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SPECTRAL FUNCTIONS OF RC FRAMES USING A NEW FORMULA FOR DAMAGE INDEX

Marijana Hadzima-Nyarko, Dragan Morić, Martina Španić

Preliminary notes
Structural damage under seismic force is quantified by the calculation of damage index (*DI*), a coefficient which represents the degree of damage of the structure, and typically ranges from 0 to 1, with the value of 1 representing collapse. Based on a few specified damage models, a new original deterministic declaration of the DI is presented. Damage Index spectral functions are performed by an extensive parametric study using different earthquakes and different structures modelled as SDOF systems. A detailed analysis of the dynamic properties of reinforced concrete (RC) frame structures, as well as post elastic parameters of vertical structural elements using a large number of available experiments is also carried out, thus relating the parameters of real buildings and seismic loads defined by peak ground acceleration to the DI coefficients of structures. The results are seismic damage spectrum functions of RC frame structures.

Keywords: *Damage Index (DI), earthquake resistance, RC frames, spectral functions*

Spektralne funkcije oštetljivosti armiranobetonskih okvirnih konstrukcija uporabom nove formule za koeficijent oštetljivosti

Prethodno priopćenje
Kvantifikacija konstrukcijskog oštećenja moguća je proračunom koeficijenta oštetljivosti (*DI*), koji predstavlja stupanj oštećenja konstrukcije i kreće se u granicama od 0 do 1, gdje vrijednost 1 predstavlja rušenje. Zasnovan na nekoliko specificiranih modela, dan je novi originalni deterministički izraz za koeficijent oštetljivosti (DI). Spektralne funkcije koeficijenta oštetljivosti izvedene su parametarskom studijom koristeći 20 različitih potresa i različite konstrukcije predstavljene SDOF modelima. Provedena je detaljna analiza dinamičkih karakteristika okvirnih konstruckija, kao i poslijeelastičnih parametara vertikalnih konstrukcijskih elemenata koristeći se velikim brojem eksperimenata skupljenim u bazu, povezujući na taj način parametre stvarnih zgrada i seizmičkog opterećenja, definiranog vršnom akceleracijom, s koeficijentom oštetljivosti konstrukcije. Rezultat navedenoga su spektralne funkcije okvirnih armiranobetonskih konstrukcija.

Ključne riječi: *armiranobetonski okviri, koeficijent oštetljivosti, potresna otpornost, spektralne funkcije*

1 Introduction

During its lifetime, a structure is subjected to loads originating from different sources. Extreme loads, such as earthquakes and hurricanes, may generate stresses and deformations on the structural members that can be so high as to cause damage or even a failure of members, or the whole structure.

One of the important areas of research, over the last few decades, has been the characterization and evaluation of structural damage. Quantifying damage often represents a difficult problem. Different methods have been developed to provide reliable predictions of the state of a damaged structure. Damage assessment investigates the potential or actual degradation state of the structure. Damage assessment techniques have been applied in different situations, such as structural assessment, retrofit and repair operations, maintenance inspections and post-earthquake evaluation.

Among the different approaches to characterize damage, damage indices are suitable tools for numerically quantifying the damage in structures sustained under earthquake loading or rank their vulnerability relative to each other. Damage indices can be determined either based on the response of the structure to a particular loading pattern or based on the dynamic response of a structure. Damage index is a mathematical model for the quantitative description of the damage state of the structures and in most cases it is in correlation with the actual damage in earthquakes. In economic terms, this coefficient represents the ratio of funds needed for the rehabilitation of structures damaged by an earthquake and the resources necessary for the construction of an identical structure.

Based on some known damage models, a new original deterministic declaration of the damage index, DI, is presented, where the structure is represented using a single degree of freedom (SDOF) model, the earthquake is modelled as time history of ground motion, and numerical analysis is performed using nonlinear time interval analysis [1].

An extensive parametric study is further performed using different earthquakes and different structures, which are classified using natural period, elastic base shear capacity, post-elastic stiffness and damping. Each of these structures is subjected to nonlinear seismic time history analysis using different real earthquakes having peak accelerations ranging from 0,1*g* to 1,35*g*. *DI* values in relation to the period, base shear capacity, post-elastic stiffness and damping are then implemented in seismic damage spectrum functions [2]. The relationship between the equivalent SDOF model and corresponding seismic damage spectrum, which is associated with a *DI* value that also depends on the ground motion of a chosen earthquake, will provide the level of damage of the SDOF model and, inherently, the level of damage of that structure. An investigation of the following parameters of RC structures: period of vibration, damping, elastic capacity and post elastic behaviour is necessary to determine an equivalent SDOF model and, eventually, to predict their seismic response. Hence, a detailed analysis of the dynamic properties of RC frame structures, as well as post elastic parameters of their structural elements using a large number of available databases of experiments is also carried out, thus relating the parameters of real buildings, seismic loads defined by peak ground acceleration and damage indices of structures. The results are seismic damage spectrum functions of RC frame structures, which provide an insight into the level of physical deterioration (degradation) of a frame structure and perform analyses of the damage level before and after an earthquake.

