensed stiffness matrix of the damaged system can be reconstructed as:

$$[\bar{K}d] = \left[\,[\bar{K}] - [M][Z]\right][H] + [M][q][\Omega]^2[q]^T[M] \tag{4.6}$$

where $[H] = [I] - [Y]$; $[Y] = [q][q]^T[M]$; $[Z] = [q][q]^T[\bar{K}]$ $[q] = [\phi]\left[[\phi]^T[M][\phi]\right]^{-\frac{1}{2}}$
$[\phi]$ is the modal matrix of the structure; and $[\Omega]^2$ is a diagonal matrix containing the eigenvalues of the system.

5. TRANSFER FUNCTION (TF)

This technique was used to eliminate SSI effects in order to process only information from structure (soil influence eliminated). For a linear and physically stable system with constant parameters its dynamic characteristics can be described through a Frequency Response Function *H(f)* (Bendat and Piersol, 1986) which is defined as the Fourier Transfer of *h(τ)* as:

$$\begin{aligned} h(\tau) &= Ae^{a\tau}, \tau \geq 0 \\ h(\tau) &= 0, \tau \leq 0 \end{aligned} \tag{5.1}$$

$$H(f) = \int_0^\infty h(\tau)e^{-j2\pi f\tau}d\tau \tag{5.2}$$

An important relationship for *TF(f)* is that being *X(f)* the system input Fourier Transform, and *Y(f)* the system output Fourier Transform:

$$Y(f) = TF(f)X(f) \tag{5.3}$$

solving for *TF(f)*:

$$TF(f) = \frac{Y(f)}{X(f)} \tag{5.4}$$

In this work, three TF were evaluated varying the input signal. Function TF-FF uses the free field motion as input and eliminates site effects from records (Stewart and Fenves, 1998). The second function (TF-FB) uses the movement at the foundation base as input. This function eliminates the horizontal components of the SSI effects and site effects (Stewart et al., 1999). A third TF, wich eleminates all the effects produced by the flexible base was also studied. This one uses the complete movement (translational and rocking) at the foundation base as input. This function was denominated TF-TR.

6. EXAMPLES

6.1. 10-storey shear-beam building model

A 10-storey shear-beam model was studied, considering a flexible base (Fernández and Avilés, 2008). The structure has a mass of 330.28 ton and a height of 30.5 m, a foundation mass of 79.53 ton, equivalent radius of 11.28 m, and an embedment of 5 m. The uniform soil layer has a βs=75 m/s (shear wave's propagation velocity), a depth of 50 m and a fundamental period T_s=2.5 s.

Three damage cases were simulated: D1SSI, D2SSI and D3SSI. D1SSI corresponds to an 86.36% reduction of stiffness on the first storey (Fernández and Avilés, 2008), D2SSI a 50% on the second storey and D3SSI a 50% on the fifth storey. Using acceleration records at every floor, the FDD was applied for each damage case to extract frequencies and mode shapes. Then the BSM was applied and location and damage magnitude was determined. Fig. 6.1 shows the identified location and magnitude of damage. It can be observed that in all models the BSM identified correctly the simulated damage locations, however, false elements were also identified for cases D2SSI and D3SSI with a magnitude even greater than simulated one. TF were applied to eliminate SSI effects and then the FDD and the BSM were also used (results in Fig. 6.2).

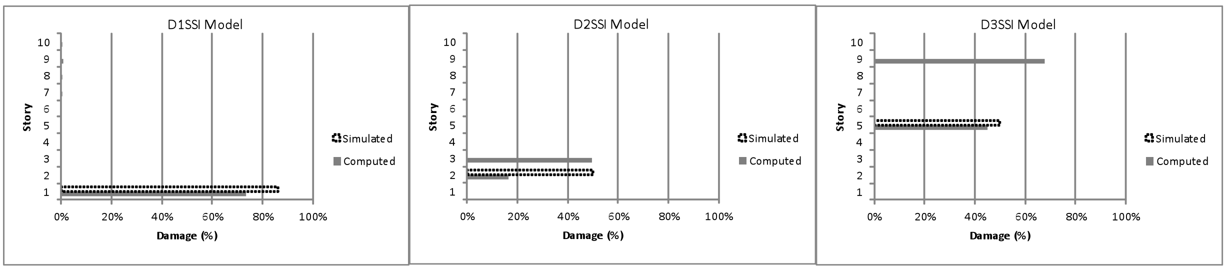

Figure 6.1. Damage location and magnitude, considering SSI. 10-storey building model.

