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Seismic damage identification of buildings in hilly areas using structural health monitoring data

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ABSTRACT: Dynamic response of a structure corresponding to a novel event such as, a seismic action can be used for identifying potential damage in the structure due to the event. After an earthquake, it is necessary to assess the damage state of a structure so that appropriate rehabilitation decision can be taken. This paper focuses on seismic damage assessment of building structures constructed in hilly areas. The buildings in these areas are usually not symmetrical in plan, and have vertical setbacks because of the features of the terrain which makes them particularly vulnerable to seismic forces. This type of buildings is referred to as step-back buildings. In this paper, a method for identification of seismic damage to step-back buildings using floor acceleration response records has been developed. A five-story step-back reinforced concrete framed building is considered for the study and the proposed method is shown to be feasible and computationally efficient.

1 INTRODUCTION

Health monitoring of structural systems is an important aspect of system identification (SI) in Civil Engineering and is of immense utility in planning post-earthquake rehabilitation program. Though the conventional method of physical inspection cannot be completely done away with, precious time can be saved by identifying the areas and/or buildings which need to be inspected physically. The use of system identification in structural health monitoring is based on the knowledge that vibration characteristics of mechanical systems are functions of stiffness properties of the system. Since the damage to a mechanical system manifests in stiffness and/or strength reduction, it follows that the vibration characteristics of the system will also be affected due to damage. Thus, in principle, it is possible to assess the health of a structure by using system identification techniques. Further, the change in vibration characteristics, *viz.*, natural frequencies, damping, and mode shapes, can only indicate that the structure has suffered some damage. For engineering applications, mere identification of the presence of some damage in a structure is not enough. It is also important that the location of damage also be identified. Monitoring the change in modal parameters of the structure is not very useful in locating the damage in a building as the lower vibration modes, for which reliable estimates can be obtained from the response data; do not contain sufficient information about the local behavior. On the contrary, the higher modes – containing information about the local behavior of the structure – cannot be identified accurately from the response data (Humar et al., 2006). This is because of insufficient energy of ground motion in the high frequency range to excite higher vibration modes of the structural system. Moreover, a prior knowledge of the modal parameters of virgin structure is a must for the use of methods based on change in modal parameters. This requirement makes it difficult to use these methods for the modal parameters estimated from ambient vibration records do not agree with the estimates obtained from the strong motion response records (Trifunac, 1972). Therefore, the use of parameters estimated from ambient vibration surveys as representative of the state of virgin structure may lead to a wrong assessment of the health of structure.

A new method has been presented in this study for identifying and locating the damage in multi-story framed buildings which does not require information about the state of virgin structure. The study is limited to the identification of damage in the framed buildings because of column failures. Although the strong column – weak beam concept is universally accepted as a sound design philosophy for lateral resistance, there have been several instances where the failure of columns initiated the damage (Villaverde, 1991). Studies reported in Yousuf and Bagchi (2008) indicate that in spite of using the capacity design (i.e., strong column – weak beam) approach, columns in a moment resisting frame with infill panels have significant vulnerability to lateral loads. Moreover, columns are more vulnerable in the case of reinforced concrete (R.C.) buildings where the construction has been completed in several phases, e.g., the upper floors may be constructed a couple of years after the construction of lower floors – a common practice in developing countries. The approach presented in this study, which can be used to identify and localize, at the floor level, damage in a R.C. framed building.

The damage in a particular story has been simulated by reducing story strength, represented by the sum of stiffness of all columns. It is assumed that no further damage occurs anywhere in the building due to a subsequent base excitation except in the damaged story. The response of building is computed by using the constant average acceleration implicit time marching scheme. The member forces at each node are calculated assuming elastic behavior which is then checked against a pre-defined yield surface. If the computed force state lies outside the yield surface then a plastic hinge is deemed to have developed at the node. However, if the point lies inside the yield surface then the solution proceeds as an elastic solution. On the formation of a plastic hinge at a node the forces and stresses are modified and the calculated unbalanced forces are redistributed (by means of suitable modifications in stiffness and damping matrices) in subsequent iterations until a specified convergence criterion is satisfied. The computations are terminated when the global stiffness matrix becomes singular (Thanoon, 1993). It is assumed that yielding takes place only at generalized plastic hinges, modeled as members of zero length.