2 Damage indices of RC structures

When subjected to an earthquake, a structure might suffer excessive deformations, causing structural damage in individual members or parts of the structure. The nature and amount of structural damage depends on the quality of the materials that compose the structural and non-structural elements, on the configuration and type of structural systems and on the nature of the loads acting on the structures. Global indices quantify damage for the whole structure or for parts of the structure when several of its structural elements are considered. They provide an overall assessment of structure performance based on damage distribution and level of degradation sustained by its individual components.

Several approaches as well as critical reviews for structural damage evaluation have been proposed in literature and reports [3, 4]. Depending on how they are defined, damage indices can be categorized as deterministic or probabilistic indices [5, 6], structural or economic indices [7, 8], structural or non-structural indices [7]. Other categorizations include indices based on deformation, stiffness, or energy, or even a combination of two or more of them, noncumulative (i.e. peak response values) or cumulative indices, low-cycle versus high-cycle fatigue indices, global indices as a weighted average of local indicators or modal indices, etc. [9].

A graphical presentation of existing damage indices is given in Fig. 1.

Damage can be described as the level of physical degradation with precise defined consequences to residual capacity of resistance and deformations, where a specific level of damage without any capacity of resistance and deformations means failure [10].

As previously mentioned, damage indices in various models can be based on maximum values of structural response parameters or cumulative values and summing nonlinear deformation cycles, such as Park and Ang [11], who define DI as a linear combination of plastic deformation (ductility) and energy dissipation. The DI developed by Park and Ang for RC structures attempts to account for the damage caused by cyclic deformations into the post-yield level. They define DI as a linear combination of the damage caused by excessive deformation, and repeated cyclic loading, captured in the form of dissipated energy. The general form of the Park-Ang damage formulation is as follows:

$$DI = \frac{\varDelta_i}{\varDelta_u} + \frac{\beta}{P_\mathrm{Y} \cdot \varDelta_u} \int \mathrm{d}E, \tag{1}$$

where:
$\varDelta_i$ – is the peak deformation,
$\varDelta_u$ – is the ultimate deformation capacity under monotonic loading,
P_Y – is the calculated yield strength (the smaller value of the yield strength or the ultimate strength),
$\mathrm{d}E$ – is hysteresis dissipated energy,
β – is a constant which depends on the structural characteristics and controls the strength degradation in correlation with the dissipated energy.

The Park and Ang DI can be calculated both at the element level and the global level (obtained by combining the weighted values of element level damage indices).

The Mizuhata and Nishigaki model [12], similar to Park's model, defines DI as a linear combination of plastic deformation (ductility) and energy dissipation as the result of maximum deformation, failure deformation under monotonic load and a number of real cycles which leads to failure:

$$DI = \frac{|\Delta_{\max}|}{\Delta_u} + \sum_{i=1}^{k} \left(\frac{n_i}{N_{fi}} \right)^{0,91} \cdot \left(1 - \frac{\Delta_i}{\Delta_u} \right), \tag{2}$$

where:
$|\Delta_{\max}|$ – is the maximum displacement,
Δ_u – is the collapse displacement,
n_i – is the number of cycles (with displacement range Δ_i) actually loading,
N_{fi} – is the number of cycles (with displacement range Δ_i) to failure.

Hwang and Scribner [13] developed a model which contains Gosain's energy index [14] normalized by dissipated energy, stiffness and maximum displacement in the i^{th} cycle, also with initial stiffness, yield displacement and a number of cycles in which P_i>0,75P_Y:

$$I_\mathrm{e} = \sum_{i=1}^{n} E_i \cdot \frac{K_i}{K_\mathrm{e}} \cdot \left(\frac{\Delta_i}{\Delta_\mathrm{Y}} \right), \tag{3}$$

where:
E_i – is the energy dissipated in the i-th cycle;
K_i – is the secant stiffness of the i-th cycle;
K_e – is the initial stiffness;
Δ_i – is the maximum deformation in the i-th cycle;
Δ_Y – is the yield deformation;
n – is the number of cycles for which is $P_i \geq 0,75 \cdot P_\mathrm{Y}$.