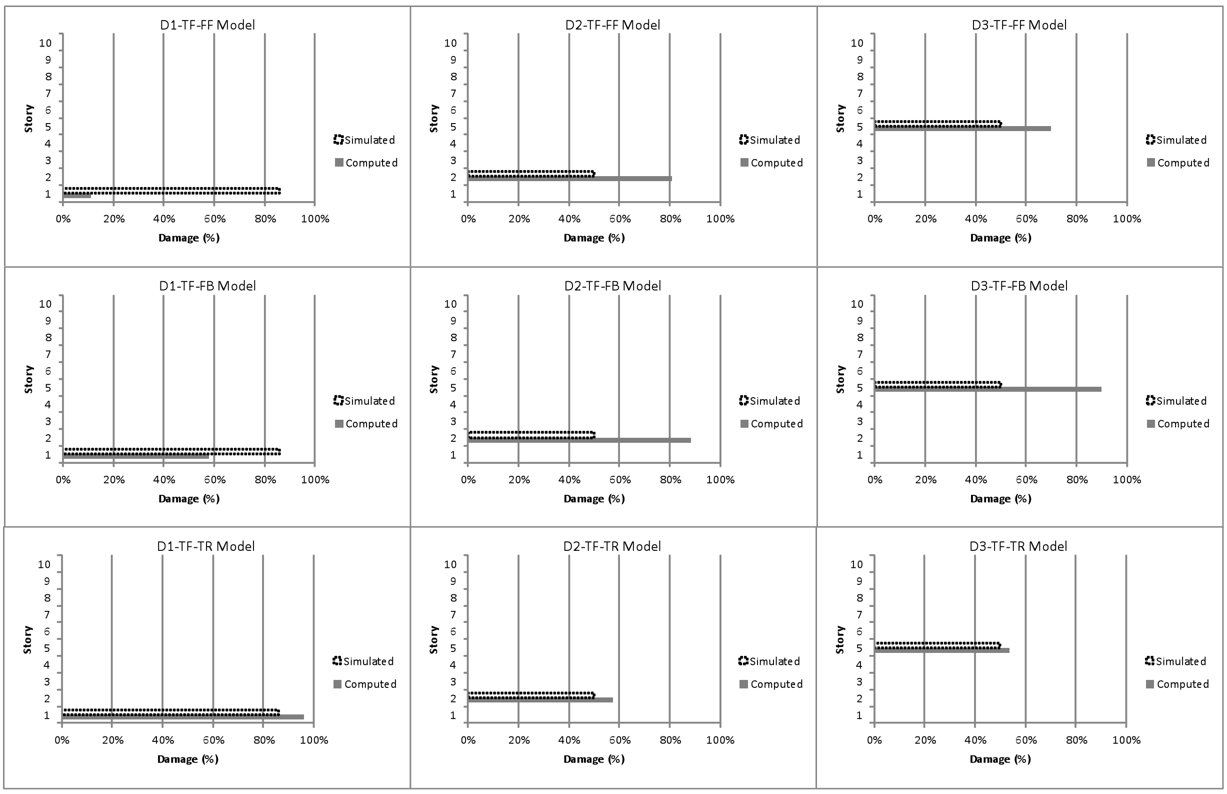

Figure 6.2. Damage location and magnitude using TF (no SSI). 10-storey building model.

For most cases error values of damage magnitude are greater than those commonly accepted in engineering (10%), however there is a tendency to decrease the error when the FT-TR is used. Despite these high error values estimation can be considered favorable in terms of safety for all cases except D1-TF-FF and D1-TF-FB.

SBM was formulated considering a fixed base. This is why damage localization results improve significantly (zero false elements) when SSI effects are neglected. Based on these results, it is recommend to use TF-TR to deconvolute signals.

6.2. 5-storey shear-beam building model

In order to study a mid-height common building a 5-storey shear-beam structure was studied. The structure has a mass of 41.28 ton and a height of 18.0 m. Foundation mass equal to 9.9 ton, equivalent radius of 5.64 m, and an embedment of 2.5 m. Soil has same properties as in last example.

