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Nonlinear Seismic Analysis of a Semi-Rigid Frame Building

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Summary

Full-scale pseudodynamic tests of a 4-story RC frame building in Ispra demonstrated semi-rigid behaviour of beam-column joints due to the slippage of the steel bars. In the paper the mathematical modelling of the structure is discussed. The results of nonlinear dynamic analyses are compared with experimental results and with the results obtained by a simplified nonlinear seismic analysis.

1. Introduction

A series of pseudodynamic tests on a full-scale four-story reinforced concrete building designed according to Eurocodes 8 and 2 was carried out in the European Laboratory for Structural Assessment (ELSA) of the Joint Research Centre of the European Commission in Ispra. The test results indicate "that the slippage of the steel bars in the joints dominates the behaviour of the structure leading to a very pronounced pinching of the story shear-drift diagrams, consequently reducing the potential dissipation capacity of the structure" [1]. Due to this effect, the behaviour of beam-column joints can be described as semi-rigid. In order to numerically simulate the seismic structural response of such a structure appropriate mathematical modelling is required. In the paper, a relatively simple mathematical model of the building structure is presented. The main results of nonlinear dynamic analysis are compared with experimental results and with the results obtained by a simplified nonlinear seismic analysis.

2. Description of the structure and of the tests

The general layout of the structure is shown in Fig. 1. The materials used for the test structure are normal-weight concrete C25/30 as specified by Eurocode 2, and B500 Temcore rebars and welded meshes. The design was made in accordance to Eurocodes 2 and 8, assuming effective peak ground acceleration 0.3 g, soil type B, importance factor 1, ductility class H and behaviour factor q=5. The detailed data on the reinforcement are given in [2]. The lumped