A new original deterministic declaration of DI, given in [1], is based on some essential characteristics of the previously described damage models. Morić, Hadzima, and Ivanušić [1] propose that the seismic response analysis of regular structures is acceptable if it is done as a simplified non-linear dynamic analysis with the time history function of ground motion as input load, and an SDOF model with known weight, elastic stiffness, damping, elastic base shear capacity and post-elastic stiffness representing the structure. The DI coefficient is defined as a linear combination of plastic deformations, stiffness degradation and energy dissipation of a structure during an earthquake:

$$DI = \frac{1}{30} \left[D + \Delta K + \sqrt[3]{\frac{N_\mathrm{Y} E_\mathrm{H}}{W}} \right], \tag{4}$$

where:

$D = u_{\max} / u_{Y}$ is the displacement ductility demand ($u_{\max}$ – maximum top displacement, u_{Y} – yield displacement);
$\Delta K = K_e/K'$ is the relative degradation of stiffness at the end of the earthquake;
$K_e = BS_Y/u_Y$ is the initial structure stiffness (BS_Y – elasticity limit base shear);
$K' = BS_{\max}/u_{\max}$ is the residual secant stiffness of a structure after an earthquake ($BS_{\max}$ – maximum base shear force);
N_Y is the number of yield excursions reached during the earthquake;
E_H/W is the hysteresis energy per unit of structure mass, dissipated during an earthquake.

- Damage Models
  - 1-*Local Damage Indices*
    - 1-*Non - Cumulative Indices*
      - 1-Ductility Ratio (DR)
      - 2-Interstory Drift (ID)
      - 3-Slope Ratio (SR)
      - 4-Flexural Damage Ratio (FDR)
      - 5-Maximum Permanent Drift (MPD)
      - 6-Pushover-Based Damage Index
    - 2-*Cumulative Damage Indices*
      - 1-*Deformation - Based Models*
        - 1-Normalized Cumulative Rotation (NCR)
        - 2-Cumulative Displacement Ductility
        - 3-Normalized Cumulative Plastic Deformation
        - 4-Maximum Deformation-Based Model
        - 5-Low-cycle Fatigue Model
      - 2-Energy-Based Models
      - 3-*Combined Models*
        - 1-Park-Ang Damage Index
        - 2-Bracci et al. Damage Index
  - 2-*Global Damage Indices*
    - 1-Weighted Average Indices
    - 2-*Modal - Based Indices*
      - 1-Roufaiel-Meyer Global Model
      - 2-Maximum Softening
      - 3-Final Softening
    - 3-*Financial Indices*
      - 1-Ratio of the repair cost to the replacement cost
      - 2-Gunturi-Shah Financial Index

Figure 1 Classification of damage indices [9]

The importance of the *DI* in the damage model is to describe the condition of a structure after an earthquake. In such an approach, the *DI* value can be used to declare a decreased residual earthquake resistance and increased residual damping coefficient of a structure [1].

There are various ways to categorize the damage indices and the simplest is to correlate the damage indices and observed damage. For example, in [11] and [15] structural damage is classified as: None, Minor, Moderate, Severe and Collapse. Similarly, in [16], the following categorization is defined: Undamaged or minor damage, Repairable, Irreparable, Collapsed. In [1], the *DI* values are implemented in pre and post earthquake damage analysis by relating the *DI* values with the values of damage level identification (S) [18], defined in the Croatian codes for post disasters damage assessment (Tab. 1). According to Aničić et al. [17], buildings classified within the first three damage levels do not pose any threat to users. The repair or reconstruction of such damaged buildings to their prior state of earthquake resistance is considered acceptable and they can still be used after minor repairs. Buildings classified as level four or five are seriously damaged and need to be evacuated. Structural rehabilitation (reconstruction, strengthening) of such damaged buildings should be done based on technical documentation created using standard procedure. Buildings classified as level six are unusable and cannot be rehabilitated. A similar classification is applied in this paper.

The validation of the proposed formula for *DI* was done by comparing the obtained results with those of the CAMUS3 experiment, done by the Camus working group, in TRM-ECOEST 2 Research programme in EMSI Sacley, France [1, 19].

Damage index spectral functions were obtained using different earthquakes and different structures. All the structures were modelled as an SDOF system using defined weight (W), damping (ξ), elastic stiffness (K_e), yield base shear (BS_Y) and post-elastic stiffness (K_2):

- For all SDOF systems a constant weight W = 1000 kN is assumed.
- Damping is defined as 2 %, 5 % and 10 % of critical (3 structure conceptions).
- Variation of the elastic stiffness is a function of the basic period of the system representing real regular structure. Elastic stiffness is modified in such a manner as to realise a change in basic period in steps of 0,1 s, from 0,05 s to 10 s (15 structure conceptions).
- Yield base shear defines the yield point and the end of elastic stiffness and it is modified in ten levels, from $0{,}1W$ to $1{,}0W$ (10 structure conceptions).
- Post-elastic stiffness, which represents residual stiffness after yield point is reached, is modelled as a

percentage of initial elastic stiffness. Variations of this parameter are done, also, in five steps: K_2 = 0,00; K_2 = 0,2K_e, K_2 = 0,4K_e, K_2 = 0,6K_e and K_2 = 0,8K_e (5 structure conceptions).