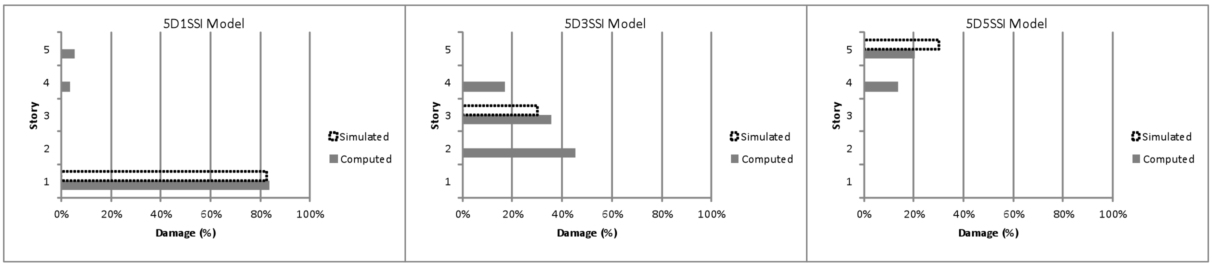

Figure 6.3. Damage location and magnitude, considering SSI. 5-storey building model.

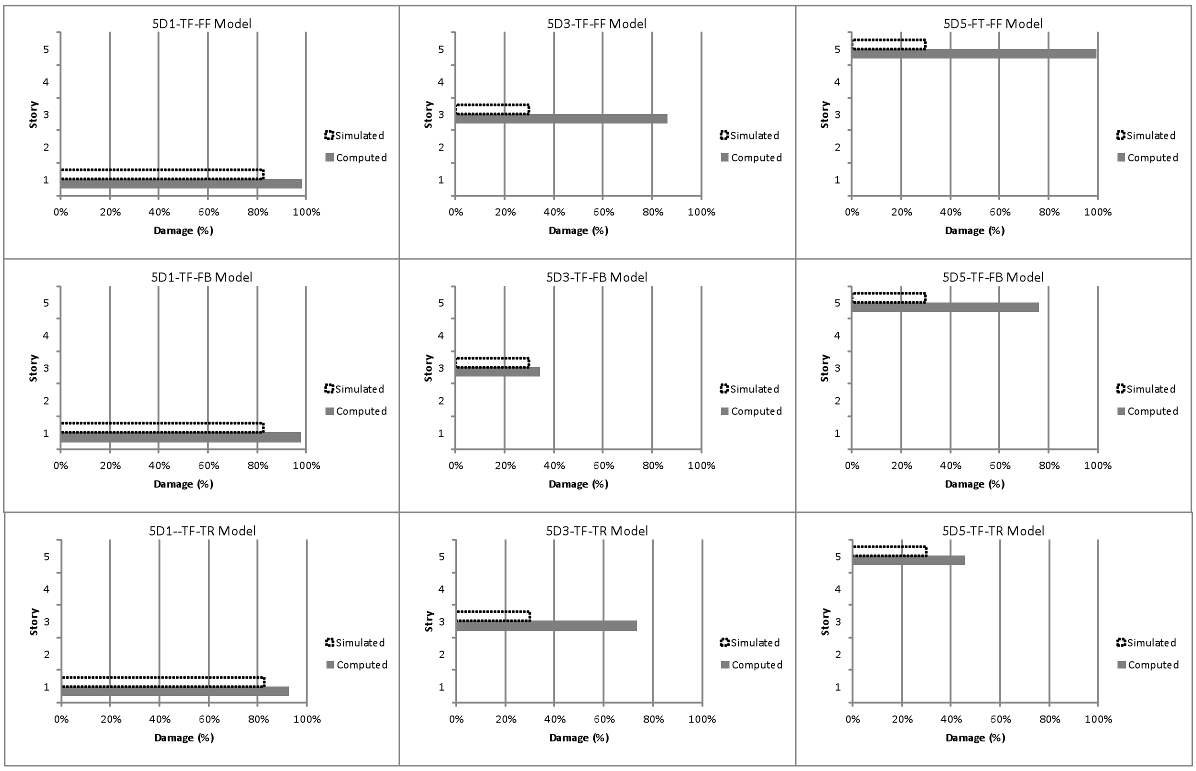

Figure 6.4. Damage location and magnitude, using TF (no SSI). 5-storey building model.

