Selection of Ground Motion Intensity Measure for Reinforced Concrete Structure

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Abstract

This paper presents analysis of different ground motion intensity measures that can be used in assessing the behavior of reinforced concrete structures. For this analysis, eight-story reinforced concrete building is selected with frame structural system in one direction and wall system in the other direction. The nonlinear time-history analysis for the considered structure is performed wherein the structure is exposed to forty natural records. Maximum of all interstorey drift ratio is selected as the seismic response parameter. After performing nonlinear analyses, diagrams that present relation between response parameter and considered intensity measured were constructed using regression analyses. The main aim is to find the most efficient intensity measures for the case of flexible reinforced concrete structures (frame systems) and stiff reinforced concrete structures (ductile wall systems), that are founded in two different soil types (rock and soft soil). Analyzing a large number of results and after their statistical analysis, it were adopted conclusions on the efficiency of individual ground motion intensity measures.

Keywords: Intensity measure, reinforced concrete structure, nonlinear time-history analysis, regression analysis

1. Introduction

Important question in estimation of the seismic response of structures is the choice of the appropriate intensity measure (IM). IM serves as a link between seismic characteristics of earthquake (magnitude M, source-to-site distance R, faulting style and soil type) and the estimation of the structure behavior. Ideally, the chosen IM should
have enough information about the earthquake, so according to it, the response of the structure can be predicted with certainty. Unfortunately, it can be said that today there is still no a parameter that can meet this requirement.

In this paper, different IMs are examined in terms of seeking the one that was the most efficient by performing a large number of nonlinear analyses of a eight-story reinforced concrete building. The most efficient means, those which gave the least dispersion of results of structural response for considered IM. It is investigated the influence of soil type (type A and type C according to EN 1998-1) [1] and the type of structural system (stiff and flexible).

Smaller dispersion of results means fewer nonlinear time-history analyses (fewer earthquakes) needed for reliable assessment of the seismic response of structures. It can easily be shown, that for the same confidence interval, dispersion reduction n times, reducing the required number of earthquakes (analyses) for n² times [2]. If we want, for example, 10% one-sigma confidence interval, the number of earthquakes is n = [ / 0.10]².

In this study it is investigated only the efficiency of IMs, although it is desirable to IM possess more other characteristics. Thus, for example, features such as convenience (easiness of obtaining adequate IM), efficiency (the ability to establish good mathematical dependence between seismic response and IM) and sufficiency (independence of the structural response to IM for given magnitude and distance from the fault) are certainly desirable

2. Ground motions intensity measures, IM

Ground motion IM should be capable of capturing significant features of ground motion that affect response of structure in quantitative form. The basic characteristics of ground motions that are interesting in terms of the elastic and inelastic behavior of structures are: amplitude, frequency content and duration.

The choice of IM depends on the:

- performance objective,
- engineering demand parameter,
- existence of the attenuation relation between the intensity measure and seismological-geological parameters: magnitude, source-to-site distance, faulting style and soil type. (Attenuation curves are obtained by statistical analysis of recorded ground motions and until today have been performed for a large number of locations and for the specific intensity measures (PGA, Sa (T1), Arias's intensity and other)).
- structural system.

The following IMs are analysed in this paper (units od measure are in brackets):

- Peak ground acceleration, PGA (m/s²)
- Peak ground velocity, PGV (m/s)
- Peak ground displacement, PGD (m)
- Spectral response acceleration value, Sa(T1) (m/s²)
- Spectral response velocity value, Sv(T1) (m/s)
- Spectral response displacement value, Sd(T1) (m)
- Predominant period, Tp (s)

Today the most common measure of seismic intensity is peak ground acceleration, PGA. This measure has natural connection with inertial forces and for specific types of structures (very stiff structures) maximum dynamic force, which appears in the structure, is directly relative to PGA.

Peak ground velocity, PGV is also useful IM because damages of numerous buildings are connected with energy, and energy depends on velocity. For the structures, which are sensitive for the intermediate frequencies, this measure may be better connected with the structure damages than PGA.

For very flexible structures it can be said that the maximum displacement of structure is going to be equal to the peak ground displacement, PGD. Ground displacements over time are obtained by double integration of (the first integration gives the time history of velocity) registered accelerograms and presents just approximately accurate ground displacements on location. Errors, which are inevitably present in the recorded accelerograms and that are trying to corrected during processing and filtering signals, are setting larger in the time-history of ground
displacement due to the double integration. Therefore, PGD as a IM is still less in use than the measures PGA and PGV.

