SEISMIC DESIGN OF R/C FRAMES: AN ASSESMENT OF BEHAVIOUR FACTOR AND A COMPARISON OF EUROCODE 8 AND BULGARIAN SEISMIC CODE

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SUMMARY

The results from one joint research investigation for R/C frames, carried out at ISMES, ITALY within the international project in support of the new Eurocode 8 (EC8) are presented in this paper. The purpose of this paper is to answer the main questions concerning the estimation of the behavior factor of RC frames according to EC8 and their connection with Bulgarian Code for seismic design of structures BGSC, as well as to show the possibilities for harmonisation of BGSC with the international standards.

INTRODUCTION

To avoid explicit nonlinear structural analysis in design, the basic principle of EC8 is taken into account by performing a linear analysis based on a reduced response spectrum, called for shortly "design spectrum". This reduction is accomplished taking into account the influence of soil conditions and introducing the behavior factor q. In the paper are also examined the methods for the assessment of the q - factor. The various methods can be grouped into three categories: (1) methods based on the ductility factor theory, (2) methods based on an extrapolation of inelastic dynamic response analyses of single-degree of freedom systems and (3) methods based on an energy approach. In this paper is proposed the method for assessment of q factor based on cumulative damage criteria. We have also presented a methodology for modelling and analysis of reinforced concrete frames subjected to seismic excitations, and for evaluation of structural seismic damage. In the paper is presented the model of the structure and is described the procedure for damage quantification, in term of local and global indexes according to Park, Ang and Wen method. The application of the analytical models are illustrated with the analysis of a reinforced concrete two stories one bay frame, designed and detailed to the final draft of EC2 and EC8, and of the same frame, but designed according BGSC. In the paper are used different types of earthquake records. The yield and ultimate limit state are defined on the basis of pushover analysis. The analytical results of the behavior of R/C frames using the program for nonlinear dynamic analysis IDRAC2D-4.0 are compared with the experimental results from shaking table tests and is made a comparison of the behaviour factor in EC8 and the response coefficient in Bulgarian code respectively on the basis of experimentally received results. We have also made a comparison of the reinforcement detailing requirements for EC8 and BGSC.

NON-LINEAR MODELING OF THE FRAME AND R/C ANALYSIS

The methodology for modelling and analysis of R/C frame structures and for the evaluation of the structural seismic damage parameters of the structures had been elaborated [Vasseva 1996].

The research program of this type of structures included the following main tasks: design of R/C frame; choice of the analytical hysteresis models to simulate the nonlinear behaviour of the frame elements; definition of criteria for the damage model; selection of set of earthquakes that reflect the expected seismic hazard; series of analyses of the frame with nonlinear dynamic model under different earthquakes with different intensities,

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evaluation of the ductility demand; interstory drifts; hysteretic energies; estimation of the received vulnerability functions and experimental testing to check the validity of the results obtained in the analytical and numerical studies.

In the studies, reported here, the IDARC2D program [Valles 1996] was used. This program had carried out which adopted a member by member macro - modelling the nonlinear analysis. Beams and columns were represented as the inelastic component elements with distributed plasticity and the cyclic behaviour of the member cross-sections was simulated by a degrading moment-curvature relationship with a nonsymmetric triliniar envelope curve. The hysteretic model depended on four parameters which control the stiffness degradation at unloading and reloading, the strength deterioration and the slip and pinching effects.

The purpose of this numerical analysis was to assess the q factors of the frames under consideration when the nonlinearities resulting from some damages were observed through the simulation of the essential characteristics of the hysteretic behaviour and comparing the analytical response with the experimentally recorded response. The hysteretic parameters were initially assigned based on well-detailed ductile sections, according to the requirements of EC8. The moment-curvature envelopes for sections of beams and columns were received. During the nonlinear dynamic analysis the sequences of cracking and yielding were analysed and deformations, stresses, relationship moment-curvature and damage indexes were received by the fibre method.

The correct evaluation of the q factor, defined as the ratio between the acceleration leading to collapse and one corresponding to the occurrence of first yielding requires several dynamic analyses for different ground motions In this methodology the main problem is the definition of the collapse conditions or of an “admissible” level of structural damage. So the first step is the damage evaluation.

A modified version of the Park, Ang and Wen (1985) damage model was used. This model was based on moment rotation and dissipated hysteretic energy as follow:

$$D = \frac{\theta_m - \theta_r}{\theta_u - \theta_r} + \frac{\beta}{M_y \theta_u} E_t \tag{1}$$

Where θm - maximum rotation attained during load history; θu - ultimate rotation capacity of section; θr - recoverable rotation at unloading; β - strength degrading parameter = HBE; My - yield moment of section; Et - dissipated hysteretic energy.