Three damage cases were simulated: 5D1SSI, 5D3SSI and 5D5SSI which correspond to an 82.5, 20 and 30% reduction of stiffness on the first, third and fifth storey respectively (Fernández and Avilés, 2008). Fig. 6.3 shows the identified location and magnitude of damage when SSI effects are still included on measured records. In all damage cases some locations were wrongly identified as damage when SSI were included. For case 5D1SSI, this problem is not so harmful, since both elements were identified with a false magnitude less than 5%. However, for cases 5D3SSI and 5D5SSI, these false locations correspond to a magnitude values greater than 10% in all cases.

Surprisingly all false location of damage disappeared when TF were used to eliminate SSI effects. These results are presented in Fig. 6.4. It can be observed that all computed damage magnitudes are greater than simulated ones which is favorable in terms of safety. Error values using FT-FB and FT-TR are smaller than when FT-FF was used.

7. CONCLUSIONS

In this work the BSM was applied to a 5 and 10-storey shear-beam models without baseline modal information to evaluate damage eliminating SSI effects. Damage cases were simulated decreasing storey stiffness. It can be concluded that the BSM identified all damaged elements without false indication when signals were cleaned using Transfer Functions. It is then recommended to apply deconvolution techniques to eliminate translational and rocking moving components of soil before using the BSM method to detect damage in structure without baseline modal parameters.

ACKNOWLEDGEMENT

The authors would like to acknowledge gratefully Instituto Politécnico Nacional for supporting this research project (20110376).

REFERENCES

Barroso, L. and Rodríguez, R. (2004). Damage detection of a benchmark structure without baseline information. *ASCE Journal of Engineering Mechanics.* **130(2)**, 142-151.

Baruch, M. and Bar-Itzhack, I.Y. (1978). Optimal weighted orthogonalization of measured modes. *American Institute of Aeronautics and Astronautics.* **16(4)**, 346-351.

Bendat, J.S. and Piersol, A.G. (1986). Random data. Analysis and measurement procedures, John Wiley and Sons.

Brincker, R., Zhang, L. and Andersen, P. (2000). Modal identification from ambient responses using frequency domain decomposition. *18th International Modal Analysis Conference.* 625-630, San Antonio, Texas.

Chopra, A. (1995). Dynamics of structures, Prentice Hall, Nueva Jersey.

Escobar, J., Sosa, J. and Gómez, R. (2005). Structural damage detection using the transformation matrix. *Computers and Structures.* **83**, 357-368.

Fernández, L. and Avilés J. (2008). Efectos de interacción suelo-estructura en edificios con planta baja blanda. *Revista de Ingeniería Sísmica.* **79**, 71-90.

Gazetas, G. (1983). Analysis of machine foundation vibrations: State of the art. *Soil Dynamics and Earthquake Engineering.* **2**, 2-42.

Gazetas, G. (1991). Foundation vibrations, Foundation Engineering Handbook, H Y Fang, Van Nostrand Reinhold, Nueva York.

Kausel, E., Whitman, R.V., Morray, J.P. and Elsabee, F. (1978). The spring method for embedded foundations. *Nuclear Engineering and Design.* **48**, 377-392.

Rodríguez, R., Escobar, J.A. and Gómez, R. (2009). Damage location and assessment along structural elements using damage submatrices. *Journal of Engineering Structures.* **31(2)**, 475-486.

Rodríguez R., Escobar J.A., and Gómez, R. (2010). Damage detection in instrumented structures without baseline modal parameters. *Journal of Engineering Structures.* **32:6**, 1715-1722.

Sieffert, J.G. and Cevaer, F. (1992). Handbook of impedance functions, Ouest Editions, France.

Stewart, J.P. and Fenves, G.L. (1998). System identification for evaluating soil- structure interaction effects in buildings from strong motion recordings. *Earthquake Engineering and Structural Dynamics.* **16**, 1-21.

Stewart, J.P., Fenves, G.L and Seed, R. (1999). Seismic soil-structure interaction in buildings. I: Analytical aspects, Postprints, UC Los Ángeles.
Wolf, J.P. (1985). Dynamic soil-structure interaction, Prentice-Hall, Nueva Jersey.
Whitman, R.V. and Bielak, J. (1980). Design Earthquake Resistant Structures: Foundations, E. Rosenblueth, Pentech Press, Inglaterra.