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APPLICATION OF THE CAPACITY SPECTRUM METHOD TO R.C. BUILDINGS WITH BEARING WALLS

Kaspar PETER[1] And Marc BADOUX[2]

SUMMARY

The paper presents results of a research project investigating the seismic evaluation of reinforced concrete structures with bearing walls. The investigation is based on the analysis of nonlinear structural models with the capacity spectrum method, an approximative nonlinear static analysis procedure, and timehistory analyses. The latter serve as reference. Efforts focus on the realistic modelling of reinforced concrete buildings with bearing walls and on the determination of the application limits of the capacity spectrum method.

INTRODUCTION

This contribution presents preliminary results of a research project underway at the Swiss Institute of Technology in Lausanne, Switzerland. The aim of the research project is to investigate a nonlinear static analysis procedures, namely the capacity spectrum method, for the seismic evaluation of existing r.c. buildings with bearing walls. The ability of the capacity spectrum method will be tested on example buildings using the results of Nonlinear Dynamic Analyses (NLDA) as reference.

With the growing awareness and concern of the seismic risk, there is a need for efficient and reliable tools for seismic evaluations, for the structural engineer's practice or for vulnerability studies. It was shown by many researchers that for frame buildings, the capacity spectrum method is, within some limits, an efficient and reliable seismic analysis tool ([Freeman, 1998], [ATC-40, 1997], [Sasaki et al., 1998] and many others). Though bearing wall buildings are common everywhere and in some countries the dominating construction type, only few research projects study the application of the capacity spectrum method on bearing wall buildings [Kunnath et al., 1996]. As the seismic response of bearing wall buildings is different from the seismic response of frame structures, the applicability of the capacity spectrum method for bearing wall buildings has to be investigated specifically.

CAPACITY SPECTRUM METHOD

Nonlinear models usually produce more realistic seismic response analysis than elastic models. However, nonlinear dynamic analyses of entire buildings require a lance computation effort. This computation effort can be substantially reduced when the computation is limited to a nonlinear static analysis combined with a procedure to determine the maximum deformation for the nonlinear static analysis. Using that approach, the seismic response of a r.c. structure might be estimated, not only for the ultimate response but for various impact levels. We focus here on the Capacity Spectrum Method (here CSM), that we want to test applied on existing r.c. bearing wall buildings.

The Capacity Spectrum Method (CSM) was developed by S. A. Freeman [Freeman, 1998] for frame buildings. Its concept have been introduced in several US guidelines for seismic evaluations such as the ATC-40 [Applied Technology Council, 1996] and the NEHRP guidelines for the seismic rehabilitation of buildings [FEMA-273, 1997]. The CSM is an approximative procedure to analyse with a nonlinear static analysis (push over) the seismic response of a structure. The CSM helps to analyse the seismic response of a structure in terms of forces and displacement and permits to describe efficiently the seismic performance of structures. The CSM concept is

[1] *Institute of Statics and Structures, Chair of Reinforced and Prestressed Concrete http://ibapwww.epfl.ch/default.asp*
[2] *Institute of Statics and Structures, Chair of Reinforced and Prestressed Concrete E mail: marc.badoux@epfl.ch*

based on the supposition, that the maximum lateral story drifts describe efficiently the seismic building response. A next supposition is, that the maximum lateral story drifts are dominated by deformations of the fundamental mode of the originally elastic system.

With a push over analysis the nonlinear response of a structure under lateral loading – according the fundamental mode – is computed (Figure 1, top left). The nonlinear response is described with the push over curve, which plots the base shear versus the roof displacement (Figure 1, top right). This push over curve can be transformed into the "capacity spectrum" (Figure 1, bottom) using the structure's originally elastic dynamic properties (participation factor and modal mass coefficient). This capacity spectrum is represented in the Acceleration Displacement Response Spectrum format (ADRS), using spectral displacements ($S_d=S_a/\omega^2$) and spectral accelerations (S_a) (Figure 1, bottom).