2 DAMAGE IDENTIFICATION

Assuming that reliable estimates of the dynamic properties of the structural system in its virgin state are available, the modal parameters identified from the recorded response data to an earthquake can be used to assess the integrity of the structural system. The extent of damage is determined by the maximum softening index (δ) defined as (DiPasquale and Çakmak, 1990)

$$\delta = \frac{(T_1)_{\text{final}} - (T_1)_{\text{initial}}}{(\mathrm{T}_1)_{final}} \tag{1}$$

where, $(T_1)_{final}$ is the fundamental time period in damaged state; $(T_1)_{initial}$ is the fundamental time period in the virgin state of the structure. The basic assumption in this approach is that the damage affects the fundamental period of the structure but the changes in associated mode shape are negligible. A further refinement to this approach is to define a two-dimensional damage index by considering the changes in periods of vibration in the first two modes, Nielsen et al. (1992). For regular buildings the two-dimensional damage index gives a rough idea of the location of damage as either in lower or upper half of the structure. Moreover, this approach requires prior information about the natural periods of virgin structure which may not always be available. Therefore, a new approach for detecting and locating the damage is developed in the following. This approach is particularly suited for detecting damage in buildings constructed in stages. For such buildings the joint between the column stub (from an earlier phase of construction) and the casting of new column during the next phase of construction forms the weakest link in the entire structural system. Such buildings have a tendency to develop plastic hinges at such junctions during earthquakes

2.1 Damage Localization

As a first approximation, let us assume the floor response process to be a zero mean, ergodic Gaussian process with the sample realizations being characterized by nodal responses (representation of floor responses) of one component out of three orthogonal components. For example, the sample realization in the longitudinal direction is obtained as the longitudinal component of floor acceleration response. Similarly, realizations of the response in other two orthogonal directions can be characterized. The 3 x 3 temporal covariance matrixes of these three time histories represents the covariance matrix, C of the response process. Thus the ij^{th} element of the temporal covariance matrix C_{ij} is give by

$$C_{ij} = \frac{1}{N-1}\sum_{k=1}^{N}(a_i(k)-\mu_i)\ (a_j(k)-\mu_j)\,;\, i,j=1,2,3 \tag{2}$$

$$\mu_i \ (=\frac{1}{N}\sum_{k=1}^{N} a_i(k)) \tag{3}$$

where, a_i and a_j denote the two orthogonal components of acceleration response, N is the total number of data points in the time history, and μ_i is the temporal mean of the i^{th} component of acceleration response process. In this study, the covariance matrix C is measured by using, the determinant norm, and the trace norm respectively. The physical significance of scalar norms of the covariance matrices of the random process used are interpreted as follows. The diagonal terms of the covariance matrices, compact probabilistic representation for the random process, are variances. As the process is assumed to be zero mean, the variances are equal to mean square values. Thus, the energy of the random field can be expressed by sum of mean square values, i.e., the trace norm of the covariance matrix. Here, the product of mean square values, i.e., determinant norm, may be assumed as the 3D representation for strength of particle motion which constitutes the random processes. Now, the term damage location indicator (DLI) may be defined as:

$$\text{DLI} = \frac{Trace}{(Determinant)^{1/3}} \tag{4}$$

The DLI would be computed from the response records of undamaged and damage state of a structure. The overlapping of DLI values, obtained from different floor response records, means no relative change in motion due to additional energy input, which can be interpreted as no relative difference of motion among the floors. It may also indicate, that the floors (top of damaged columns at a particular floor level) are in motion as a whole, i.e., behaving as a rigid body. Thus, DLI can be used for localization of damage in a structure due to strength degradation of columns at story level. It is also important to note that the DLI should not be used as a generalized term and it is used mainly for resplendent representation of damage localization at story level. Now, the methodology will be implemented through simulation studies on a typical five-storey step-back building.

