Seismic Damage Measures for Moment RC Frames

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SUMMARY:
In the seismically active areas, there is a growing demand to estimate the damage level of the structures. Performance-based seismic design concepts are suggested in most codes for such estimation. At that, performance levels of buildings are usually specified in terms of displacements instead of direct damage measures.

This work aims, therefore, to define proper explicit seismic damage indices, which may be used as seismic design criteria for reinforced concrete frames. Most of the existing damage indices are defined locally. Only a few global but usually empirical damage indices are known.

Global and local seismic damage indices are proposed in the present contribution, that are mechanically well-founded and range between 0 (no damage) and 1 (failure). They are suitable for pushover quasi-static and direct time history analysis. The application of the proposed damage measures is illustrated on a test problem by use of the in-house developed nonlinear finite element program FRAME.

Keywords: seismic damage measure, moment reinforced concrete frames, nonlinear dynamic analysis

1. INTRODUCTION

In the earthquake areas, there is a growing demand to estimate the damage level of existing structures that have already experienced earthquakes or new structures that can certainly face new earthquakes in the near future. Most codes of practice for seismic design, evaluation and retrofitting, like ATC 40, FEMA 356, Eurocode 8, Turkish Code DBYBHY-2007, utilize the performance-based seismic design concept for such estimation. Various performance levels are usually specified, typically ranging from collapse prevention to immediate occupancy. Such performance levels are then associated with a limitation of the damage state with respect to a given earthquake. However, either descriptive criteria or displacement limits are used for the damage state instead of direct damage measures. This work aims, therefore, to define proper explicit seismic damage indices, which may be used as seismic design criteria for moment reinforced concrete frames.

Several measures for seismic damage assessment have previously been described and are reasonably well known. A general overview of these measures can be found, for example, in [Yao et al. 1986; Kratzig & Meskouris 1998]. Although some of them reflect purely empiric approaches, such as the inter-storey drift, others refer to the use of the cross-section level, such as the ductility index, stiffness index or the energy index. The latter approach uses the moment-curvature relationship of a beam section. Thus, they are generally not suitable for the entire structure. Some damage measures are empirical in nature and mechanically inconsistent. For example, the damage index by Park & Ang [1985] is widely used, although it combines relative displacement and relative energy in one value. One of the consistent damage measures has been proposed by Hanganu et al. [2002] for the finite element analysis of monotonically loaded structures. Another consistent damage measure has been proposed in [Krätzig & Petryna 2001; Petryna & Krätzig 2005] based on the reduction of structural stiffness. However, the structural stiffness is a multi-dimensional function and several difficulties appear by derivation of a scalar global damage index from a multi-dimensional functional.

Obviously, seismic design needs not only proper local damage indices, but also proper global ones. It is, however, hardly possible to define a global damage measure for all structural types and all structural materials such as reinforced concrete, steel and masonry. The present contribution, therefore, proposes global seismic damage indices only for moment reinforced concrete frames. They are mechanically well-founded and range between 0 (state without damage) and 1 (failure or collapse). They will be suitable for the pushover quasi-static analysis as well as for direct time history analysis.

The application of the proposed damage measure is illustrated on a test problem by use of the nonlinear finite element analysis. For that purpose, the finite element program FRAME has been developed within the Matlab environment [Matlab User Manual 2006] capable of both static and dynamic analysis of reinforced concrete frame structures. At that, structural members are modelled by classical beam elements with rectangular cross-sections. Elasto-plastic bilinear moment-curvature relationships are used as a nonlinear material model including hysteretic behaviour under seismic loading. Spread plasticity model along the length of the element is utilized to assemble the element stiffness matrices. An incremental-iterative solution algorithm with tangent stiffness of cracked members has been implemented for nonlinear static analysis. On the other hand, the Newmark-Beta method with constant average acceleration is applied for the step-by-step time integration of the dynamic governing equations.

2. HYSTERETIC MATERIAL MODEL AND RELEVANT MEMBER MODEL

Pushover quasi-static and response history analyses are usually applied as nonlinear analysis procedures in seismic design. In both procedures, constitutive material equations for reinforced concrete can be defined either as stress-strain or moment-curvature relationships. In the present work, the nonlinear material model of a reinforced concrete member is derived from the moment-curvature relationship of its cross-section depending on the type of loading.