In the original Park model had been used different strength degrading parameters for damage and local member hysteresis. Since the purpose of the β parameter was to provide correlation between strength loss and damage, the present version used the same parameter for both damage computation and hysteretic modelling, i.e. β =HBE. This damage index can be used directly to determine damage at each member cross-section. Dissipated hysteretic energy was used as weighting factor to compute the component damage index. Two additional indices were also determined: a story level damage index and an overall structural damage index. Both indices were computed using weighting factors based on dissipated hysteretic energy at the component and story levels. Damage index was assigned at the element level (beams and columns), as well as at the story levels, and the overall structure. Part of the damages was due to permanent deformations and other part was due to strength deterioration from hysteretic behaviour. The received parameters (required ductility, dissipated energy, stories drifts) were connected with the degree of damages, observed during the provided shaking table test. The response of the frame was evaluated for the different intensity of seismic excitations.

CASE STUDY

3.1. Nonlinear dynamic analysis of R/C Frames designed according to EC8

A model of a two-story reinforced concrete frame, typical to construction practice of reinforced concrete structures in many countries like Italy, Turkey, Bulgaria was investigated [Vasseva 1996]. A typical R/C one bay two-story frame has been designed and detailed to the final draft of EC8 for the “high” ductility class. The preliminary design had been carried out at the University of Patras according to the EC2 (Eurocode 2 1992) and the current version of EC8 (Eurocode 8 1994) for ductility class H, assuming the load 11kN per story and high seismicity (peak ground acceleration 0.5g, soil type B, importance factor 1). The structure satisfied the regularity requirements both for vertical and horizontal configuration, so it was designed as a frame with high regularity in ductility class H with the behaviour factor q=5. All of the frame's elements were detailed in accordance to the EC8 rules.

The materials used for the specimens were normal-weight concrete C25/30 as specified EC2 and B500 Tempcore rebars. The mechanical characteristics of the materials were evaluated on the basis of the tests performed on the material samples at ISMES Laboratory (Vasseva 1996). Accepted concrete strength in numerical analysis was 33 MPa. Tensile tests have been performed on the rebars to be used and the complete

resulting stress-strain diagram to failure has been received. The steel had an average yielding strength of 581 MPa for ϕ12 and ultimate strength of 670 MPa with elongation of 20%. The steel exhibits good ductility in the material testing.

The expected earthquake ground motions were represented by the selected accelerogrames with different intensity scale factor λ - 0.3g; 0.4g; 0.5g; 0.7g, 0.8g, 0.85g.

Acc1: The artificial accelerogram from Patras University with design spectrum corresponded to EC8 spectrum -

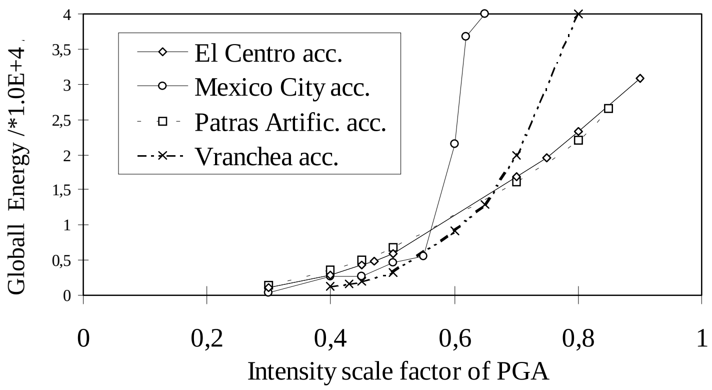

Fig.1 Vulnerability Functions for Global Hysteretic Energy - R/C Frame - EC8

Duration 10 sec, amax=0,3g=294 cm/sec2

Acc2: El Centro Site Imperial Valley Irrigation District, 18.5.1940; Max In. M=6.6; amax=341.7 cm/sec2; Duration 53,76 sec

Acc3: Vranchea, 4.3.1977; M=7.2, Depth =60-100km; Duration-40.09sec, amax=194,92 sm/sec2

Acc4: Mexico City -St.1. 19.9.1995, amax=97cm/sec2; Duration 190 sec,

The dynamic analysis was performed considering an integration time step of 0.001 sec. There was no predetermined basis for the choice of hysteretic parameters. The modified three parameter hysteretic model was used with a stiffness degradation coefficient HC=1.0 for column and 2.0 for beams, strength degradation coefficient HBE=0.05 (energy-based strength decay parameter) and HBD=0.0 (ductility-based strength decay parameter) (very little deterioration in strength) and a pinching coefficient HS=1.0 (target slip or crack closing parameter (indicating no pinching). From the nonlinear analysis the maximum values of the shear forces, displacements and story drifts were received for the different accelerograms with different scaling factors.