3. SIMULATED STUDY ON A TYPICAL BUILDING

The methodology developed is now implemented via simulation studies in a typical building and two characteristically different type of ground motions are considered for simulation of building response. The building is asymmetrical in plan also along height and five-storied with storey height 3m each (Fig.1). The geometrical and material information of the building are: the beam and column cross-section dimensions are 300mm x 300mm and are assumed to be same at all locations in the building. The slab thickness for all floors has been assumed to be 150mm. All constitutive properties of the reinforced concrete elements have been derived assuming the use of characteristic strength of concrete and steel 20Mpa and 415Mpa respectively. The building is assumed to be situated on firm strata and the soil-structure interaction (SSI) effects have been neglected. The damping is assumed to be Rayleigh damping with modal damping of 5% in the first and sixth mode. The resulting modal damping in the intermediate modes is approximately 5% as well, whereas the higher modes, with damping proportional to the square of the natural frequency, are damped out of the response.

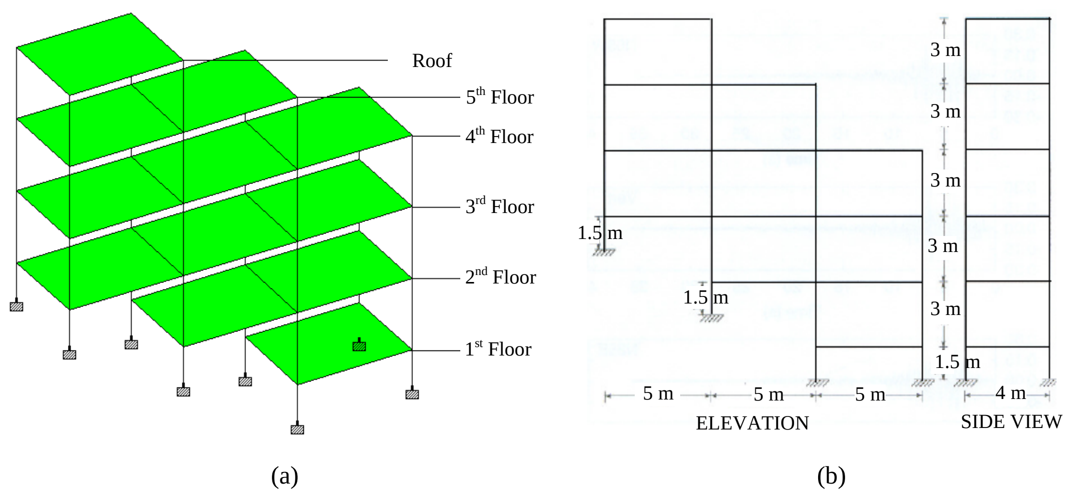

Figure 1: Geometric details of the building (a) Perspective view, (b) Schematic view

Now, the building is assumed to be affected by damage at ground floor and third floor respectively. As the acceleration response records at the optimal location of sensors are assumed to be available for the building in damage condition (Datta et al. 2001). And this acceleration response is generated numerically (Datta, 2001) corresponding to two real ground motions. During numerical simulation the damage was found to be localized at particular floor columns via the formation of plastic hinge at nodes. The response records from first, third floors and roof are used for damage detection and quantification purpose. The system parameters (frequencies) are identified from response records through an ARX model developed using an Multiple Input – Multiple Output (MIMO) format where three input and nine output data patterns are utilized. Then using time period and Eq. (1) damage is quantified in terms of damage index. Damage localization study is carried out by finding DLI, which is computed from the time window data (recorded time histories are divided in segments of 3.4s and 10s window length). The DLI can be studied as time dependent parameter and the nature of segmented data is assumed to be better suited for the methodology developed, particularly when the system behavior is non-linear.