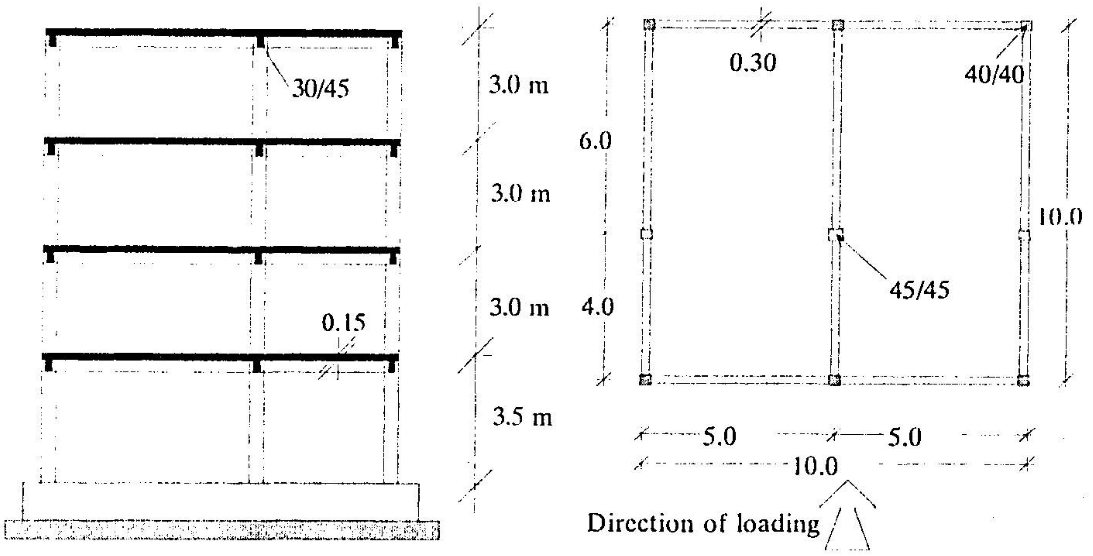

Fig. 1. Layout of the test structure

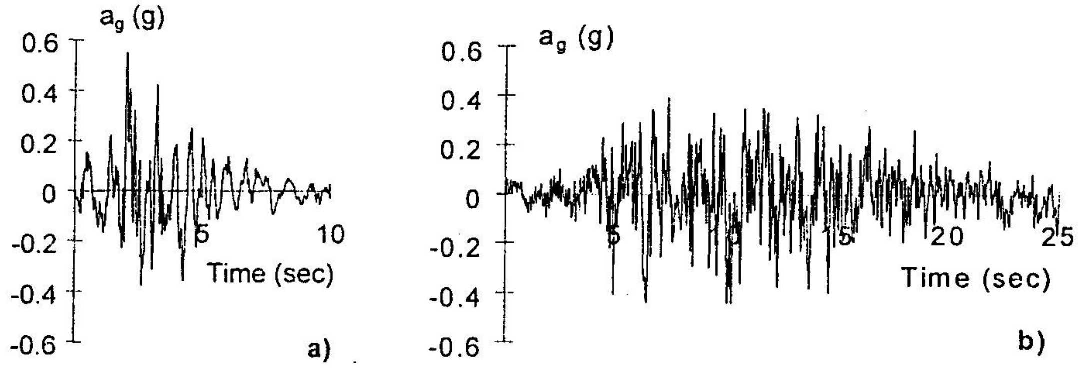

Fig. 2. Accelerograms used in the study
(a) Friuli based accelerogram used in the high-level pseudodynamic test
(b) Montenegro based accelerogram

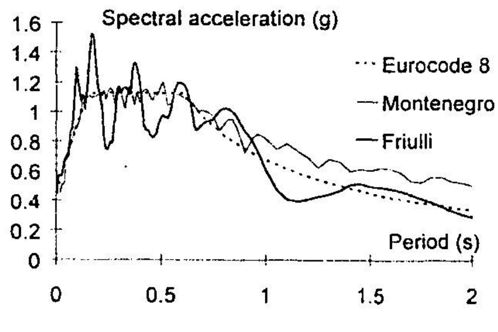

Fig. 3. Elastic spectra for 5 percent damping

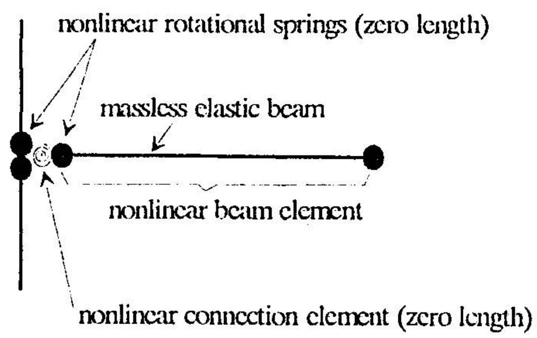

Fig. 4. Arrangement of elements

masses amount to 86.9, 85.9, 85.9 and 83 tons in stories 1 to 4, respectively. An artificial accelerogram (Fig. 2a) was generated to fit the elastic response spectrum given by Eurocode 8 for soil profile B and 5% damping (Fig. 3). The accelerogram is based on an actual record from the 1976 Friuli earthquake The duration is 10 seconds. A low-level and a high-level pseudodynamic tests were performed. In the first test, the accelerogram was scaled to 40% of design intensity, i. e. to 0.4 x 0.3 = 0.12 g nominal ground acceleration. In the second test, the design intensity was increased by a factor of 1.5. The nominal peak ground acceleration was thus 0.45 g. More detailed data on the test can be found in [2] and [3].

3. Mathematical modelling

All analyses were performed with a modified version of the DRAIN-2DX. A new element was implemented in the program and used for modelling of all beams and columns: a perfectly elastic, massless beam element with two nonlinear rotational springs at the two ends. The moment-rotation relationship for each spring was defined by a trilinear envelope (representing the three typical parts of the moment-rotation relationships of RC sections: up to cracking, between cracking and yielding, after yielding) and by Takeda's hysteretic rules. Asymmetric backbone curves were used for beams, the behaviour of which when subjected to positive and negative moments is different. In addition to beam elements, simple rotational connection elements, defined in the DRAIN-2DX as type 04 elements with "inelastic unloading with gap", were used to model the influence of the slippage of steel bars in the joints. They were placed between beams and joints. The rigid connection was assumed between columns and joints. The arrangement of elements is shown in Fig. 4.