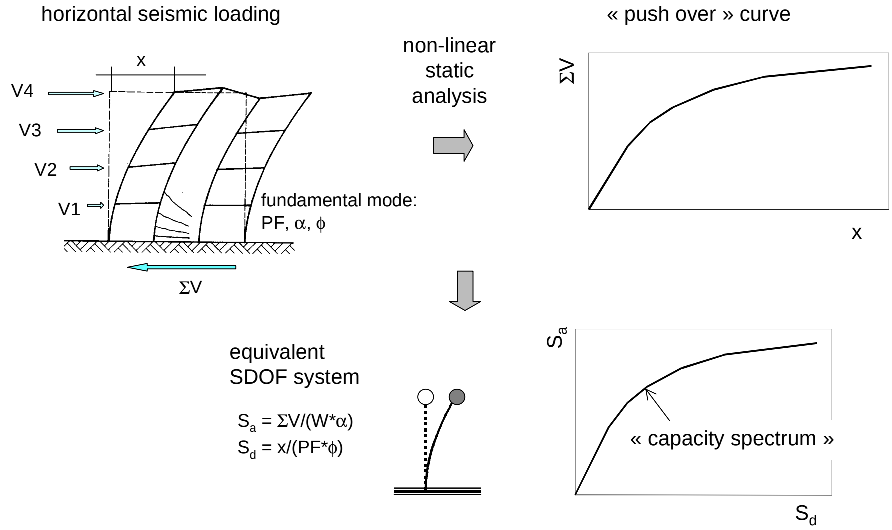

Figure 1.The capacity spectrum as a result of the push over analysis

The elastic response spectrum can be plotted in the ADRS as well (Figure 2, left). The intersection of the capacity spectrum and the elastic response spectrum yields the target displacement in terms of spectral displacement and acceleration, under the condition that the amount of equivalent viscous damping is known. The equivalent damping accounts for the effects of hysteretic and material damping. In the CSM, the originally elastic stiffness is relevant for the displacement demand, but the so called "substitute structure" [Shibata, 1976]. The substitute structure is a virtual substitute for the real hysteretic system with elastic properties, i.e. the secant stiffness (k_{secant}) of the real system and equivalent damping ($\beta_{equiv.}$) to equal the hysteretic and material damping of the real system (Figure 2, right). Several rules exist to determine the equivalent damping [ATC-40, 1997], but for bearing wall buildings, the authors have no rule selected yet.

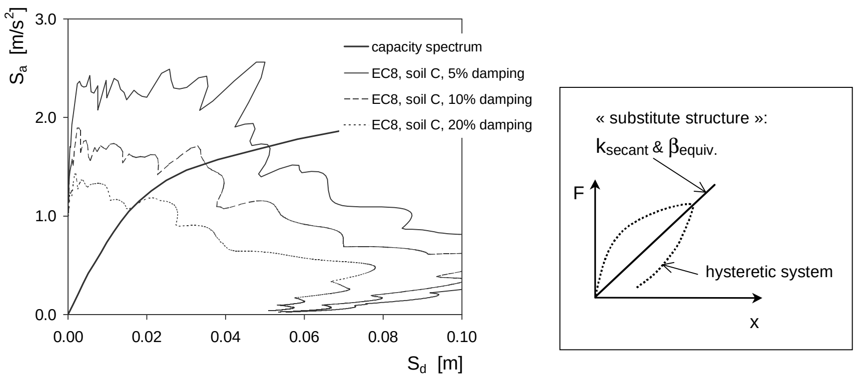

Figure 2.Capacity spectrum method using the substitute structure approach

The graphic representation of the building response in the ADRS is a great help not only for the understanding of the analysis results but for the design of potential retrofit measures as well [Badoux, 1998]. As a result of the simplifying assumptions on which it is built, the CSM is not well adapted for the analysis of structures where the fundamental mode does not dominate the maximum story drifts, such as tall buildings (more than about 15 stories), or buildings under shock-like impacts ("near source effects"). The results of the CSM further depend strongly of the reliability of the nonlinear structural model and the determination of the equivalent damping for the target displacement.