Taking into account all the combinations of the structure parameters defined above, 2250 various structures were obtained. Each of these structures was subjected to nonlinear seismic time history analysis using 20 different real earthquakes. All calculations were run using the program NONLIN [20], which implements step by step time-history numerical integration. The results of provided analysis are: time-history of top displacements with FFT analysis in frequency domain, base shear–displacements hysteresis response, yield excursions and cumulative energy transformation. These structure response parameters were then input into the new original deterministic declaration of *DI* (4).

Table 1 Physical interpretation of damage index (*DI*)

Damage index (*DI*)	Structural damage description	Possibilities of technical and economic reparation	Code damage level (S) (1° to 6°)
0 ≤ *DI* ≤ 0,3	insignificant	repairable	1° - 2°
0,3 < *DI* ≤ 0,5	moderate	repairable	3°
0,5 < *DI* ≤ 0,8	severe	repairable	4°
0,8 < *DI* ≤ 1,0	heavy	repairable	5°
1,0 < *DI*	extremely high level or collapse	non-repairable	6°

Fig. 2 presents the obtained *DI* values for a given period of an SDOF model with the following parameters: BS_Y=0,3W, K_2=0,2K_{el} and ξ=2 % for the accelerograms ranging from 0,1*g* to 1,35*g*. Depending on their maximum peak acceleration, the earthquakes can be grouped into 4 main groups:

- *Weak* earthquakes - maximum peak acceleration less than 0,15*g* (green colour);
- *Moderate* earthquakes - maximum peak acceleration between 0,15*g* and 0,24*g* (blue colour);
- *Strong* earthquakes - maximum peak acceleration between 0,25*g* and 0,35*g* (red colour);
- *Catastrophic* earthquakes - maximum peak acceleration greater than 0,6*g* (black colour).

Using the spectral damage functions in Fig. 2, one can determine the possible value of *DI* for a given period of the defined structure. Since only a given set of values of the input parameters (level of post-elastic stiffness, the yield base shear and damping) describes RC frame structures, additional analysis was performed using a database of RC columns under cyclic loading in order to determine these values.

3 Post-elastic stiffness of the column specimens

The collapse of a building or bridge is normally caused by the failure of a major vertical load-carrying element, e.g. a column in the case of frame structures. Semi-empirical and empirical approaches have usually been used for the quantification of the deformation capacity of RC columns [16, 21]. A database that contained the results of tests of rectangular cross-section RC columns loaded with standard cyclical testing procedure displayed in the form of load-displacement curve was created using data from two publicly available databases, PEER and Kawashima Laboratory:

a. PEER database (http://www.ce.washington.edu/~peera1) provides the results of over 400 cyclic, lateral-load tests of RC columns. The database describes tests of: spiral or circular hoop-reinforced columns (with circular, octagonal or rectangular cross-sections), rectangular reinforced columns and columns with or without splices. The database provides the column geometry, column material properties, column reinforcing details, test configuration, axial load, digital force-displacement history at the top of the column, top displacement that preceded various damage observations, references etc., for each test.
b. The database from the Kawashima Earthquake Engineering Laboratory of Tokyo Institute of Technology (http://seismic.cv.titech.ac.jp/index.html) contains details of 107 tests of rectangular RC columns and spiral columns. The geometry of column, material, reinforcement, and characteristics of loads and test results are given for each test.

A subset of the specified databases is used in the study performed in this paper. In establishing this reduced database, the following criteria were considered, referred to herein as "RC_ColumnsDatabase":

Only rectangular columns satisfying all the following conditions were considered:

- Columns with applied axial load were considered;
- Columns with complete data, e.g. known geometry, column material properties and column reinforcing details were used.
- Columns subjected to standard cyclic testing procedure were considered.

Considering the above criteria, 265 test specimens remained in the database, and were used in the study. Since NONLIN [20] implements bilinear force-deformation relations, post-elastic stiffness was determined using the yield point and maximum displacement for each specimen. The secondary stiffness is the first of two properties required for nonlinear analysis. The secondary stiffness is the slope of the post-yielding portion of the force-displacement response of a structure. This is also known as post-yield or post-elastic stiffness (K_2). This defines the nonlinear response and the behaviour of the system in the hysteretic cycles under loading and unloading. K_2 is always less than the initial stiffness (K_{el}). The value may be positive, representing strain hardening, or zero, representing an elastic-perfectly plastic response.