The moment-rotation relationships in rotational springs representing nonlinear behaviour of beams and columns were calculated based on moment-curvature relationships and assuming the antisymmetrical moment distribution along the length of the element. The initial stiffness was calculated using experimentally obtained moduli of elasticity E (from 28.5 to 35.3 kN/mm^2). The effective widths of the slab contributing to the beam of 1.5 m and 0.9 m were adopted for the internal and external frames, respectively [1].Cracking in beams was determined using the concrete tensile strength, which amounted to 10 percent of cylindrical strength, and axial force due to dead load. In columns, the strength at cracking was reduced to one half of the computed value. In such a way the influence of varying axial force due to horizontal loading was approximately taken into account. Analytical procedures for the determination of post-yield stiffness are quite unreliable. Based on several calculations with different assumptions the following values were chosen for post-yield stiffness expressed in percentage of the secant stiffness to the yield point: columns 15%, beams 2% and 20% for positive and negative moments, respectively. It is well known that the region in which slab reinforcement yields progressively spreads with increasing beam rotation. When calculating the ultimate moment and curvature, in the case in a negative bending moment (with the slab in tension), this phenomenon can be approximately taken into account by assuming a greater effective width. As a result, a relatively high post-yield stiffness is obtained.

In the Takeda's hysteretic model the unloading stiffness is controlled by the parameter α. This parameter proved, in the case of the investigated building, to exhibit a quite important effect on the time-history of the structural response. Best correlation with the experimental results was obtained by using the maximum possible value, i. e. $\alpha = 1.0$. As a consequence of this assumption, the unloading stiffness after yielding is relatively small and little energy is dissipated in inelastic cycles after yielding.

The initial stiffness of connection elements was determined according to Fillippou et al [4]. The semi-rigidity of joints introduced by the connection elements has increased the resulting beam end rotations in elastic region for about 50 percent. The yield moments of connection elements were assumed to be the same as the (positive and negative) yield moments of the corresponding beams. The post-yield stiffness was arbitrarily chosen as 5 percent of the initial stiffness. The hysteretic behaviour is shown in Fig. 5.

The influence of the viscous damping is, due to the low hysteretic energy dissipation capacity, important even in the inelastic region. Only mass proportional damping was used. The best correlation with the experimental results was obtained by using one percent of critical viscous damping, determined using the initial first mode period.

The analysis was performed in sequence. First, the low-level excitation was applied. It was followed by the high level excitation. For comparison, the structural model without the connection elements (i. e. with rigid joints, R - model) was analysed. In addition to the ground acceleration time-history, used in the tests, an additional accelerogram was generated (Fig. 2b) It fits the same spectrum (Fig. 3), but it is based on the Petrovac record from the 1979 Montenegro earthquake. Its duration is 25 seconds. In addition to dynamic time-history analyses, a simplified nonlinear analysis called N2 method [5] was performed. It consists of a static analysis of the MDOF model under monotonically increasing horizontal loading (push-over analysis) and a spectral analysis of the equivalent SDOF model. The relative values of horizontal loads amounted to 0.528, 0.934, 0.926, and 1.0, from the first to the top story, respectively. In order to obtain the results, which are comparable to the results of time-history analyses, the Eurocode 8 design spectrum (normalised to 0.45 g effective peak ground acceleration) for one percent damping was used.

4. Results

The most important results are shown in Figs. 6 - 13. If not stated otherwise, the results correspond to the mathematical model with connection elements (SR - model) and to the time-history analysis with the high-level Friuli based accelerogram. It can be seen that the developed mathematical model yields results which correlate quite well with the test results (with some exception of the top story). As far as time- histories are concerned, such a good correlation would not have been possible without fitting of some parameters. However, the maximum response values proved again to be relatively insensitive to moderate variations of structural properties. As an exception, the computed maximum rotations of beams in the upper part of the structure are much larger than the measured rotations.