REALISTIC MODELS FOR RC STRUCTURAL WALLS UNDER SEISMIC LOADING

Displacement based analysis methods such as the CSM need structural models that predict reliable inelastic deformations. Such models, especially simple ones, are less established than models for the ultimate resistance. Nonlinear finite element analyses are a possible solution, but they are generally expensive and time-demanding for engineer's practice. In this study, macro-models calibrated against laboratory tests were used. The analytical tool is software Idarc2d, Version 5, developed by Professor Reinhorn and co-authors [Valles et al., 1996] (plus update version 5), modified as needed for the study. The modifications concern mainly the fiber model and the computation of the hysteretic loops.

The cyclic response of r.c. walls has been researched in several test series. Results from two experimental campaigns were used for the calibration of the flexural behaviour of the wall macro models. One campaign was conducted at the Swiss Institute of Technology, Zürich, under the direction of Prof. H. Bachmann. Six walls were tested under static cyclic loading [Dazio, 1999] and six others on a shaking table [Lestuzzi, 1999]. These twelve walls have a rectangular concrete section. The other experimental campaign included r.c. walls with boundary elements (barbel shaped sections and flange walls), realized in the seventies in the USA [Oesterle, 1976 & 1979]. Based on these test reports, the following conclusions relevant for the seismic evaluations can be drawn:

- the stiffness measured on "uncracked" r.c. walls can vary considerably from the sections stiffness computed as the product of the Young's modulus of concrete (E_c) measured in a cylinder test and of the moment of inertia of the uncracked gross section (I_c). The tests revealed section stiffness values varying between (0.2) and (0.8)*E_c*I_c. In the following, for walls with an axial compression higher 2 MPa or walls with boundary elements an uncracked section stiffness of (0.7)*E_c*I_c is selected and for walls without boundary elements and smaller axial loading than 2 MPa, the stiffness is chosen equal to (0.3)*E_c*I_c.

- most tested r.c. walls were designed for ductility and detailed with a special confinement reinforcing at the wall edges. But one wall tested on the shaking table was designed according to conventional rules, i.e. no specific confinement was put in place. This wall nevertheless revealed a considerable hysteretic energy dissipation capacity as well.
- after the first rebar fracture, the resistance of the walls did not deteriorate completely but a rather stable rocking mode was observed.
- the dynamic response of r.c. wall is rather variable.

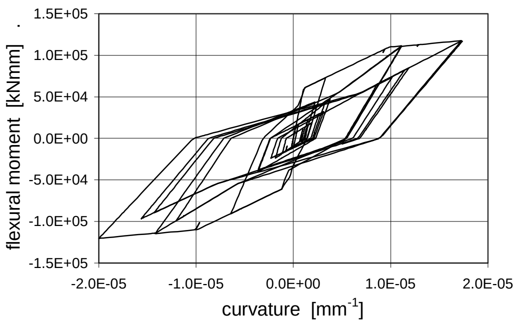

Figure 3.Three-linear hysteretic model for the flexural behaviour of a r.c. wall implemented in Idarc2d [Valles, 1996 plus update version 5]

To calibrate the shear response of r.c. walls, the tests of Oesterle, Maier [Maier, 1985] and NUPEC's seismic shear wall tests [NUPEC, 1996] were included. For structural masonry walls only few test results are available ([Ganz, 1982], [Schwegler, 1994]). To compute the resistance of seismically loaded structural masonry walls, Canadian guidelines for seismic evaluations of existing buildings were used [Bruneau, 1994]. They compare satisfyingly with the test results by Ganz and Schwegler.

The wall macro-model incorporated in Idarc2d uses two springs, one on the top and one at the bottom, to model the flexural response. A third spring represents the wall's shear response. All springs can show a trilinear hysteretic behaviour, as shown in Figure 3. This permits to model the uncracked, the cracked and the yielding section. Under cyclic loading the hysteretic energy dissipation is included. The properties of the trilinear envelope are computed with a fiber model, based on the mechanical section properties. The extensive calibration work concluded in a modification of the fiber model. Applying the calibrated model to the shaking table tests by [Lestuzzi, 1999], the wall response could be computed with satisfying correspondence.

