THE BASELINE STIFFNESSES METHOD FOR DAMAGE IDENTIFICATION WITHOUT BASELINE MODAL PARAMETERS AND DAMAGE ASSESSMENT OF A REINFORCED CONCRETE BUILDING

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ABSTRACT :

In this paper the Baseline Stiffnesses Method, BSM, is proposed to identify damage in buildings without knowing their undamaged state. The method is aimed at determining stiffness degradation of structural elements in buildings without knowing baseline modal parameters. To compute a reference state, the BSM utilizes based on eigenvalue calculations, solely, modal information from the damaged system and the approximated lateral stiffness from its first storey. This reference state is compared to the damaged one to detect and measure severity of damage. The proposed method is applied to two structures for both simulated and real damage. Results are analyzed and the advantages of using the proposed method are discussed.

KEWWORDS: Damage, buildings, baseline state, modal parameters.

1. INTRODUCTION

Buildings may be damaged under external forces such as earthquakes. If structural damaged is not identified on time, the system would collapse leading to loss of human lives. Even for the case of visual inspection, a previous damage state of the structure is needed to determine structural degradation. This state, also called baseline or reference state, can be computed based on modal parameters. The main problem is that baseline modal parameters are unknown and just information from the damaged state of the building is available. These facts prompt the need of having health monitoring methods based on parameters that serve as indicators of damage.

A contribution to the solution of this problem is the Baseline Stiffnesses Method, BSM, which is proposed herein to identify damage in buildings without knowing their undamaged state. This method is aimed at determining the stiffness degradation of structural elements in buildings without knowing baseline modal parameters. In order to compute a reference state based on eigenvalue calculations, the BSM utilizes, solely, modal information from the damaged system and the approximated lateral stiffness from its first storey. This reference state is compared to the damaged one to detect and measure severity of damage.

2. BACKGROUND

Several methods have been oriented for the purpose of obtaining a reference state of structures to be used to identify damage without knowing baseline modal parameters. For example: Stubbs and Kim

(1996) presented a sensitivity method, which, based on an iterative procedure, identifies baseline modal parameters of a structure. Kharrazi et al. (2000) applied sensitivity techniques and experimental data to fit an analytical model. The Stiffness-Mass Ratios Method (Barroso and Rodríguez, 2004) computes a baseline state for buildings with shear-beam behaviour and regular distribution of mass and stiffness. However, their method just identifies storey damage and can not measure stiffness degradation at every damaged structural element.

Applications of the BSM method demonstrate its capability to localize and measure severity of damage at every structural element for both simulated and real cases.

3. BASELINE STIFFNESSES METHOD

In order to identify damage in buildings without baseline modal parameters (undamaged state), the BSM is developed herein. This method utilizes stiffness-mass ratios to determine a reference state (baseline) from a structure based on modal parameters from the damaged system and the approximated lateral stiffness of the first storey. The identified reference state is compared to the damaged one to determine location and magnitude of damage (loss of stiffness, in percentage).
For a damaged plane frame of s number of floors and i mode shapes, experimental natural frequencies ω and their corresponding mode shapes $[\phi]$ can be computed using signal processing techniques. Lateral stiffness and mass matrix of the frame, $[\overline{K}]$ and $[\overline{M}]$ respectively, are unknown and of dimensions $s \times s$.

On the other hand, using the procedure developed by Barroso and Rodríguez (2004), it is possible to compute a vector $\{u\}$ of the frame ratios k_i/m_i with dimensions $2s-1\times 1$:

$$\{u\}=\left\{\left(\frac{k_1}{m_1}\right)\ \left(\frac{k_2}{m_1}\right)\ \left(\frac{k_2}{m_2}\right)\ \cdots\ \left(\frac{k_i}{m_i}\right)\ \left(\frac{k_{i+1}}{m_i}\right)\ \cdots\ \left(\frac{k_s}{m_s}\right)\right\}^T \tag{3.1}$$