In the case of the model without the connection elements (R - model), the displacement time-history for the low-level test, where the structural behaviour is essentially elastic, does not correlate well with the observed time-history. However, in the case of the high-level excitation, not only the maximum values but also the complete time-history agree quite well with the measured results. It seems that the time-history response of the tested structure is

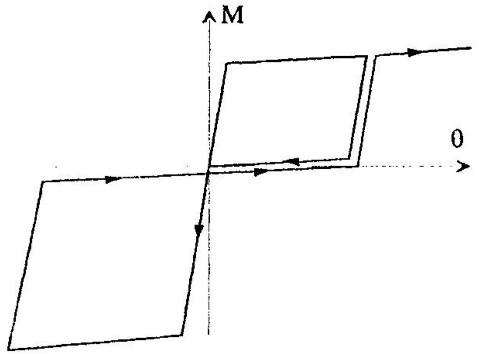

Fig. 5. Moment - rotation relationship for connection element

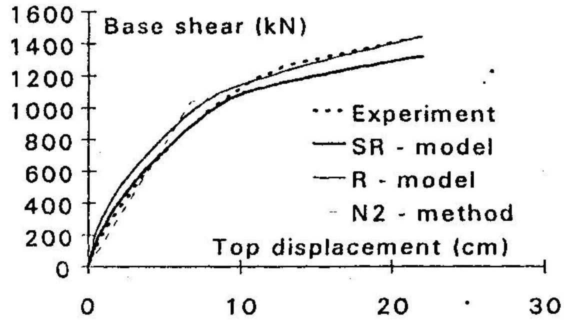

Fig. 6. Top displacement - base shear relationships

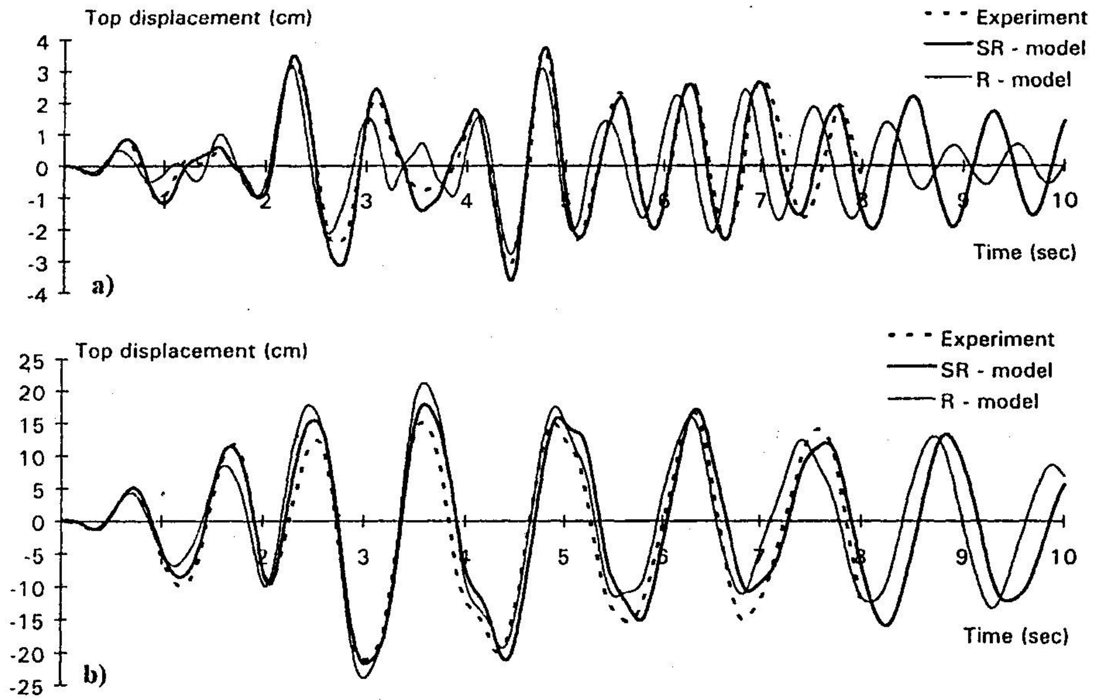

Fig. 7. Time - history of top displacement: (a) low-level excitation
(b) high-level excitation