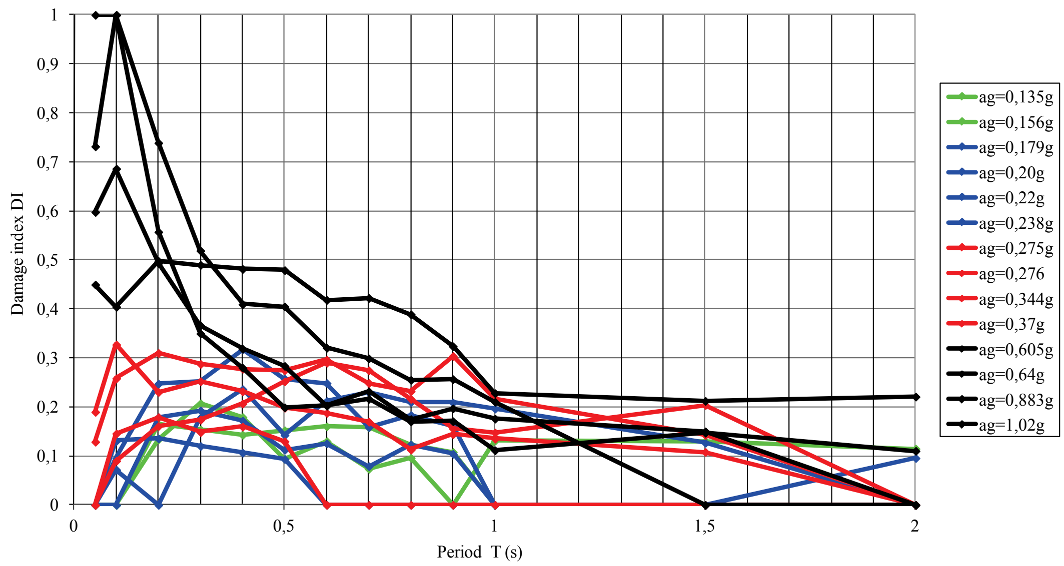

Figure 2 Spectral damage functions for an SDOF model defined by the following parameters: $BS_Y = 0{,}3W$, $K_2=0{,}2K_{el}$, $\xi= 2$ %

An RC specimen with positive post-elastic stiffness is displayed in Fig. 3.

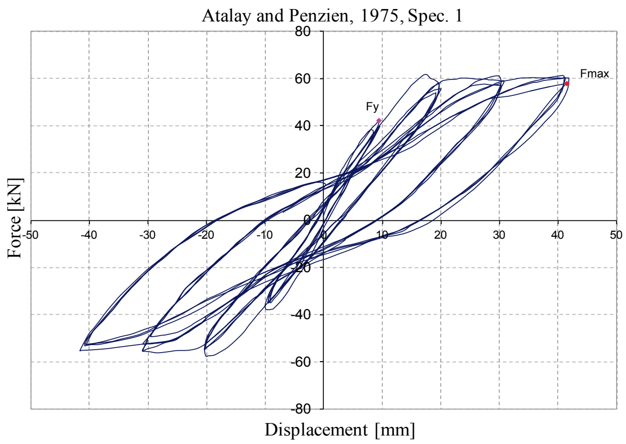

Figure 3 RC specimen with positive post-elastic stiffness

Results for all specimens included in "RC_ColumnsDatabase" are given in Fig. 4.

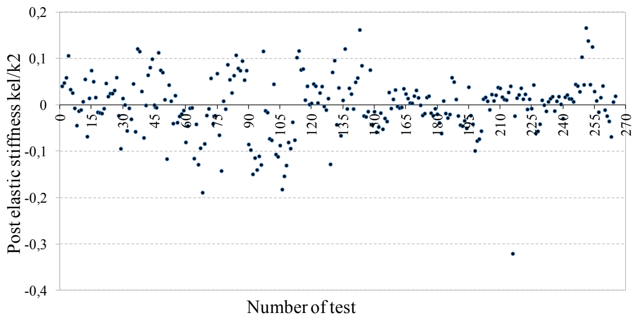

Figure 4 Post-elastic stiffness of all RC columns specimens

It can be concluded that, looking at the results for all 265 specimens given in Fig. 4, the greatest post-elastic stiffness values are less than 20 % of the initial stiffness. The columns behaved as if they either had no residual stiffness (all samples with negative post-elastic stiffness) or developed a small post-elastic stiffness (about 10 to 17 % of the initial stiffness).

4 Yield base shear of RC frame structures

Defining the yield base shear is one of the most important steps in seismic design. It determines the point of yielding and the end of the elastic earthquake resistance. The post yield structural response is of major interest to the structural designer since the building would experience damage when the yield base shear capacity of an idealized bilinear system is exceeded. The yield capacity of a building is defined as the lateral force required causing the yielding of the most rigid lateral force-resisting element in the building. It depends on the failure mechanism and potential absorption capacity of nonlinear deformation. In practice, it is expressed with respect to the weight of the building. After computing the weight and yield base shear of the building, the base shear coefficients are then computed by dividing the yield base shear by the weight of the building.