This vector is computed utilizing modal parameters from the damaged structure and the first storey approximated lateral stiffness k_1 assuming a shear beam behavior. It is well known that this assumption is valid for a limited number of real cases, however, it is proposed just to define an initial condition; flexural effects can be included afterwards. In this sense, k_1 can be determined as:

$$k_1 = \sum \frac{12EI_1}{h_1^{\,3}} \tag{3.2}$$

Substituting k_1 into eq. (3.1), some parameters p_i are obtained using back substitution:

$$\begin{aligned}
p_1 &= k_1 \\
p_{i-j} &= \frac{p_{i-(j+1)}u(j+4)}{u(j+5)} \\
&\vdots \\
p_{i-1} &= \frac{p_{i-2}u_4}{u_5} \\
&\vdots \\
k_i &= \frac{p_{i-1}u_2}{u_3}
\end{aligned} \tag{3.3}$$

for $j = 2,3,\ldots,(i-2)$

Once every k_i is known, the lateral stiffness matrix of the structure without damage, $\left[\overline{K}\right]$, is calculated. In order to calculate m_i, m_1 is utilized in eq. (3.1) 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 of $\left[\overline{M}\right]$ (Rodríguez, 2007). The difference is null if k_1 is $\frac{k_1}{c}$, where c is a coefficient that adjust shear to flexural behavior and corresponds to the greatest eigenvalue of $\left[\overline{M}\right]\left[\overline{Ma}\right]^{-1}$. Thus, when the modification by $\frac{k_1}{c}$, 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, geometry of its structural elements and a unit elasticity modulus. Thus, approximated stiffness matrices $[ka_i]$ for each element are obtained. The global approximated stiffness matrix of the structure is:

$$[Ka] = \sum [ka_i] \tag{3.4}$$

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

$$\left[\overline{Ka}\right] = [T]^T [Ka][T] \tag{3.5}$$

where

$$[T] = \begin{bmatrix} [I] \\ -[Ka_{22}]^{-1}[Ka_{21}] \end{bmatrix}$$

for: (3.6)

$$[Ka] = \begin{bmatrix} [Ka_{11}] & [Ka_{12}] \\ [Ka_{21}] & [Ka_{22}] \end{bmatrix}$$

For a shear beam building, $\left[\overline{K}\right]$ and $\left[\overline{Ka}\right]$ just differ on material properties, specifically, on the magnitude of the elasticity modulus that can be represented using the matrix $[P]$ as:

$$\left[\overline{K}\right] = [P]\left[\overline{Ka}\right] \tag{3.7}$$

Solving $[P]$ from eq. (3.7):

$$[P] = \left[\overline{K}\right]\left[\overline{Ka}\right]^{-1} \tag{3.8}$$

On the other hand, stiffness matrices for each structural element of the undamaged state of the structure are calculated as:

$$[k_i] = P[ka_i] \tag{3.9}$$

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 the matrix $[P]$, eq. (3.8). Eigenvalue computations are performed because are useful to obtain characteristic scalar values of a matrix, in this case of $[P]$. It was found (Rodríguez, 2007), that the average of these eigenvalues is precisely P .

Once the undamaged state of the structure, represented by $[k_i]$, is identified, it is compared against the stiffness matrix of the damaged structure, $[Kd]$, using the Damage Submatrices Method, DSM, (Rodríguez and Escobar, 2005). This method is applied to locate and determine magnitude of damage, in terms of loss of stiffness, in percentage, at every structural element.

4. THREE-STOREY FRAME

The proponed method was applied to a three-storey frame (Figure 4.1), whose baseline modal parameters are known. It is a reinforced concrete plane frame from a building on a soft soil in Mexico City (Fierro et al., 1999).

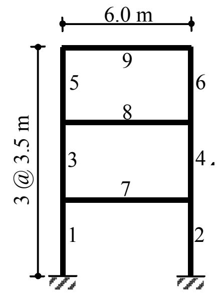

Figure 4.1. Three-storey frame

Columns are 0.30 x 0.40 m and beams are 0.30 x 0.60 m. Table 4.1 presents simulated damage cases and results computed using the BSM and the ones from Fierro et al. (1999) utilizing the Transformation matrix method, TMM.

Table 4.1. Damage detection of a three-storey frame applying the BSM and the TMM.

Damage case	Damaged element	Theoretical damage (%)	Computed damage (%)		e (%)		Iteration
			TMM	BSM	TMM	BSM	
C1	1	10	10	10	0	0	6
	2	20	20	20	0	0	
C2	1	50	50	50	0	0	6
	2	30	30	30	0	0	
C3	7	30	30.1	30	0.3	0	3
	8	20	20	20	0	0	
	9	50	50.5	50	1	0	

It can be observed from this Table that both methods determined location and magnitude of damage adequately with relative error values (e) smaller than 1 %. Also, for the same number of iterations the BSM produced zero error values which demonstrate its efficiency to estimate magnitude of damage.