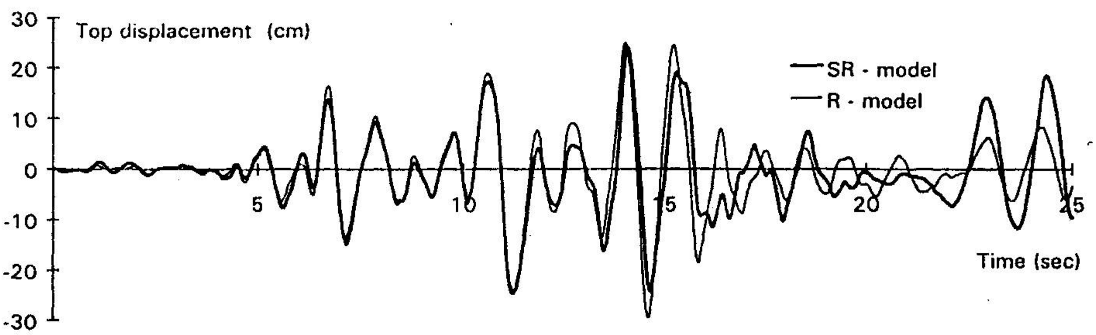

Fig. 8. Time - history of top displacement for the Montenegro based accelerogram

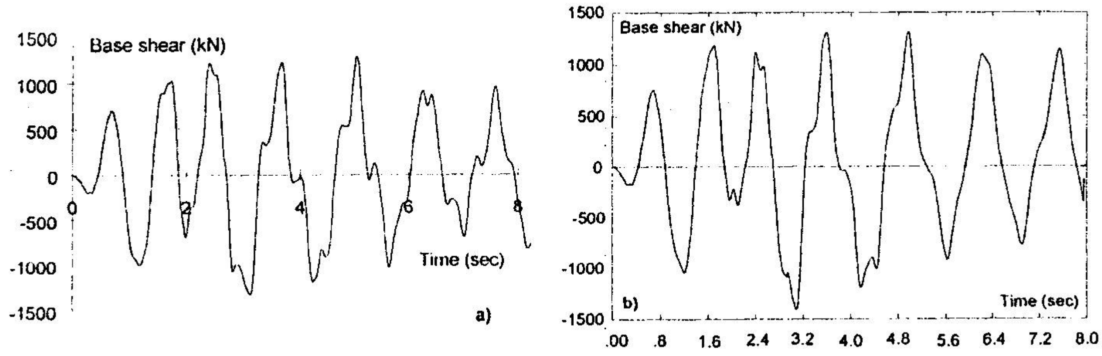

Fig. 9. Computed (a) and experimentally obtained (b) time - histories of base shear