The moment-curvature relationship under monotonic loading can be defined as a piecewise linear diagram containing the characteristic points associated with the cracking, yielding and failure states of the member. In dynamic analysis, the moment-curvature relationship is hysteretic and can be achieved by some theoretical or empirical rules from the moment-curvature relationship under monotonic loading. Several hysteretic models for reinforced concrete have been developed in the past. One of the well-known hysteretic models is the model by Takeda & Sozen [1970], which has been then modified by several authors. Such hysteretic models are usually empirical in nature. A well-known model developed on theoretical observation is the bilinear hysteresis model on the basis of the plasticity theory. According to the plasticity theory, the bending stiffness during elastic loading and unloading is equal to the initial bending stiffness under the monotonic loading as shown in Fig.2.1. The ultimate moment is defined either by the concrete failure under compression or the reinforcement failure under tensile forces. In the present work, this bilinear hysteresis model will be used for the seismic analysis and determination of the seismic damage measures for the moment RC frames.

In nonlinear finite element analysis, the structural stiffness matrix is constituted by assembly of the element stiffness matrices. The latter are based on the bending stiffness of the members which can be determined from the moment-curvature relationship of the relevant cross-sections. Several member models for reinforced concrete can be found in the literature. They are generally subdivided into two groups such as one-component and multi- component models. In a multi-component model, the element is considered as a system of different multi elements acting in parallel. While one element behaves only elastically, the other behaves plastically. In a one-component model, the member consists of only one part which includes all the properties. In this model, the plastic behaviour is considered in two different ways. In the first one, plastic deformations are assumed to occur only at the end of the member. This model is called lumped inelasticity model. However, it is more realistic to consider plasticity as spread deformation within the member. In this work, the spread plasticity model by Soleimani et al. [1979] is implemented, as shown at Fig.2.2.

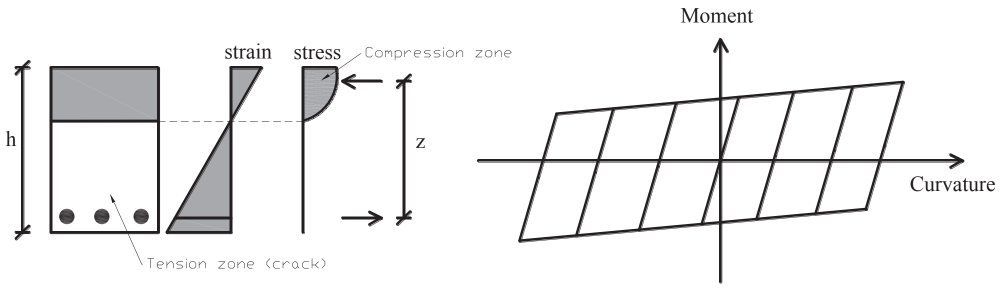

Figure 2.1. Hysteretic material model

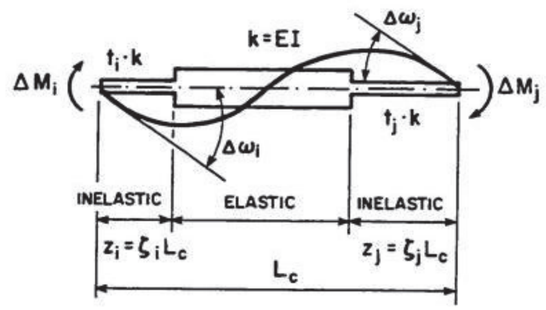

Figure 2.2. Spread plasticity member model

3. SEISMIC DAMAGE MEASURE

The damage in material is usually defined as softening of its stiffness and its extent can be quantified as the relative reduction of its actual modulus of elasticity (E_x) with respect to an initial value in the non-damaged state (E_0) and the critical value (E_f) at failure:

$$D = \frac{E_0 - E_x}{E_0 - E_f} . \qquad (3.1)$$

The obvious advantage of this damage index is that it is a scalar and ranges from 0 (state without damage) to 1 (failure).