A parametric study was performed in [2] which contains seismic damage spectrum functions for structures with elastic earthquake resistance expressed by yield base shear, modified in ten levels, from $0{,}1W$ to $1{,}0W$. For example, structures with $0{,}1W$ represent elastic structures with low earthquake resistance; structures with $0{,}3W$ are elastic structures with a high earthquake resistance, while structures with $0{,}6W$ have extremely high elastic earthquake resistance.

The yield displacement, and consequently the point which determines yield base shear, was determined using the given force-displacement curves of all test results. Since the database described in Section 3 contained columns with known axial force, N, it was possible to express the yield base shear as BS_Y/N. A subset of 207 samples having transverse reinforcement, referred to as "BS_Y_ColumnsDatabase", was taken from "RC_ColumnsDatabase" in order to investigate the influence of longitudinal reinforcement, transverse reinforcement, axial force and quality of materials on the yield base shear.

Ratios of longitudinal reinforcement and transverse reinforcement, axial force, quality of materials and geometry of the specimen affect the yield base shear but the dependence of yield base shear on these parameters is difficult to express in mathematical form. Therefore, a multilayer perceptron (MLP) neural network was used to model this dependency. Using similar methods as in [22], a sensitivity analysis procedure was performed using the trained neural network to determine the influence of these pre-defined parameters on the value of yield base shear [23]. The sensitivity analysis procedure consisted of analysing the predictive importance of different combinations of the input parameters. The greater the increase in the generalization error of the neural network obtained by omitting an input parameter, the more important the parameter. The results indicate that the most important parameter is the normalized axial load followed by longitudinal reinforcement ratio while the least important parameter was the transverse reinforcement ratio. Based on these results and by performing regression analysis using genetic algorithm on the data (Fig. 5), an expression relating the yield base shear to the normalized axial load was determined:

$$BS_{\mathrm{Y}} = 0{,}116 \cdot \mathrm{e}^{-1{,}39 \cdot \nu} + 0{,}476 \cdot \mathrm{e}^{-10{,}70 \cdot \nu}, \tag{5}$$

where ν is normalized axial load.

The data in the database depicting the relationship between the yield base shear and normalized axial load, as well as a graphical representation of (5) for the determination of yield base shear of RC frame structures are shown in Fig. 5. Several expressions were assumed in [23] and, based on the mean squared error (MSE) for the entire data set, the expression given by (5) had the least MSE. The assumed expression was obtained for the collection of experimental data and its accuracy depends on the results of experiments.

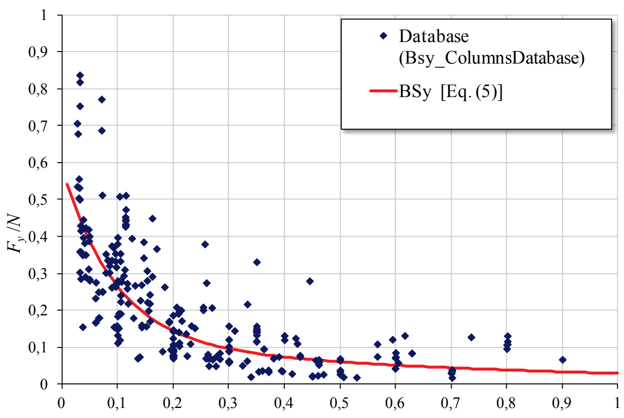

Figure 5 Database data and graphical representation of (5) for the determination of yield base shear of RC frame structures

One of the aims of the research results given in [23] was to propose a simple expression for the yield base shear. Analysis showed that the yield base shear depends mostly on one parameter, the normalized axial load. This parameter can be chosen by the designer through the section size and column tributary floor area, but for the majority of the existing buildings in seismic environments which do not satisfy modern code requirements, analysis shows that its value is greater than 0,3. This conclusion was arrived at by performing the analysis explained as follows.

The normalized axial load of RC frame structures was determined using a database of 600 different models of RC frame structures, each with a rectangular plan shape and moderate number of storeys [24]. The longitudinal length (L_x), transversal length (L_y) and the global height (H) excluding the foundation were the considered variable parameters of a given structure model. A 3D space frame model with the length of bay of 5,0 m in both longitudinal and transversal directions was the basic layout model, which represents a lateral load resisting system consisting of MRF RC frames in both the longitudinal and the transversal directions. Interstorey height was constant and equal to 3,0 m. The building dimensions considered were as follows:

- longitudinal length: L_x = [5,0; 10,0; 15,0; 20,0; 25,0; 30,0; 35,0; 40,0; 45,0; 50,0] m;
- transversal length: L_y = [5,0; 10,0; 15,0] m;
- building height (H) was between (3,0 ÷ 30,0) m corresponding to 1 ÷ 10 storeys.