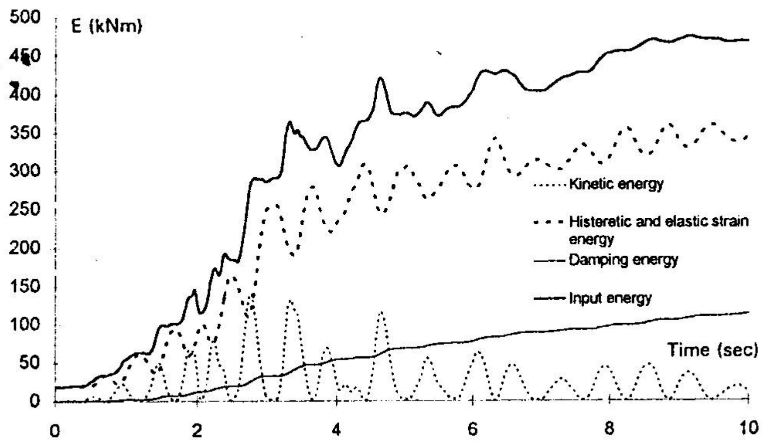

Fig. 10. Computed time-histories of energies

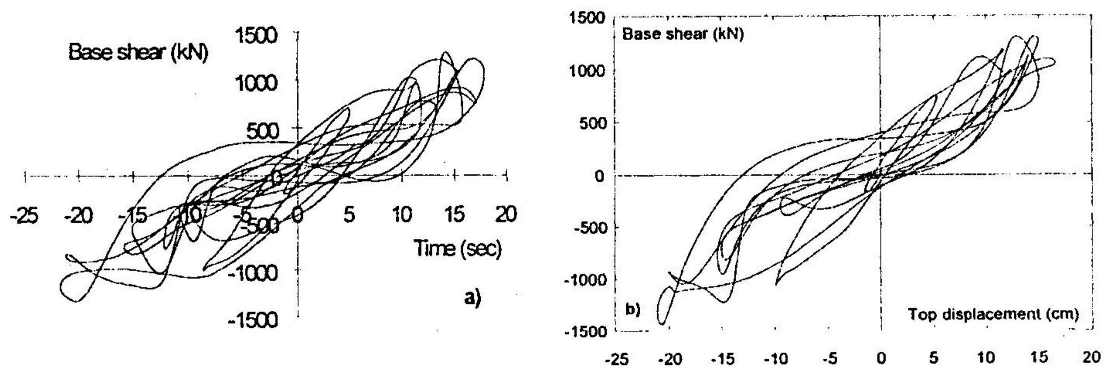

Fig. 11. Computed (a) and experimentally obtained (b) base shear - top displacement relationships

highly influenced by the low energy dissipation capacity during non-peak inelastic cycles. This feature was incorporated inboth structural models (with and without connection elements) by using the unloading stiffness parameter $\alpha = 1$. The connection elements influence the stiffness in the elastic range and the hysteretic behaviour in the inelastic range. It seems that the second

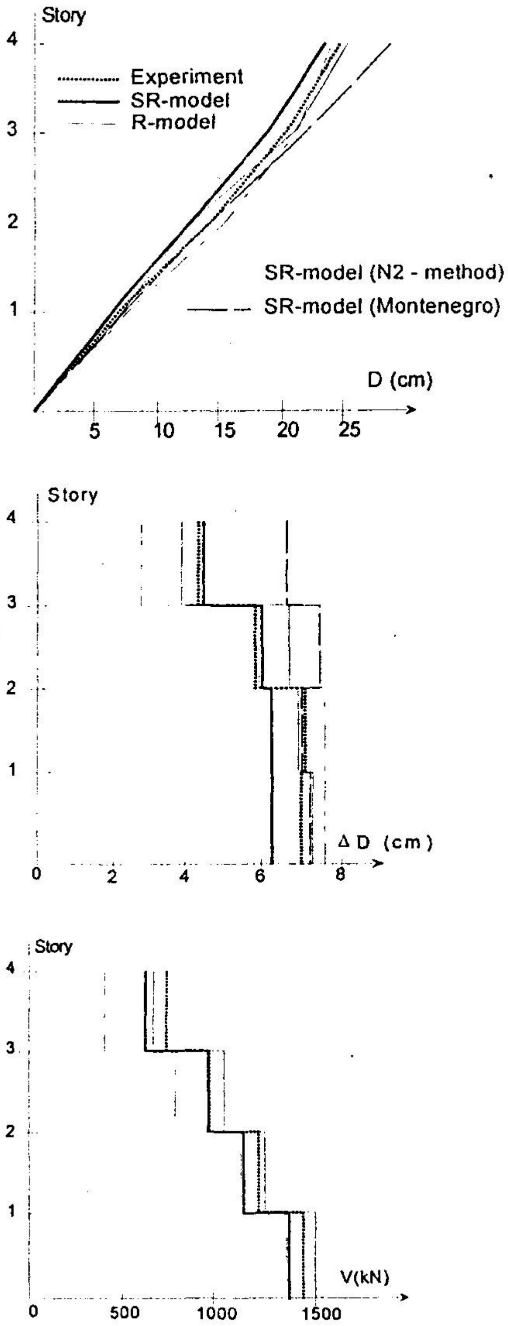

Fig. 12. Vertical distribution of maximum displacements, story drifts and shear forces