A significant dispersion of the computed earthquake parameters was obtained for the equal PGA of the different earthquake because of the effect of the frequency content variation.

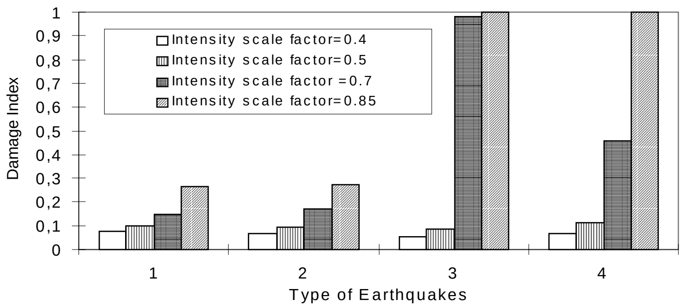

Fig.2 Damage index for different eartquakes and different intensity scale factor - R/C Frame -EC8

1- art. acc.Patras; 2- acc. El Centro;
3-acc. Mexico City; 4-acc. Vranchea

The calculated damage index corresponded to the physical damage and to the observed damages during the shaking table test. Part of the damages was due to permanent deformations while part was due to strength deterioration from hysteretic behaviour. To these damages corresponded the maximum hysteretic energy. Hysteretic energy was also a known measure of structural damage. From the numerical analysis the following results were received:

- in regards the global response the maximum values of the top displacement, interstorey drifts and base shear increase with the intensity.

-the variation of the shear forces were negligible during the different earthquakes and different scale factor

-the behaviour of the frame under consideration was very close for two type of earthquakes - art. acc. Patras and El Centro on one hand and Mexico City and Vranchea earthquake on the other hand, because of the different frequency content (Fig.1,2)

-the intensity scale coefficient 0.65 was the coefficient when the increase of all the parameters started for Mexico and Vranchea earthquake. This increase was very fast for the interstory drift. The value of this factor for two other earthquakes was 0.85.

-the comparison of the vulnerability functions for hysteretic energy, story drift and damage indices indicated that a measure of damages based on hysteretic energy alone may have not been reliable. The closest criteria with building damage observed during earthquakes were the dissipated energy and a structural overall drift.

The variation of this combined damage index (local and global) with the increase of the intensity was shown in Fig.2. For increased intensity they have higher values.

-the results from numerical analysis show that the frame structure during the nonlinear analysis with the design earthquake - Patras - amax = 0.5g performed very well The structural damages were mostly concentrated in the beams. The most dangerous for this frame was the Mexico City earthquake.

-significant dispersion of the computed earthquake parameters was obtained for the equal PGA of the different earthquake because of the effect of the frequency content variation.

3.2. The estimation of the behaviour factor

The behaviour factor was defined using the different methods. In Table 1 are given the values of the behaviour factor for the R/C frame designed according to EC8, received through the results from nonlinear dynamic analysis with the above mentioned different accelerograms.

Table 1 The values of the behaviour factor for R/C frame designed according to EC8

<table>
<tr><td colspan="10">Behaviour Factor</td></tr>
<tr><td rowspan="2">Excitation</td><td rowspan="2">qeng1</td><td rowspan="2">q*hys</td><td rowspan="2">q**hys</td><td colspan="2">q*dis</td><td colspan="2">q**dis.</td><td rowspan="2">q*eng</td><td rowspan="2">q**eng</td></tr>
<tr><td>qμ</td><td>qs</td><td>qμ</td><td>qs</td></tr>
<tr><td>Patras</td><td rowspan="2">5.68</td><td rowspan="2">3.36</td><td rowspan="2">3.93</td><td>2.47</td><td>1.6</td><td>3.23</td><td>1.7</td><td rowspan="2">6.9</td><td rowspan="2">6.92</td></tr>
<tr><td>art. acc</td><td colspan="2">3.95</td><td colspan="2">5.49</td></tr>
<tr><td rowspan="2">EL Centro</td><td rowspan="2">3.56</td><td rowspan="2">4.82</td><td rowspan="2">6.37</td><td>3.82</td><td>1.7</td><td>5.16</td><td>1.9</td><td rowspan="2">3.55</td><td rowspan="2">3.8</td></tr>
<tr><td colspan="2">6.49</td><td colspan="2">9.8</td></tr>
<tr><td rowspan="2">Mexico</td><td rowspan="2">1.92</td><td rowspan="2">4.63</td><td rowspan="2">7.91</td><td>2.8</td><td>1.2</td><td>4.3</td><td>1.3</td><td rowspan="2">0.68</td><td rowspan="2">0.49</td></tr>
<tr><td colspan="2">3.36</td><td colspan="2">5.59</td></tr>
<tr><td rowspan="2">Vranchea</td><td rowspan="2">2.89</td><td rowspan="2">5.8</td><td rowspan="2">8.25</td><td>3.08</td><td>1.4</td><td>4.37</td><td>1.51</td><td rowspan="2">1.75</td><td rowspan="2">1.56</td></tr>
<tr><td colspan="2">4.30</td><td colspan="2">6.59</td></tr>
</table>