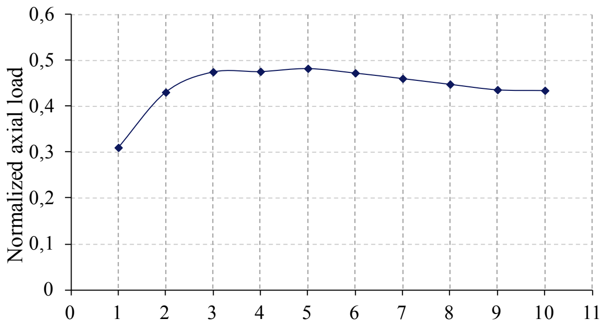

Figure 6 Normalized axial load of the interior columns of an RC frame model

The self-weight of the structural elements and a live load of 2 kN/m^2 were taken as the construction load in the models. The following material requirements of EC8 was considered: a concrete class lower than C20/25 shall not be used in primary seismic elements for ductility class DCH (high ductility class). Therefore, the concrete class C25/30 was used, implying that the cylindrical compressive characteristic strength of concrete $f_{\mathrm{c,cyl}}$ is constant among the structures and equal to 25 N/mm^2. In accordance with the necessary requirements given in EC8 [25] were modelled the dimensions of cross sections of all elements of the structure. All cross sections in the beams had the same 25 cm basis and a height of 45 cm, which satisfy geometrical constrains of EC8 (the width of primary seismic beams shall not be less than 200 mm and requirement which take advantage of the favourable effect of column compression on the bond on the horizontal bars passing through the joint). Two datasets of modelled structures were created:

- the cross sections of the columns of the first dataset increased from 25/25 for one storey to 70/70 cm for 10 storeys,
- the cross sections of the columns of the second dataset increased from 30/30 to 75/75 cm for 10 storeys.

The axial force in columns was known for each model in the database. Since the internal columns have larger values of axial forces, they were taken as relevant.

The values obtained for the normalized axial load of interior columns of a great number of randomly selected RC frame models from the database were greater than or equal to 0,3. For a randomly selected model, the normalized axial load is shown in Fig. 6.

Since the normalized axial load was at least 0,3, it can be concluded, using (5) and Fig. 5, that the values of the yield base shear of RC frames is at most 0,1W.

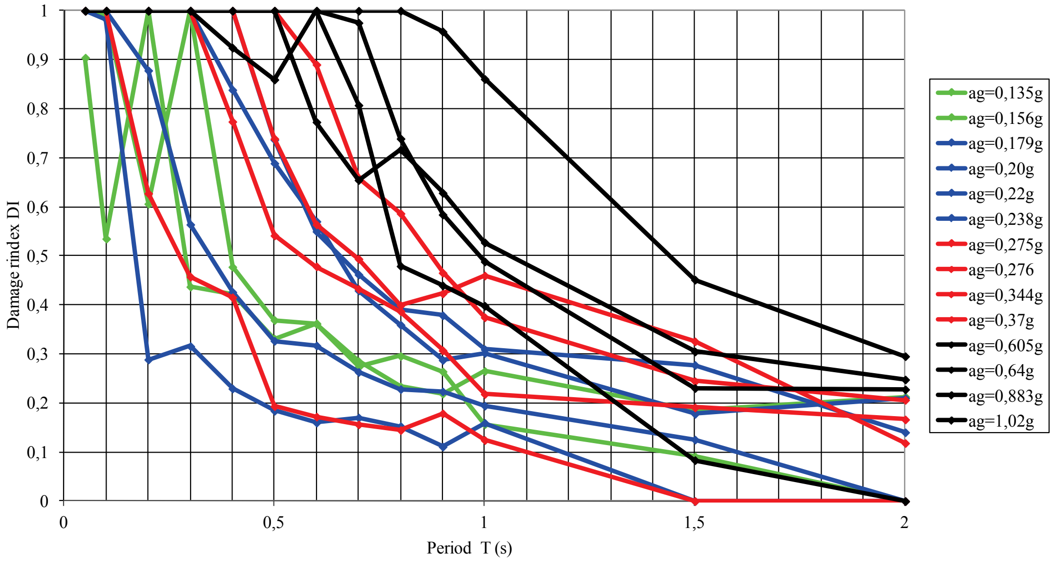

Figure 7 Spectral damage functions for RC frames defined by the following parameters: $BS_Y = 0{,}1W$, $K_2 = 0{,}0K_{el}$, $\xi = 5$ %