3.1 *Damage due to Ground motion Type-I*

The building is assumed to be damaged and analyzed due to ground motion Type-I (Uttarkashi earthquake). The ground motion time history records of the Uttarkashi earthquake are shown in Fig 2(a). The damage state of the building has been simulated reducing story strength by 55.55%. The response records due to the ground motion of the un-damage and damaged building are used to compute DLI as described earlier. The DLI computed from the floor responses are plotted against centre of each time window shown in Fig.2(b) and 2(c). The legends are used to present DLI values corresponding to floor responses and first, third floors, roof in un-damage state and damaged state of the building. In these figures, D and UD indicate damaged and undamaged states, respectively. From the Fig. 2(b) it is observed that the DLI values computed from first, fourth and fifth floor responses are merged with each other. Therefore, it may be stated that at the first floor and above, the building did not experience with significant change in motion, floor wise. However, from the un-damage state, it was found that there are three well separated curves with respective DLI values of first, third floor and roof responses.

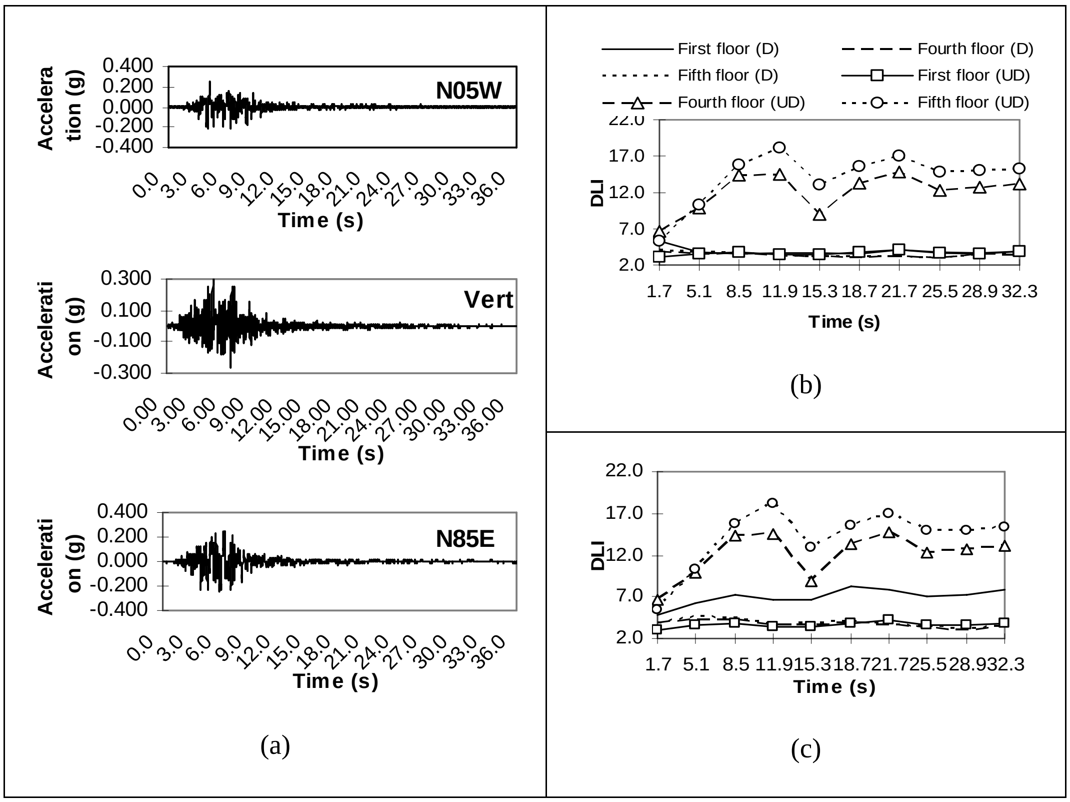

Figure 2: Ground motion Type-I: (a) Time histories of Uttarkashi Earthquake; (b) Damage Localization at the Ground floors; (c) Damage Localization at the Third floor.