In table q* are the value of the behavior factor at the ultimate limit state with 2% interstory drift;q** the value of the behavior factor at the ultimate limit state with 3% interstory drift.

qeng1 – this is the factor defined as the ratio between the work of the seismic elastic loading and the work of seismic loading during during the nonlinear behaviour of the frame for the loading corresponding to the max value of seismic loading for which the frame has been designed.

qhys - this is the factor defined on the basis of the histeretic energy corresponding to the ultimate limit state and this one at the first yielding of the frame

qdis - this is the factor defined through the displacement ductility of the frame and the corresponding overstrength coefficient

qeng - this is the factor defined through the ratio of the work of seismic loading of the elastic structure and deformation capacity of the frame at the given ultimate limit state.

The most closed results of the behavior factor to this used in the preliminary design of the frame was this received through the qeng taking into account the global damage index of the frame.

At this stage of the research was difficult to give final evaluation of the behaviour factor. There was too big dispersing of the results.

3.3. Nonlinear dynamic analysis of R/C frames designed according to BGSC

The same analysis procedure was performed for the frame structure designed to BGSC [Bulgarian Seismic Code]. The input data for damage analysis were chosen taking into account the reinforcement bars in columns and the beams, the type of the hoops. The design seismic loading for this frame were determined assuming the behaviour factor q=5, or the corresponding response coefficient R in BGSC[R=1/q)] -0.2. In this case the requirement of EC8 for lateral confinement were not taken into account. The procedure for capacity design was not used too. The spaces for the transverse reinforcement were less in the beams and in the columns. Details of the design of this frame were reported elsewhere Vasseva (1996)

The frame was designed for the same behaviour factor as the frame, investigated in the previous section, but the vulnerability functions shown in Fig.3 have more high values for the global hysteretic energy and the global damage indices are higher in this case too (Fig.4). It was evident from all the received results that the R/C frame in second case suffered more damages than the frame designed to EC8 under one and the same earthquake.

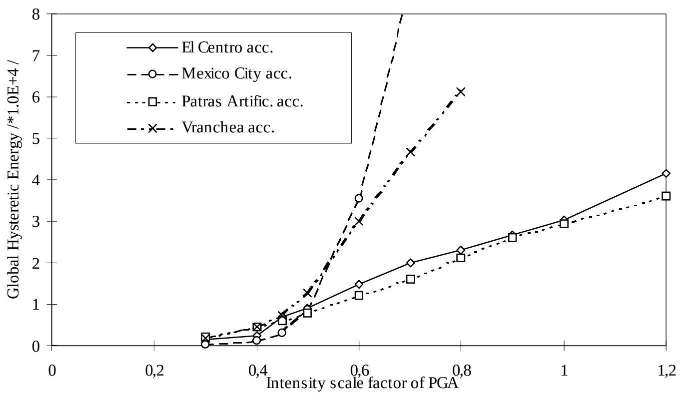

Fig.3 Vulnerability Functions for Global Hysteretic Energy - R/C Frame -BGSC

One of the reasons for this is the fact that in BGSC the response coefficient R was not explicitly connected with the ductility requirement. The behaviour factor is associated not only with the material, the structural system,