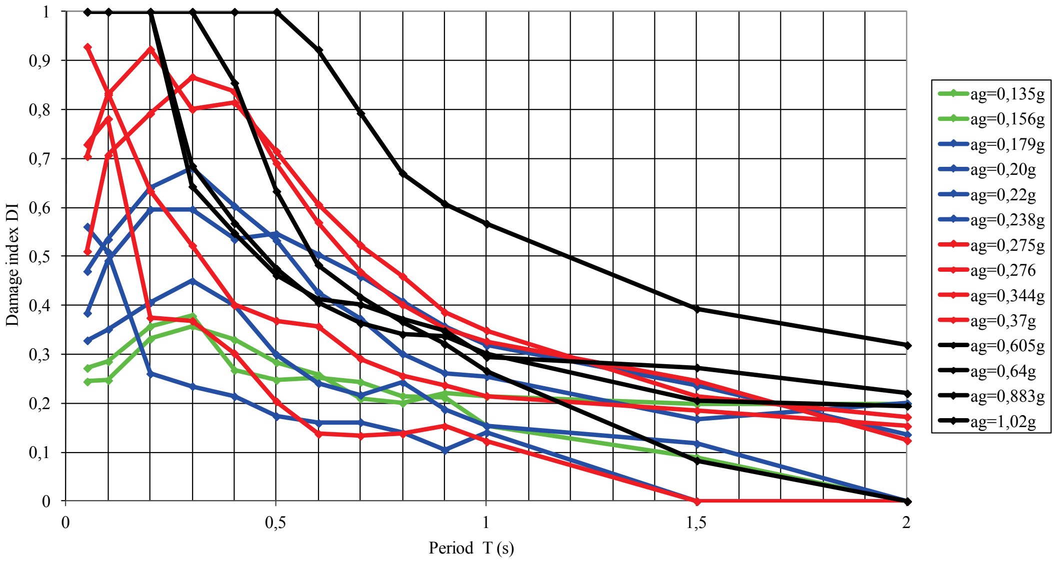

Figure 8 Spectral damage functions for RC frames defined by the following parameters: $BS_Y = 0{,}1W$, $K_2 = 0{,}2K_{el}$, $\xi = 5$ %

5 Seismic damage spectrum functions for RC frames

The results in sections 3 and 4 show that RC frame structures have post-elastic stiffness between $0K_{el}$ and $0{,}2K_{el}$ and a yield base shear value of at most 0,1W. Since damping of RC structures is usually assumed to be 5 % [26], a set of SDOF models was created using these parameter values as well as by varying the fundamental period within the range of 0,05 s to 10 s. *DI* values for these models were determined using Eq. (4) and implementing 20 different earthquakes as loads. Figs. 7 and 8 display the values of damage indices with respect to the fundamental period for different earthquake loads - spectral damage functions.

RC frame structures without post-elastic stiffness (Fig. 7) can be heavily damaged even by weak earthquakes. An increase in post-elastic stiffness increases the earthquake resistance of RC frame structures in case of a weak or moderate earthquake (earthquakes with peak accelerations corresponding to 0,24g) (Fig. 8).

6 Conclusion

Damage index (*DI*) interprets the level of structure damage by relating its values to the values of damage level identification, defined in the codes for post disasters damage assessment. The *DI* depends on the natural period, elastic base shear capacity, post-elastic stiffness and damping of the structure. In order to achieve approximate values or approximate boundaries of the abovementioned parameters of RC framed structures, it was necessary to provide experimental results. Analysing data from two publicly available databases that contain RC columns under standard cyclical loading indicates that the post-elastic stiffness is less than 20 % of the initial stiffness. The columns either have no residual stiffness or develop small post-elastic stiffness. The results of only 20 % of initial stiffness give us the maximum values, which can be used for post-elastic behaviour. Also, it is shown that the yield base shear depends mainly on the normalized axial load and an expression for this relationship is provided using genetic algorithms. Using 600 different models of RC frame structure models with known axial force in columns, it is shown that the normalized axial load is generally greater than 0,3, implying that the values of yield base shear of RC frames are at most 0,2W. Since the least important parameter is damping, it was decided that it is reasonable to provide seismic calculation with the value of 5 % of critical. Using the provided analyses, all parameters were obtained for the construction of the seismic damage spectrum functions. Seismic damage spectrum functions are constructed for RC frames using the following parameters: BS_Y=0,1W, K_2=0,0K_{el} and 0,2K_{el} and damping with 5 % of critical for different values of periods of structures. These spectrums were constructed for 20 different earthquakes ranging from 0,1g to 1,13g and using an original deterministic declaration of DI. The use of the constructed damage spectrum for RC frames is as follows: for a given RC frame with a calculated fundamental period of vibration (T_0), one can easily read off the value of potential damage (DI value) for chosen earthquakes (among 20 different proposed earthquakes).

7 References

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