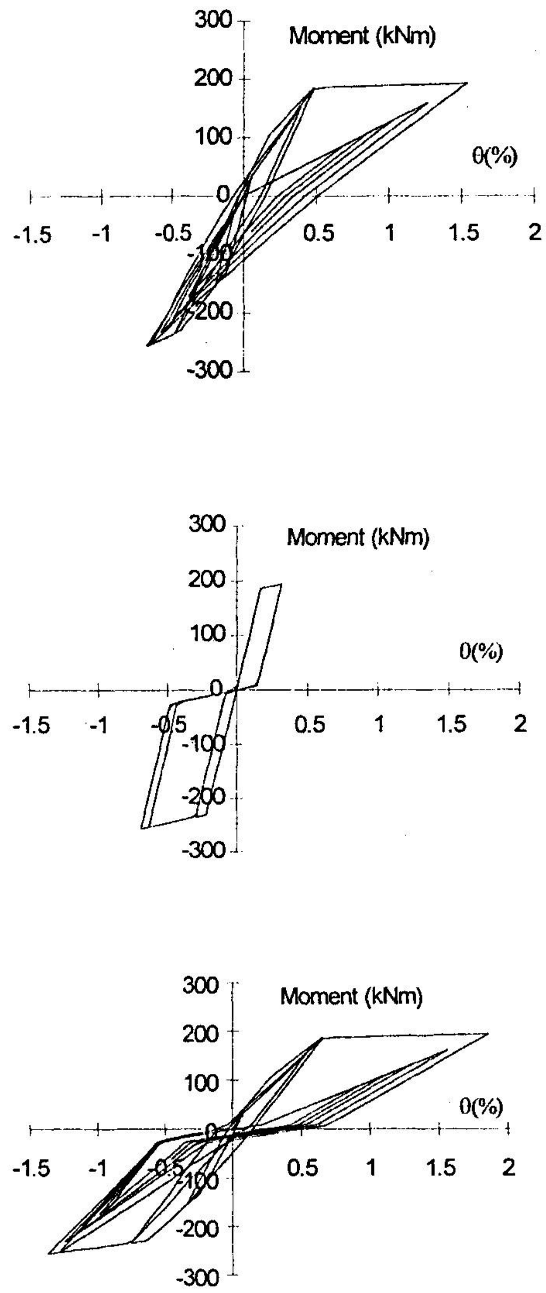

Fig. 13. Moment-rotation relationships for a typical beam, conection element, and combined beam and connection element

effect has only a minor influence on the overall structural behaviour. The major difference between the both models subjected to the high-level excitation can be observed in the computed rotation ductilities which are larger in the case of the model without connection elements. The maximum computed rotation ductility factor in the SR - model amounts to 3 and 4.4 for columns and beams, respectively. The corresponding values in the R - model amount to 3.5 and 6.2.

The structural response to the second (Montenegro based) accelerogram differs to some extent from the corresponding results for the first (Friuli based) accelerogram, although both of them are supposed to fit the Eurocode 8 design spectrum (Fig. 3). The differences in response are a consequence of only a small part of uncertainties connected with the input ground motion which characteristics can be only roughly predicted. Having in mind this fact, one can appreciate a relatively simple method, like the N2 method, which is able to produce the main results, determining seismic demand, with similar accuracy as a more sophisticated nonlinear dynamic analysis.In the case of the investigated building are a small exception underestimated response quantities in the upper story. The N2 method has been designed for analysis of structures vibrating mainly in the fundamental mode. Higher mode effects can be taken into account only if a correction procedure is applied.

5. Conclusions

It has been shown that the semi-rigid behaviour of a reinforced concrete frame, which is caused by the slippage of the steel bars in joints, can be adequately simulated by a combination of conventional nonlinear elements, i. e. beam elements with concentrated plasticity, and connection elements. While some fitting of parameters is necessary for obtaining close correlation of computed and measured response time-history, are the most important global response characteristics, like maximum displacements, story drifts and story shears, relatively insensitive to the details of mathematical models (having in mind the uncertanties involved in seismic design). Moreover, they can be predicted with a reasonable accuracy by a relatively simple nonlinear procedure which includes a nonlinear static analysis of the MDOF system and a spectral analysis of an equivalent SDOF system.

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