5. REINFORCED CONCRETE BUILDING IN VAN NUYS CALIFORNIA

In order to evaluate the BSM applied to a real damage case, the seven-storey reinforced concrete building, located in Van Nuys California, was studied. The 1994 Northridge earthquake damaged the structure (Trifunac et al. 1999). Figures 5.1 and 5.2 show the geometry of the structure.

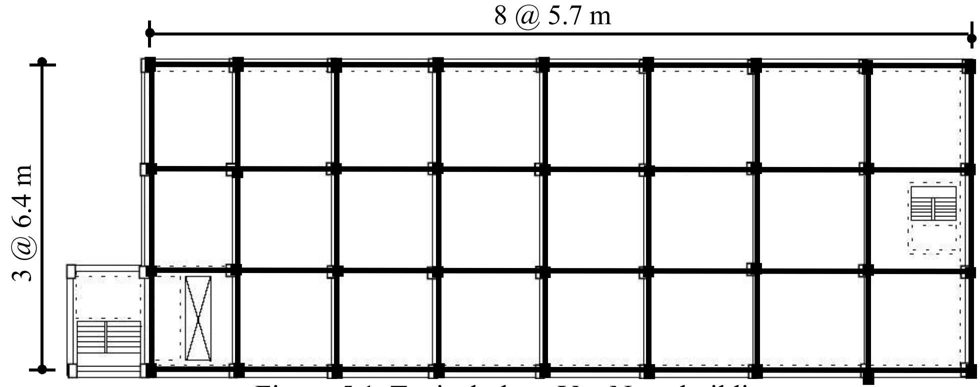

Figure 5.1. Typical plant. Van Nuys building.

According to Trifunac et al. (1999) exterior columns dimensions are 0.35 x 0.50 m; interior columns of the first storey are 0.50 x 0.50 m. All other interior columns are 0.45 x 0.45 m. Beams are 0.40 x 0.55 m. Slab depth: floor 1, 0.25 m; floors 2 to 6, 0.21 m; roof, 0.20 m. Modulus of elasticity: $E = 25.5$ GPa, except for the first storey columns ($E = 28.9$ GPa).

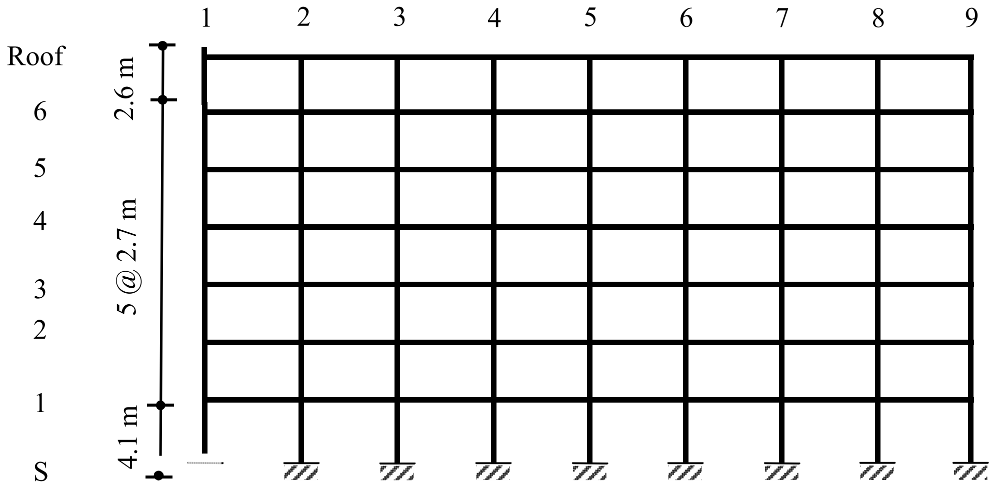

Figure 5.2. Typical longitudinal frame. Van Nuys building.

On site measurements obtained from the March 20, 1994 Northridge aftershock were processed applying the Frequency Domain Decomposition method (FDD) from Brincker et al.(2000) to determine modal parameters from the damaged structure. Dynamic data from floors 2, 3, 6 and roof were also available and using the FDD method, the first two sets of modal parameters were computed. Table 5.1 shows the computed frequencies and those by Trifunac et al. (1999).