Parameters of the maximum ground motion amplitudes, as PGA, PGV and PGD, do not show any data about frequent contents and earthquake duration, which also have influence on the structure behaviour, so it is necessary to consider other measures, trying to describe the earthquake intensity more accurate.

Spectral acceleration Sa, spectral velocity Sv and the spectral displacement Sd describe the maximum response of single-degree-of-freedom (SDOF) system to a particular input motion as a function of the natural period and damping ratio of the SDOF. For here considered structure, viscous damping is adopted at the amount of 5%. Spectrum values indirectly, i.e. over the response of the SDOF system, reflect strong ground motion characteristics. Spectral values depend on amplitude, frequency content, and to a lesser extent on the earthquake duration.

Predominant period Tp is a useful (though, in some cases insufficient) parameter by which is characterized frequency content of earthquake. It is defined as the period of vibration for which the rounded Fourier amplitude spectrum has a maximum value [3]. However, the predominant period can be determined in another way. Namely, it was noted that it could establish the relationship between PGV and PGA and predominant period. In this paper, it is used Eq.1 that is proposed by Heidebrecht and later analyzed by Fajfar [4]:

$$T_p = 4.3 \cdot \frac{PGV}{PGA}$$

(1)

The selected ground motion IMs can additionally be divided into two groups: the IMs that do not depend on the characteristics of the structure (PGA, PGV, PGD and Tp) and the IMs that depend on the characteristics of the structure (Sa(T1), Sv(T1), Sd(T1)).

3. Description of the considered reinforced concrete structure

3.1. Basic data about RC structure

RC structure that is analysed in this paper, presents eight-story building with total height of 24.0m and the storey height of 3m. Floor plan of the building is shown on Fig. 1. Building has 5m spans in both directions, four spans in the X direction and three spans in the Y direction. Structural system is RC frame system in X direction and RC wall system in the Y direction. The floor construction is RC monolithic slab with thickness of 15cm. The dimensions of the beams are 40/45cm and 20/45cm. The dimensions of the columns are 55/55cm. Thickness of the walls is 20cm. The class of concrete is adopted C35/45 according to EN 1992-1 [5] for all elements. Yield strength of longitudinal reinforced steel is fy=400 MPa, fy=240 MPa for transverse reinforcement and fy=500 MPa for mesh reinforcement.
Design of the considered structure was done according to European regulations EN 1998-1 [1] and EN 1992-1 [5]. The fundamental periods of structure for X and Y direction, obtained from linear analysis, are $T_{1x}=1.28s$ and $T_{1y}=0.57s$.

3.2. Modeling of RC structure for nonlinear analysis

For the purpose of performing nonlinear time-history analysis, model of the structure was created using Perform- a 3D [6].

Modeling of inelastic beams and columns was performed using Fema chord rotation model [6,7], by which they are modeled with two plastic hinge at both ends and elastic segments between them. Plastic hinges at the ends of the element are rotation hinges. This method of inelastic behavior modeling of beams and columns is quite satisfactory in the case of usual frame structure in which appearing of plastic hinges is expected at the ends of the elements. Trilinear behavior with strength loss for hinge moment-rotation relationship and the hysteresis loop with stiffness degradation is adopted (see Fig. 2).

Modeling of beam-column joint was performed using the Panel Zone element [6,7]. It consists of four rigid links, hinged at the corners and rotational spring that provides strength and stiffness.

Shear wall element [6,7] was used for the modeling of wall. Wall are modeled by defining the fiber cross-section composed of a number of fibers. The area and location of reinforcement within the cross-section and the properties of the concrete are defined using individual fibers. Concrete and reinforcing steel are modeled with nonlinear characteristics.

4. Selection of ground motions

The considered structure is exposed to the action of forty ground motions from the European strong-motion database [8]. Twenty ground motions are recorded on the rock (the first set), and the other twenty ground motions are recorded on soft soil (second set). These motions were characterized by surface-wave magnitudes, $M$, in the range between 6 and 7 and closest distances to the rupture surface, $R$, between 9 and 50 km. The motions recorded on rock, are scaled so that median of their PGA is 0.4g, while the motions recorded on soft soil are scaled so that median of their PGA is 0.3g. It should be noted that in the EC8 regulations, it is prescribed that recorded motions...
should be individually scaled to the value of the design ground acceleration, until here scaled motions have different maximum ground accelerations. This approach has the advantage of including different earthquake intensities and because of that it is possible to establish relationship between the intensity measure and response of the structure (see Eq. 2), which is necessary in the application of "performance based" methodology in probabilistic format [9].