EXAMPLE BUILDINGS

In the following sections, the seismic evaluation of an example building using the methods and tools described above is presented. This building was selected from a population of 50 representative Swiss buildings. These buildings were selected on the basis of a survey of two towns with medium seismicity focused on reinforced concrete buildings built between 1945 and 1989. This survey confirmed that bearing wall buildings are the common construction type and that frame buildings are an exception in Switzerland. More results of this survey and on the typical construction techniques and materials in Switzerland in the investigated period are given in the companion paper titled "Evaluation of the Seismic Adequacy of Older Swiss R. C. Buildings" [Badoux, 2000].

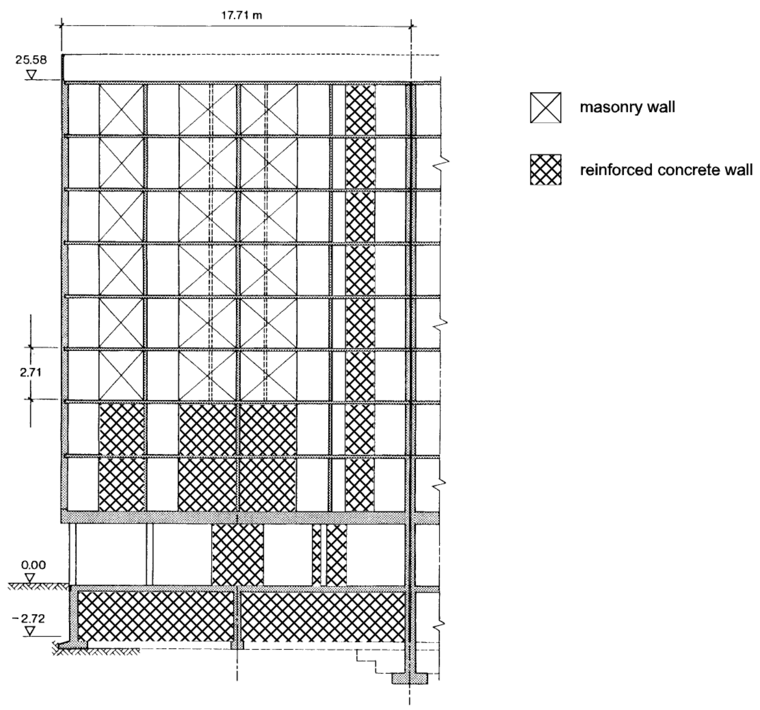

Figure 4.Longitudinal section of the example building KJA

Several buildings, with 3 to 9 stories, were studied in detail and nonlinear models based on the calibration were established. In this article we present a study on the response of one example building named KJA. It's a residential building with 9 stories above ground and one basement. It's 53.1 m long and 13.6 m large, separated by two transversal expansion joints in three blocks. A longitudinal section through one block is given in Figure 4. It shows the "cutted" structural elements and the structural walls behind the section's plane. Clearly the soft ground floor can be seen.

The structure consists of cast-in-place r.c. walls, r.c. columns and masonry walls. In all stories there are r.c. walls offering lateral resistance. The facades in longitudinal direction contain no structural walls but are formed by windows and non-structural elements. In transversal direction there are big walls without openings at the front and at the expansion joints. Under horizontal loading in longitudinal direction, the walls parallel to the longitudinal direction provide resistance and only few transverse walls interact with the walls in longitudinal direction. To compute seismically the building, a two dimensional model for the longitudinal direction was developed. It showed the following elastic eigenfrequencies: 1.4 Hz for the fundamental mode, 5.0 Hz for the second and 9.0 Hz for the third mode.