The behavior of the building above the floor damaged can best be described with rigid body motion. It means no apparent change in motion along the height of the building is taking place due to marginal or nil contribution of energy of ground motion to the building. The behavior of building motion, can be similarly explained (Fig. 2(c)) when the damage is simulated at the third floor. Thus, overall change in the stiffness of the building and its corresponding effect on the motion can easily be understood from the curves. The change in time period estimated by ARX model corresponding damage indices are shown in Table 1. The identified first mode frequency for the building, when the ground floor columns are damaged, is 0.78Hz (time period 1.28s). Corresponding to third floor damage of the building the identified frequency from the frequency response function plot is 0.549Hz (time period 1.82s). Therefore, time periods of the building corresponding to its two damage states are found to be 1.28s(computed value 1.20s) and 1.82s (computed value 1.90s) respectively. Corresponding time period of the building in un-damage condition is 0.46s (computed 0.50s). Now, the damage can be quantified using Eq. (1) for the two damage states of the building and are found to be 0.64 and 0.74 respectively.

Table 1 : Damage of Building due to Ground motion Type-I

Case study	Floor strength Reduction (%)	Computed Time period (s)	Estimated Time period (s)	Damage index, δ	Location of Plastic hinges
Un-damaged	Not Applicable	0.50	0.46	Not Applicable	Not Applicable
Ground floor Damaged	55.55	1.20	1.28	0.64	Ground floor columns
Third floor Damaged	55.55	1.90	1.82	0.74	Third floor columns

3.2 Damage due to Ground motion Type-II

The ground floor and third floor damage state of the building is now analyzed for low frequency rich Michoacan Earthquake, long duration earthquake, ground motion Type-II (Fig.3(a)).