In Tables 1 and 2 are shown the values of PGA of all ground motions after scaling, separately for both sets of records.

| Number of ground motion | 1  | 2  | 3  | 4  | 5  | 6  | 7  | 8  | 9  | 10 |
|------------------------|----|----|----|----|----|----|----|----|----|----|
| PGA                    | 0.16g | 0.17g | 0.21g | 0.22g | 0.28g | 0.29g | 0.3g | 0.31g | 0.34g | 0.38g |
| 11 12 13 14 15 16 17 18 19 20 | 0.42g | 0.46g | 0.48g | 0.50g | 0.52g | 0.54g | 0.56g | 0.60g | 0.65g | 0.70g |
| Median value           | 0.40g    |        |    |    |    |    |    |    |    |    |

Table 2. PGA for motions recorded on soft soil and their median value

| Number of ground motion | 1  | 2  | 3  | 4  | 5  | 6  | 7  | 8  | 9  | 10 |
|------------------------|----|----|----|----|----|----|----|----|----|----|
| PGA                    | 0.11g | 0.12g | 0.13g | 0.15g | 0.16g | 0.17g | 0.24g | 0.27g | 0.28g | 0.29g |
| 11 12 13 14 15 16 17 18 19 20 | 0.31g | 0.36g | 0.38g | 0.39g | 0.45g | 0.47g | 0.50g | 0.51g | 0.55g | 0.60g |
| Median value           | 0.30g    |        |    |    |    |    |    |    |    |    |

5. Results of nonlinear time-history analysis

The maximum interstorey drift IDR_{max} is selected as the seismic response parameter. IDR_{max} represents the maximum value of all interstorey drift ratio, where the maximum interstorey drift ratio is obtained as largest story drift divided by story height. This parameter of seismic response is commonly used in the literature as an indicator of collapse.

After carrying out the nonlinear time-history analysis for selected ground motions from both sets, result diagrams are obtained, which are presented by twenty points (x_i, y_i) where x_i are seismic response parameters (IDR_{max}), and y_i are considered IMs. In order to determine the efficiency of certain IMs, regression analysis was conducted. The regression line is assumed in the form of Eq. 2 [10]:

\[
\text{IDR}_{\text{max}} = b \cdot \text{IM}^a
\]  

In this paper dispersion is defined as the standard deviation of the natural logarithms of the residuals IDR_{max} data from the regression line defined by the Eq. 2. For lognormal distribution of the seismic responses this is the natural measure of dispersion [2]. Dispersion is denoted as IDR_{max} and it is given with (Eq. 3):

\[
\sigma_{\text{IDR}_{\text{max}}} = \sqrt{\frac{1}{n-1} \cdot \sum_{i=1}^{n} \left( \text{IDR}_{\text{max},i} - \overline{\text{IDR}_{\text{max}}} \right)^2}
\]

where are: IDR_{max,i} – the obtained seismic responses and \( \overline{\text{IDR}_{\text{max}}} \) - points on the regression line which can be calculated from the Eq. 4:

\[
\ln\left( \overline{\text{EDP}_1} \right) = \ln(a) + b \cdot \ln\left( \text{IM}_i \right)
\]
The efficiency of the IMs is determined by the size of dispersion. The smaller dispersion (scattering) of results is, the more efficient ground motion IM is.

As the illustration of the received results from nonlinear time-history analyses and the regression analyses, in the Fig. 3 and Fig. 4, diagrams that presents relationship between certain IM and IDR_{max}, are shown. The values of coefficients a and b which appear in the Eq. 2 as well as dispersion $\sigma$ are given in all above mentioned figures.

![Fig. 3. Relationship PGA and IDR_{max}](image)

(a) RC frame system  
(b) RC wall system

![Fig. 4. Relationship PGV and IDR_{max}](image)

(a) Ground motions recorded on rock  
(b) Ground motions recorded on soft soil

Dispersions of the maximum interstorey drift for all selected IMs that are obtained by regression analysis of the received results from nonlinear time-history analyses for both sets of ground motions are shown in Table 3.

| Maximum interstorey drift, IDR_{max} | RC frame system | RC wall system |
|-------------------------------------|-----------------|----------------|
| (X direction)                       | (Y direction)    | (Y)            |

Table 3. Dispersions for selected ground motion IMs
On the basis of the obtained results, which are presented in the Table 3, the following conclusions can be given:

Peak ground acceleration PGA, probably the most common IM today through which most of today's regulations define the design seismic forces, gave the greatest dispersion of results (except for Tp) for both types of structural system and both types of soil.