APPLICATION OF THE CAPACITY SPECTRUM METHOD TO THE EXAMPLE BUILDING

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The KJA building was seismically evaluated for an earthquake loading according EC8, for 0.16g peak acceleration and soft soil class C. For this earthquake loading, an artificial acceleration timehistory was generated that meets rather well the EC8 response spectrum [Lestuzzi, 1999]. Of this timehistory, the elastic response spectrum is given in Figure 5, left, with a viscous damping of 5% and 10%. Representing the building behaviour, the KJA capacity spectrum is shown. To apply the CSM, the capacity spectrum and the response spectrum in Figure 5 are to intersect. This gives a spectral target displacement of S_d = 35 mm (or 52 mm roof displacement) for the intersection with the 10% damped response spectrum and a spectral target displacement of S_d = 50 mm (72 mm roof displacement) for 5% damping. At present, no rule has been selected by the authors to estimate in the CSM the equivalent damping. Therefore, a NLDA with the EC8-acceleration timehistory was executed. The NLDA yielded a spectral target displacement of S_d = 33 mm (45 mm roof displacement) and we can conclude, that a good estimation for the equivalent damping is 10%.

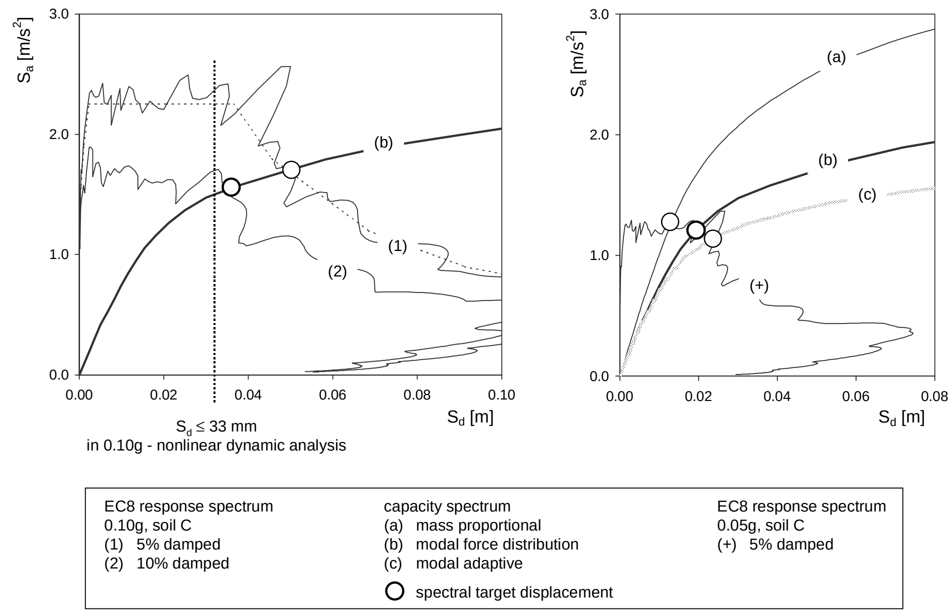

Figure 5.The capacity spectrum method applied on the example building KJA, left for the 0.10g EC8 response spectrum and right for the 0.05g EC8 response spectrum, both for soft soil class C. On the left, additionally the maximum spectral displacement (S_d = 33 mm) of the reference nonlinear dynamic analysis is given.

To study the lateral force distribution in the first step of the CSM, the push over analysis, three propositions were applied. The tested lateral force distributions are: (a) the "mass proportional" force distribution, where the lateral forces are proportional to the story masses, (b) the "modal" force distribution where the lateral forces are proportional to the product of the story mass and the story displacement in the fundamental mode, and (c) the "modal adaptive" push over analysis, that uses the same approach as (b), but in each computation step, the mode shapes are computed again and the lateral forces are newly distributed. The capacity spectra for the KJA building for these three load distributions are given in Figure 5, right. For the comparison of these load distributions, the KJA building is evaluated for an earthquake loading according to EC8 with 0.05g peak acceleration. The NLDA effected with this low impact showed few elements, where yielding just started and no important hysteretic damping has taken place. A 5% damping is therefore considered to be realistic. This should be shown by the

CSM, given in Figure 5, right, too. This figure shows for capacity spectrum (a), mass proportional, a straight line even above the intersection with the response spectrum and a fully elastic response would be concluded. For capacity spectrum (c), modal adaptive, the capacity spectrum shows a distinct curve between the origin and the intersection with the response spectrum and a corresponding inelastic structural response would be expected. Capacity spectrum (b) shows the beginning of a curve right at the intersection with the response spectrum, i.e. the inelastic response is just starting. This corresponds best with the results of the NLDA and the load distribution (b), "modal" force distribution, is the preferred one.