A similar definition can be applied to structural members by use of proper stiffness values. For example, the reduction of the bending stiffness EI of the beam elements can serve as damage measure defined on the cross-sectional level:

$$D = \frac{(EI)_0 - (EI)_x}{(EI)_0 - (EI)_f} . \qquad (3.2)$$

These damage measures are local but seismic design and evaluation need seismic global damage measure for the entire structure. Nevertheless, due to the high degree of uniqueness of civil engineering structures, the development of general damage measures, suitable for diverse structures and damage mechanisms, is still a challenge [Yao et al. 1986]. Krätzig & Petryna [2001] showed that a scalar global damage index can be quantified by the reduction of the structural stiffness represented by some scalar characteristic values λ:

$$D = \frac{\lambda_0 - \lambda_x}{\lambda_0 - \lambda_f} \,. \tag{3.3}$$

However, the choice of proper scalar values λ in Eqn. 3.3, that correlate well with the overall reduction of the structural stiffness, is a challenge. It is more suitable to define a damage index based on scalar deformation energy. Hanganu et al. [2002] proposed to define a global damage index as the relative reduction of the whole structural deformation energy (W_p) with respect to an initial value in the non-damaged state (W^0_p):

$$D = \frac{W_p^0 - W_p}{W_p^0} = 1 - \frac{W_p}{W_p^0} \,. \tag{3.4}$$

Here, the global damage index is a scalar and varies between 0 (no damage) and 1 (failure state). By use of the finite element simulation, this energy-based global damage index can be calculated from the actual displacement field as follows:

$$D = 1 - \frac{W_p}{W_p^0} = 1 - \frac{\sum_e a^T \int_{V_e} B^T \sigma dV_e}{\sum_e a^T \int_{V_e} B^T \sigma_0 dV_e} \,. \tag{3.5}$$

where a stands for the nodal displacement vector, B for the strain shape matrix and V_e for the volume of each finite element e. Additionally, σ represents the actual stress vector and σ_0 the stress vector calculated for the undamaged material and the same global displacement a. Herein, the total potential energy W_p as well as W_p^0 is accumulated over all finite elements.

However, the local and global damage indices mentioned above are suitable for the monotonic loading and can be used for example in the push-over analysis. Damage indices suitable for dynamic analysis shall be defined in another way. Several empirical local damage indices for dynamic analysis have previously been proposed. Meyer [1988] proposed a theoretical local cyclic damage index by using the absorbed energy calculated from the moment-curvature relationship. At first, the damage indices for the positive and negative directions are separately calculated. Then, the cyclic local damage index is defined by their superposition as follows

$$D = D^+ + D^- - D^+ \cdot D^- \,. \tag{3.6}$$

This local damage index is defined on the cross-sectional level and cannot be used for the damage state of the entire structure. Nevertheless, the idea of defining two separate damage indices for positive and negative directions can be combined with the monotonic energy based global damage index [Celik & Petryna 2009]:

$$D_G^{\pm} = 1 - \frac{\sum_e (a^{(e)T} - a_0^{(e)T}) \cdot K_d^{\pm(e)T} \cdot (a^{(e)T} - a_0^{(e)T})}{\sum_e (a^{(e)T} - a_0^{(e)T}) \cdot K_0^{\pm(e)T} \cdot (a^{(e)T} - a_0^{(e)T})} \tag{3.7}$$

Finally, the dynamic global damage index can be then obtained in a similar way from the superposition of the two separate global damage indices:

$$D_G = D_G^+ + D_G^- - D_G^+ \cdot D_G^- . \tag{3.8}$$

4. EXAMPLE

The new proposed global dynamic damage measure is applied on a five-storey reinforced concrete frame. The frame is 12.5 m high and 10 m wide as shown at Fig.4.1. The reinforcement and cross-sectional properties are depicted at Fig.4.1 and the material properties are given in Table 4.1.

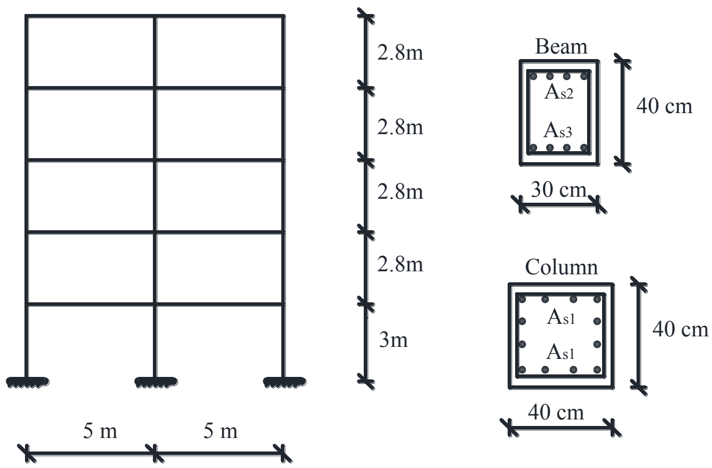

Figure 4.1. Geometry, reinforcement and cross-sections of the reinforced concrete frame