Table 5.1. Van Nuys building natural frequencies (Hz) computed using the FDD.

Mode	FDD	Trifunac et al. (1999)	*e* (%)
1	0.99	1.00	-1.33
2	3.49	3.50	-0.24

It is shown that for the identified modes, relative error absolute values are smaller than 2 %. Since there were no available dynamic measurements on three floors of the instrumented building, interpolation was performed to obtain complete mode shapes. Table 5.2 presents the mode shapes determined with the FDD.

Table 5.2. Mode shapes of the Van Nuys building utilizing the FDD.

Floor	Mode	
	1	2
Roof	-0.75	-0.50
6	-0.63	-0.06
5	-0.48	0.18
4	-0.33	0.42
3	-0.19	0.66
2	-0.02	0.49
1	0.02	0.31

Trifunac et al. (1999) also reported that two weeks after the earthquake, a structural condition of the system was performed through visual inspection. Figure 5.3 displays location of cracks along the height of the building, where six crack zones on floor 4 could be observed.

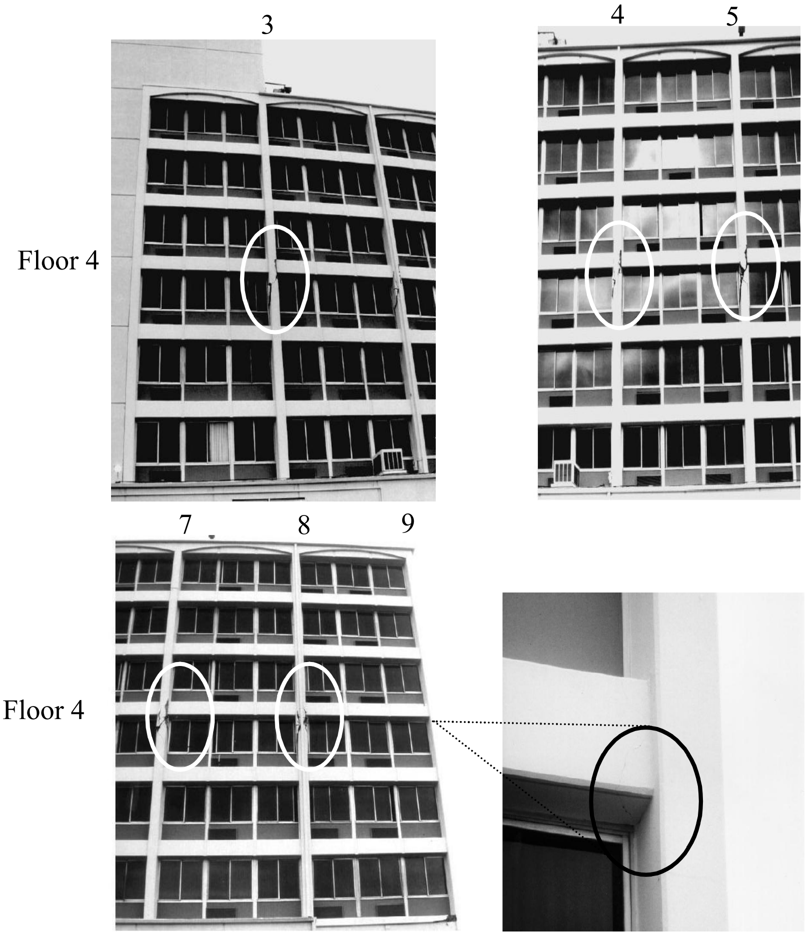

Figure 5.3. Location of damage observed on February 4, 1994.Van Nuys building. Trifunac et al. (1999).

According to Trifunac et al. (1999), cracks along axis 3 and 4 are approximately 5 cm wide; between 5 cm and 10 cm for axis 5 and 7; greater than 10 cm for axis 8 and smaller than 1 cm for axis 9. Notice that all cracks are located in or near beam-column connection zones (joints). Another crack, smaller than 1 cm, was also reported on axis 9, floor 2.

In order to identify damage in the Van Nuys building, without knowing its baseline modal parameters, the proposed BSM was applied. Modal parameter values from Tables 5.1 and 5.2 were processed. The lateral damaged stiffness matrix was fitted utilizing solely the first identified mode of vibration of the building. A mathematical model of the building was developed discretizing joints in 4 members (Figure 5.4). This Figure shows the location and severity of damage results computed with the BSM.