The KJA building was evaluated for an even stronger seismic impact than the EC8 earthquake, too: the strongest measured earthquake in the alpine region, the Friuli earthquake of may 6th, 1976, at the station Tolmezzo, a rock site [Smit, 1999]. Here, the longitudinal component is used with a peak ground acceleration of 0.36g (Figure 7, bottom right). The essential part of the strong motion lasts 4 seconds. (The artificial EC8-earthquake had a strong motion phase of about 10 seconds.)

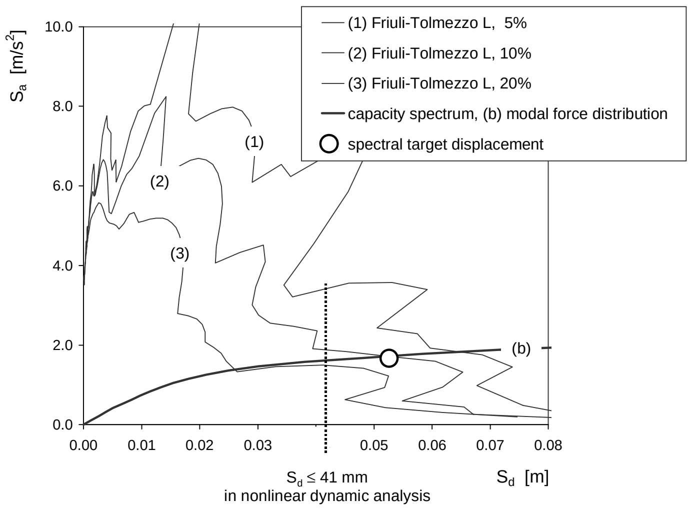

Figure 6.Capacity spectrum method of the example building KJA for the earthquake Friuli-Tolmezzo, longitudinal direction. With the maximum spectral displacement in the nonlinear dynamic analysis, the equivalent damping is estimated to be 15%.

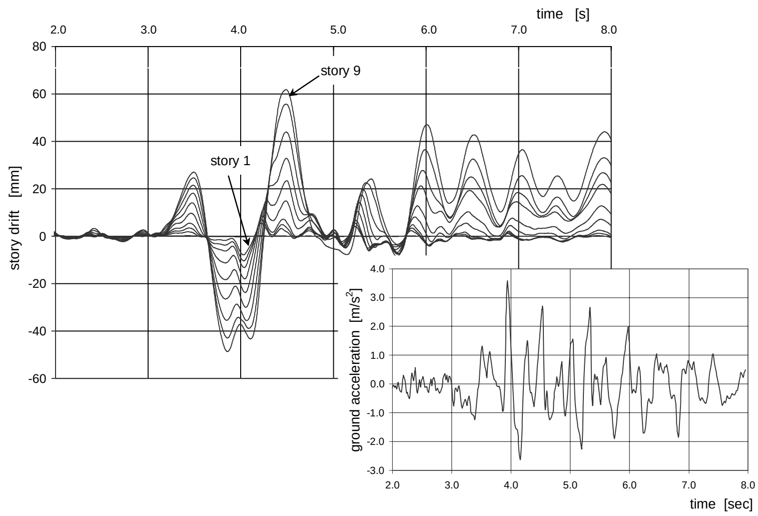

Figure 7.Story drifts in the nonlinear dynamic analysis of the example building KJA during the earthquake timehistory Friuli-Tolmezzo, longitudinal direction, shown at bottom on the right.