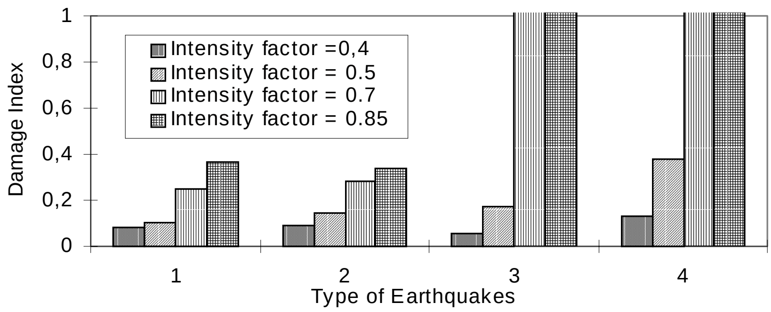

Fig.4 Damage index for different eartquakes and different intensity scale factor for BG R/C Frame 1- art. acc.Patras; 2- acc. El Centro; 3-acc. Mexico City; 4-acc. Vranchea

but also with the design procedure.

The design principles, given in EC8 and the proposing method of capacity design are connected with given ductility class, which main characteristic is the behviour factor q. The inclusion in the design project of a greater behaviour factor, than the one corresponding to the applied detailing of the structural elements, means that the structure is designed for smaller seismic action effects. That is the reason way the detailing provisions and capacity design method for analysis, given in EC8 have to be applied strictly. The provisions of detailing in order to achieve given ductility class are in general well known and are also included in BGSC, but in EC8 they are more conservative.

It should be mentioned that in BGSC some detailing provisions are taken into account in order to provide the local ductility demands in the critical regions. They are similar to these in EC8 in the quality aspect, but the difference is in the quantity. For seismic design EC8 requires an increased amount of transfer reinforcement. There are some differences in the spacing of the transfer reinforcements in the critical regions of the columns. The distance between consecutive longitudinal bars restrained by hoops in EC8 is more conservative. As a results of this there are more possibility plastic deformations to appear in columns in BGSC and the principle “week beam strong columns” is followed in the smaller degree. The open questions are the predicted values of the ultimate concrete strain; the uncertainties in material properties. In BGSC there are no additional requirements for reinforcing steel in critical regions.

3.4. Comparison of the numerical and the experimental results

Discussed briefly in this part of the paper were the results from performed shaking table test on a model of a two-story reinforced concrete frame has been tested in the ISMES-Italy Laboratory. During the test, cracks opened and closed in the critical regions of the beams in the first story and of the bottom of the columns. Only the cracks at the beam to column interfaces remained permanently open, more in the beams .The received from numerical analysis damage indexes and hysteretic energies in these beams were also higher. No local instabilities of reinforcement were observed neither during the experiment nor during the numerical analysis. The spalling of the cover was observed only in the bottom of column at first story. Beside the cracks at the beam to column interface, which were apparent in the first story and represented evidence of local yielding in the bars, the model remain apparently undamaged. The resulting mode shapes were closed to those of the structure in the beginning of nonlinear numeric analysis. The structure performed as expected. The frame developed a beam cracking and yielding, and also cracking at column base section, as was expected. It was concluded that the frame was able to withstand without collapse an earthquake consistent with EC8 response spectrum. The mentioned above method for damage assessment reflected well the observed damages.The correlation between received damage indices and the observed damage state for a given scale factor corresponds to the values from table 2 [Park,1985]. A value of DMI less or equal to 0.4 can be interpreted as repairable damage; from 0.4 to less than 1.0, as non repairable damage; and larger than 1,0 as failure.

Table 2. The correlation between received damage indices and damage states

Damage Index	0.	0.0-0.01	0.01-0.1	0.1-0.3	0.3-0.6	0.6-1.0	1.0
Damage State	None	Minor	Light	Moderate	Heavy	Major	Destroyed

CONCLUSION

Based on the results obtained for the different case studies several conclusions can be stated:

- The design and detailing of R/C structure is much more important than analysis.

- The application of behaviour factor q in seismic design depends on several of parameters

- Taking into account the great influence of the behavior factor on the value of seismic action effects and displacements it is evident that a basic re-examination and development of this factor respectively of the similar Response coefficient R in BGSC is needed. This is very important because parameters as ductility and irregularity are not included in the Response coefficients R in BGSC.

-The design philosophy of the existing code for seismic design of structures in Bulgaria is based on the behavior factor and the limit sate concepts. Therefore the harmonisation of the Bulgarian code to the EC8 will not produce conceptual problem. The distinction between principles and application rule made in EC8 help for this too.

-The final conclusion is that the results of this research are a very helpful step in understanding the issues of real nonlinear behaviour of R/C frames and the behaviour factor. This coefficient is very complicated and additional research efforts are needed before its introduction in the new Bulgarian Code for seismic design of reinforced concrete frames.

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