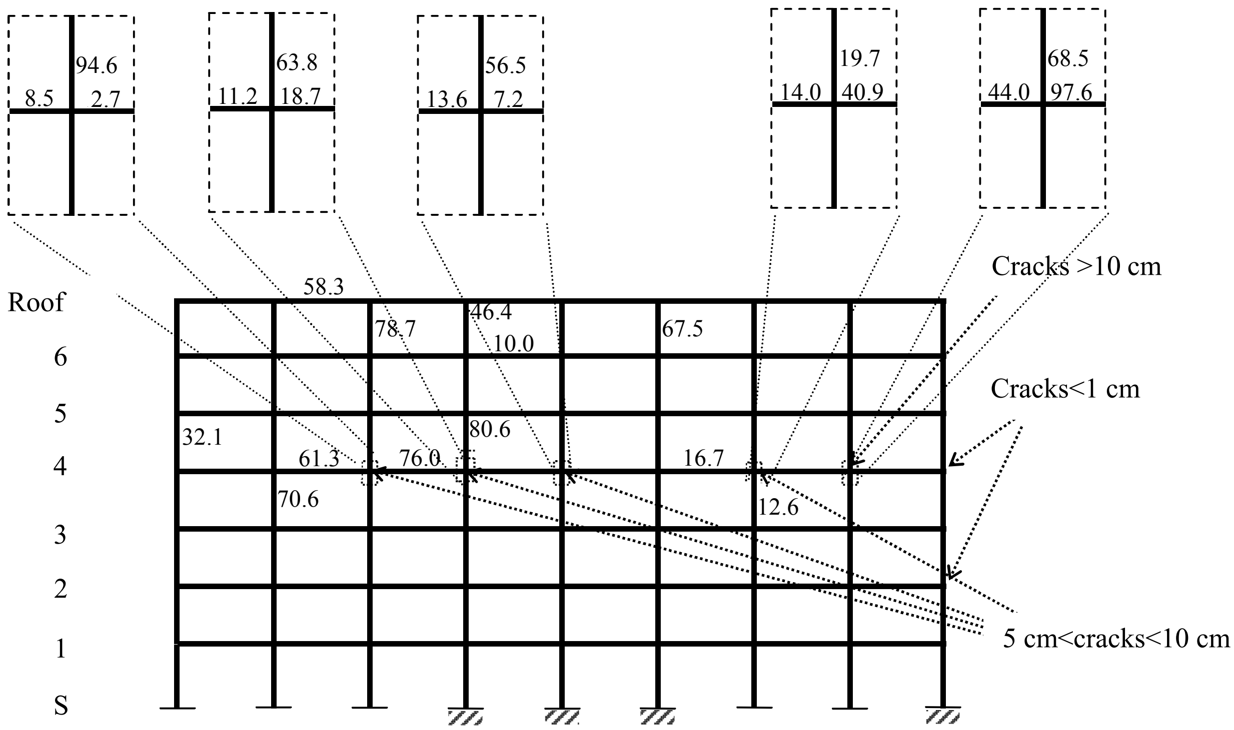

Figure 5.4. Location and damage magnitude (%) computed using the BSM. Van Nuys building.

Results from Figure 5.4 show that the BSM identified all cracks greater than 5cm wide. Also, the maximum computed damage magnitude corresponds to joints containing the widest observed cracks (Trifunac et al. 1999). Some beams and columns connecting damaged joints were identified as damaged, as well, as expected due to their connectivity. On the other hand the BSM did not locate the crack on axis 9. Finally, several members were falsely identified as damaged where there was no visual indication of cracks.

6. CONCLUSIONS

In this work the Baseline Stiffness Method was proposed to detect structural damage in buildings without baseline modal information (undamaged reference state). This method was applied to a reinforced concrete building that was damaged by the 1994 Northridge earthquake in California. The BSM located correctly the damaged members containing the reported cracks utilizing solely dynamic information of the damaged structure and the approximate lateral undamaged stiffness of the first storey. Only the first mode of vibration of the damaged building was utilized to fit the lateral damaged stiffness matrix of the system and just the first two to determine the reference state. The results from this real study case demonstrate the feasibility of the proposed method to locate damage in concrete buildings. Further studies about the relationship of the computed damage magnitude, using the BSM, and wide crack length are needed.

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