The CSM evaluation for the Friuli-Tolmezzo earthquake is shown in Figure 6. The estimated spectral target displacement is S_d = 52 mm for an equivalent damping of 10%. This corresponds to a roof displacement of approximately 78 mm. At that displacement level, the push over analysis shows significant nonlinear behaviour and important structural damage. This is confirmed by the NLDA shown in Figure 7. The maximum computed roof displacement is 61 mm (spectral displacement S_d = 41 mm).

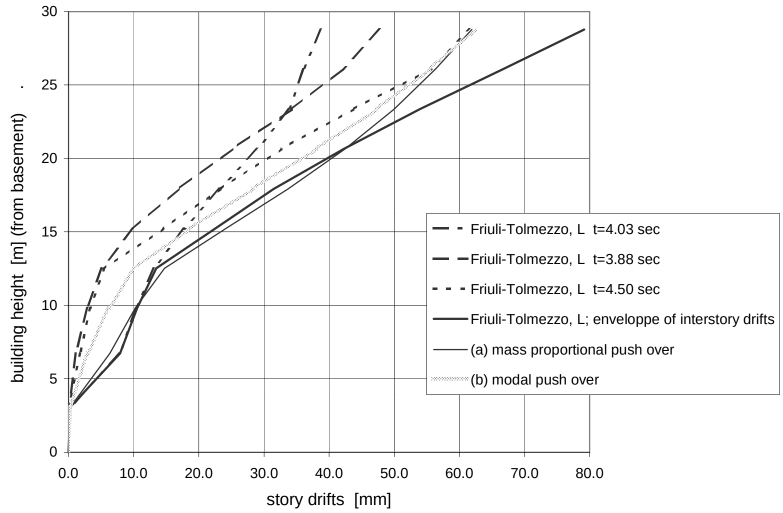

Figure 8.Story drifts of the example building during "Friuli-Tolmezzo", longitudinal direction and in the push over analyses using different lateral force distribution

The comparison of the story drifts shows that the shock-like impact at the beginning of the strong motion produces large interstory drifts in the soft ground floor. During the rest of the strong motion, this peak interstory drift in the ground floor is not reached anymore by far. Except in the ground floor, the CSM using the modal force distribution (b) covers the real maximum deformations quite well. With the mass proportional force distribution (a), it is possible to compute about the same interstory drift for the given global deformation state. With the modal force distribution (b) it is not possible to compute the big interstory drift in the ground floor at the beginning of the strong motion.

The influence of higher modes in this timehistory analysis is shown in Figure 8 as well: the sum of the maximum interstory drifts contains higher mode effects. But as the difference between this curve and the push over curve is not big (but mainly based on the different interstory drift in the ground floor) it can be concluded that the higher mode influence is of negligible importance.

CONCLUSIONS

The seismic response of a representative 9-story r.c. bearing wall building was calculated with the Capacity Spectrum Method (CSM) and Nonlinear Dynamic Analyses (NLDA). This study showed the following:

- As for frame type buildings, the CSM revealed to be a useful tool for the analysis of the seismic response of r.c. buildings with bearing walls. The analysed example shows, that the maximum seismic story and interstory drifts predicted with the CSM do correspond satisfyingly with those obtained with the NLDA.
- Of the different lateral story force distributions considered, best over all results were obtained with the modal force distribution. However, to predict deformation states occurring e.g. at the beginning of an earthquake impact (shock-like impacts or near-source effects) the mass proportional force distribution can be a useful complement.
- For the two strong earthquakes, the KJA building showed a substantial nonlinear behaviour. For both earthquakes, a good agreement between the CSM and the NLDA was found for an equivalent damping of 10%.
- For the application of displacement based analysis methods such as the CSM, reliable structural models are of a great importance. The trilinear hysteretic model is estimated to be a good basis for bearing wall buildings. The use of experimental results for calibration is still necessary for reliable analysis of wall structures.

- The evaluation of the example building confirms that even when they were not designed for seismic loading, r.c. buildings with bearing walls tend to display relatively high seismic resistance. This finding is in agreement with observations of the seismic performance of bearing wall buildings in many earthquakes [Fintel, 1995].

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