Table 4.1. Material properties of the reinforced concrete frame

Elasticity modulus of steel	E_s= 200000 MN/m^2
Elasticity modulus of concrete	E_c= 31000 MN/m^2
Yielding strength of steel	f_{sy}=500 MN/m^2
Ultimate strength of steel	f_{su}= 502.5 MN/m^2
Compressive strength of concrete	f_{cy}= 21 MN/m^2
Reinforcement area in column	A_{s1}= 10.18 cm^2
Reinforcement area in beam	A_{s2}= 4.52 cm^2
Reinforcement area in beam	A_{s3}= 8.04 cm^2

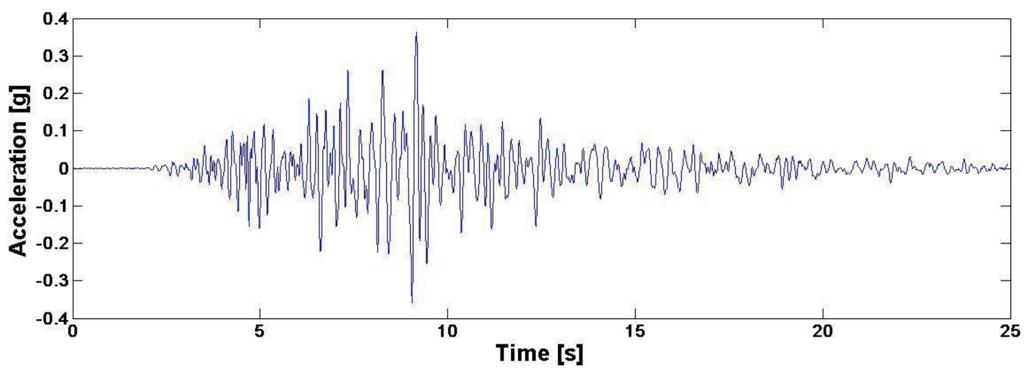

Figure 4.2. Accelerogram of the Kobe earthquake, Japan, 1995

The reinforced concrete frame is subjected to Kobe 1995 earthquake, whose accelerogram with a time step dt = 0.01 s is shown at Fig.4.2. Each beam and column of the structure is discretised by five finite elements. The Newmark-Beta method with constant average acceleration is applied with a time step dt = 0.002 s for the time integration of the dynamic governing equations.

The horizontal displacement of the top storey of the structure, as calculated during the earthquake, is given in Fig. 4.3. The maximum value reaches 35 mm. As can be seen, there is a residual inelastic displacement of 7 mm.

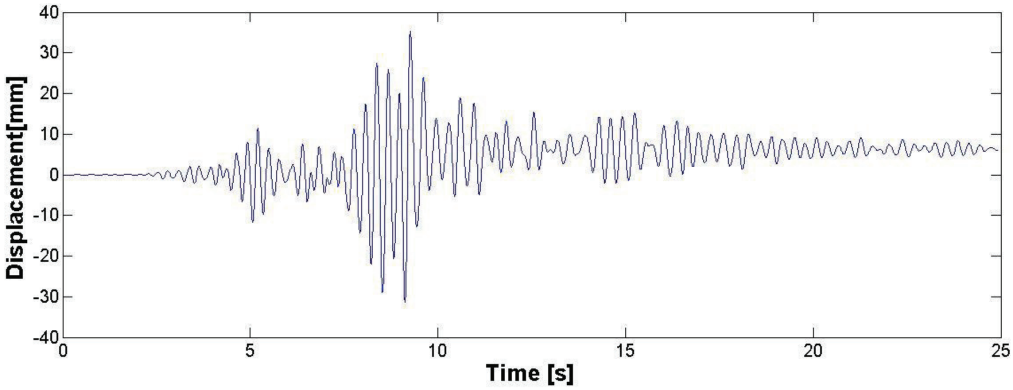

Figure 4.3. Displacement-time diagram for the horizontal top displacement

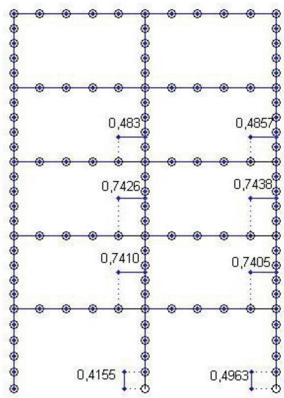

Figure 4.4. Local damage index distribution along the entire structure at the end of seismic loading