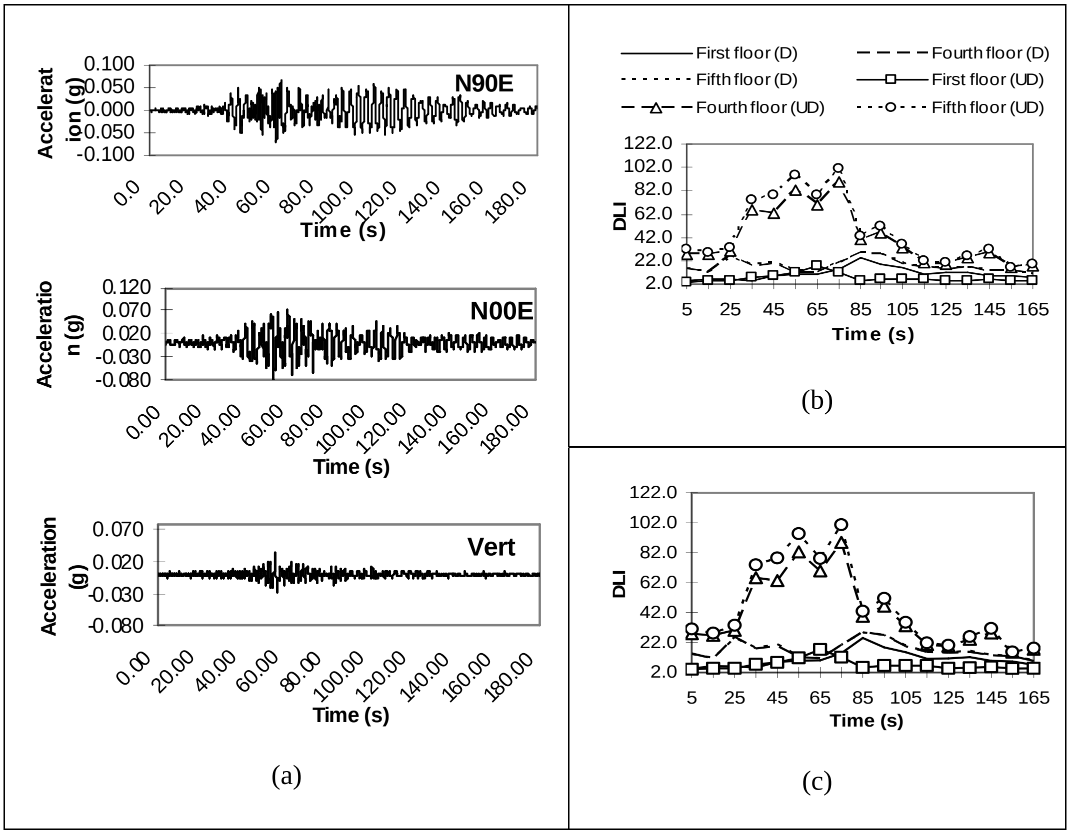

Figure 3: Ground motion Type-II: (a) Time histories of Michoacan Earthquake; (b) Damage Localization at the Ground floor; (c) Damage Localization at the Third floor.

Similar kind of damage is introduced at ground floor and third floor of the building by reducing story strength of 55.55%. The good reproduction of rigid body motion of the building above damaged ground floor is seen. When the ground floor is damaged, the DLI values corresponding to first, fourth and fifth floor responses are overlapped to each other (Fig. 3(b), (c)). Whereas, in un-damage condition the DLI values corresponding to responses at these floors of the building are well separated. Therefore, different nature of ground motion (compare with ground motion Type-I) can represent the same type of building behavior when the building is damaged at ground floor. Similar analyses are carried out when the damage is assumed to be localized at third floor. The corresponding values of damage indices are shown in Table 2.

Table 2: Damage of Building due to Ground motion Type -II

Case study	Storey strength Reduction (%)	Computed Time period (s)	Estimated Time period (s)	Damage index, δ	Location of Plastic hinges
Un-damaged	Not Applicable	0.50	0.48	Not Applicable	Not Applicable
Ground Damaged	55.55	1.20	1.15	0.58	Ground columns
Third floor Damaged	55.55	1.90	1.85	0.74	Third floor columns

3 DISCUSSIONS

Numerical-simulation study for damage analysis due to ground motion type-I and type-II has been carried out for the building. The good indication of rigid body behavior of the building portion above the damaged floors is observed for the two types of ground motions. The analysis shows that the variation of DLI values can reflect the damage condition of the structure. The damage analysis for ground motion Type-I shows that the DLI values corresponding to fifth floor response for damage state are less in comparison to un-damage state. However, for ground motion Type-II, different kind of variation is observed for fifth floor response. It is also observed that characteristic variation of DLIs is possible due to change in ground motion properties. However, the loss of structural integrity can be understood clearly from the curves for intended purpose of study. It is important to note that more number of sensor records would have provided good information on location of damage.

4 CONCLUSIONS

Damage detection has been performed for the five storey step-back building via simulation study. The detection of damage, here, includes localization of damage and quantification. Localization of damage has been carried out using a new method developed in this paper. For quantification of damage existing formula has been used. Damage localization has been carried out using a new term (DLI) derived from the methodology developed. It is found through numerical simulation study that DLI can successfully be used for locating damage at story level. For simulating damage at a particular story level column cross section and material properties (modulus of elasticity and other stress values) have been reduced proportionately such that damage can be localized through plastic hinge formation at that story level. It is important to note that innumerable number of case studies on buildings of different types and configuration has been carried out and it is observed that particular type of building damage behavior is noticed for story strength reduction of 20% and above. It is also important to note that the proposed methodology cannot be used for localization of damage at element level. Two types of ground motion with different characteristics are considered for the study to check the ground motion effects. From the study no remarkable change is observed, as far as the explanation of rigid body motion of the portion of the building above the damaged story is concerned. However, response behavior can change as the damaged building's vibration characteristics may be conducive for generating large motion due to a particular type of ground motion. The developed methodology can be used, even when pre-damage response characteristics are not available. Using this approach the type of building failure occurring can be explained from rigid body motion. Following question can be answered for a building from this study as stated Datta (2001): (a) Is there damage in the building (existence); (b) Where is the damage in the building (location); (*c*) How much damage is there (extent)

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