Peak ground velocity PGV, compared with PGA gave up two times less dispersion of results for both types of structural system and both types of soil.

Dispersion of results obtained by analyzing the behavior of structure for ground motions recorded on soft soil are more than twice then the dispersion of results for the case of ground motions recorded on rock for both types of structural system.

The greatest dispersions of results were occurred in RC frames system exposed to ground motions recorded on soft soil, while the least dispersions of results were occurred in the case of RC wall system exposed to ground motions recorded on rock.

Greater dispersions of the results are observed for RC frame structural system against the RC wall structural system, hence it can be concluded that flexible systems are more sensitive to the choice of an appropriate ground motion IM.

The most efficient IM for the case of RC frame structural systems is the spectral response velocity value, Sv(T1), while, in the case of ground motions recorded on rock, the other two spectral response values Sa(T1) and Sd(T1) are equally efficient.

The most efficient IM for the case of RC wall system is the peak ground velocity PGV, while in the case of ground motions recorded on rock, spectral response acceleration value Sa(T1) is equally efficient.

6. Conclusions

In this paper, different ground motion IMs that can be used in assessing the behavior of RC structures were investigated. The aim of the analyses was to find the most efficient IMs for the case of flexible RC structures (frame systems) and stiff RC structures (ductile wall system), that were founded in two different soil types (rock and soft soil). Conclusions on the efficiency of individual ground motion IMs were adopted after statistical analyses of large number of nonlinear time-history analyses results.

Peak ground acceleration PGA, probably the most common IM today through which most of today's regulations define the design seismic forces (including EC8), was not indicated to be an efficient IM for both types of structural system and both types of soil. On the basis of these studies, the peak ground velocity PGV is nominated as a universal IM that could be used instead of the PGA. PGV obtained for all the analyzed cases, has almost twice smaller dispersion, which means four times the smaller number of earthquakes to achieve the same reliable estimation of seismic response. Also, the use of spectral response values Sa(T1), Sv(T1) and Sd(T1) has given very good results but we should bear in mind that their calculation is more complicated because they depend on the dynamic characteristics of the structure.

For all ground motions IMs at least dispersions of results are obtained in the case of RC wall structural system on a rock, while the highest dispersions of results are obtained in the case of RC frame structural system on soft soil.

Bearing in mind that for both types of structural systems, higher dispersions of results are obtained for the case
of funding on soft soil, it can be concluded that in order to obtain reliable estimation of structural behavior in case of soft soil, it is desirable to require more number of ground motions for non-linear time-history analysis in code provisions.

References

[1] CEN European Committee for Standardization: Eurocode 8 - Design of structures for earthquake resistance Part 1: General rules, seismic actions and rules for buildings (2004), European standard EN 1998-1, CEN, Brussels
[2] Shome N. Probabilistic Seismic Demand Analysis of Nonlinear structures (1999) Doctoral Dissertation, Stanford University, Stanford, 320 p.
[3] Kramer S.L. Geotechnical Earthquake Engineering (1996) Prentice Hall, Upper Saddle River, New Jersey, 653pp.
[4] Aničić, D., Fajfar, P., Petrović, B., Szavits-Nossan, A., Tomašević, M. Earthquake Engineering (1990) Building Book, Belgrade. 642 p. (In Montenegrin)
[5] CEN European Committee for Standardization: Eurocode 2 - Design of concrete structures Part 1: General rules and rules for buildings (2004), European standard EN 1992-1, CEN, Brussels
[6] Perform 3D Product of Computers & Structures, Inc. Perform Components and elements (2006), Computers and Structures, Berkeley
[7] ASCE41 American Society of Civil Engineers Seismic Rehabilitation of Existing Buildings (2007), American Society of Civil Engineers
[8] Ambraseys et al. Dissemination of European Strong-Motion Data (2000), CD-ROM collection, European Council, Environment and Climate Research, Brussels
[9] Moehle J. P. Nonlinear analysis for Performance-Based earthquake engineering (2005) The structural design of tall and special buildings 14, 385-400.
[10] Janković S. Probabilistic seismic analysis of reinforced concrete frames (2003) PhD dissertation, Faculty of Civil engineering, University of Montenegro, Podgorica, 220p.