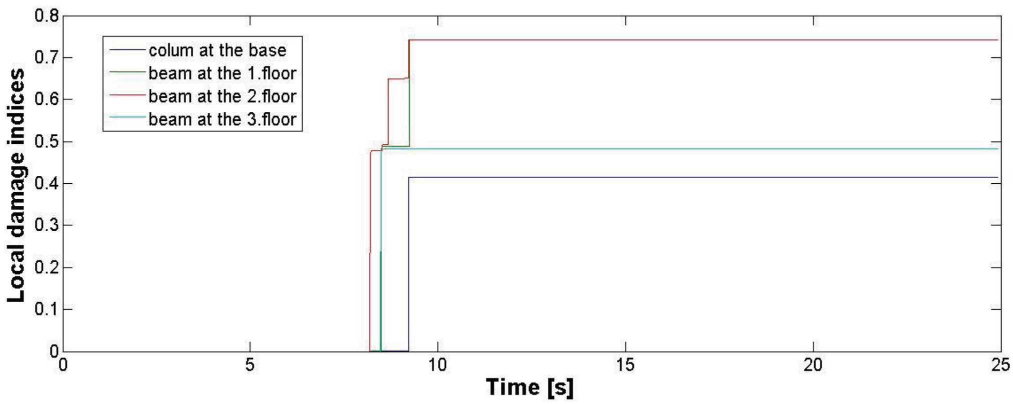

Figure 4.5. Evolution of the local damage indices of selected elements during earthquake

The evaluation of the global damage index of the structure and local damage indices of all 125 elements is performed by use of the nonlinear time history analysis. Fig 4.4 shows the local damage index distribution of the elements along the entire structure at the end of seismic loading. The mostly damaged elements are from the beams at the 1st and 2nd floor. Some elements of the beams at the 3rd floor and the elements at the base of the columns are also damaged. The local damage measures at the base of the columns are generally smaller than those of the damaged beam elements at the 1st and 2nd

floor. This result stands in a good agreement with the desired behaviour of the ductile structures under seismic loading. Accordingly, the plasticity should occur in the beam elements earlier than in the column elements, in order to provide structural safety. This can be also seen from Fig. 4.5 where damage indices of some elements of beams at 1st, 2nd and 3rd floor and base column are displayed.

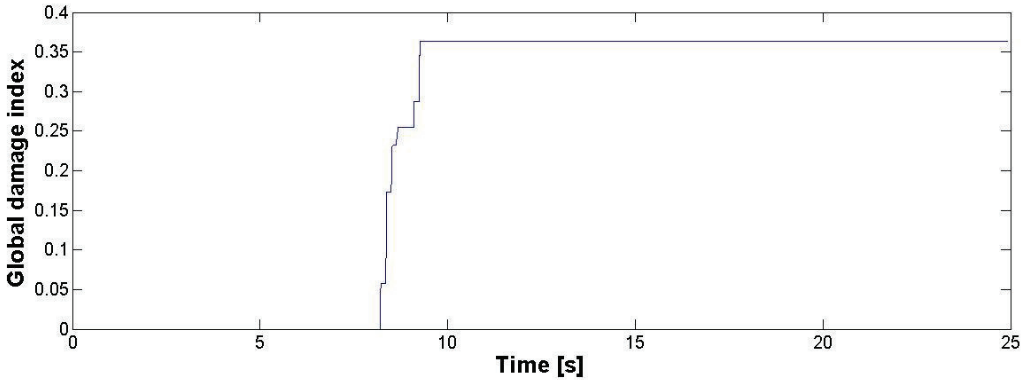

Figure 4.6. Evolution of the global damage index during the earthquake

Fig 4.5 shows also that the value of the local damage indices of the beam elements in the upper stories is smaller than those in the lower stories. That agrees well with the real structural behaviour. Global damage index of the entire structure during earthquake is also calculated and depicted in Fig 4.6. The value of global damage index at the end of seismic loading is also smaller than that of the local ones, since it reflects the state of the entire structure. The value of the global damage index strongly increases when the damage at the base of column elements and 1st floor beam elements spreads. Fig 4.2, Fig 4.5 and Fig 4.6 show also that this coincides with the time point where the maximum amplitude of the earthquake loading is reached.

5. CONCLUSION

Seismic damage measures have been proposed for reinforced concrete moment frames. This damage measure has been tested on a selected frame subjected to seismic loading. The proposed local and global damage measures stand in good agreement with a realistic behaviour and can be generally used in seismic design. However, additional investigations are necessary to calibrate these measures.

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