IAEE
Monograph
Vol. 2

The Story of the N2 Method

Peter Fajfar

International Association for Earthquake Engineering

The Story of the N2 Method

by

PETER FAJFAR

University of Ljubljana, Slovenia

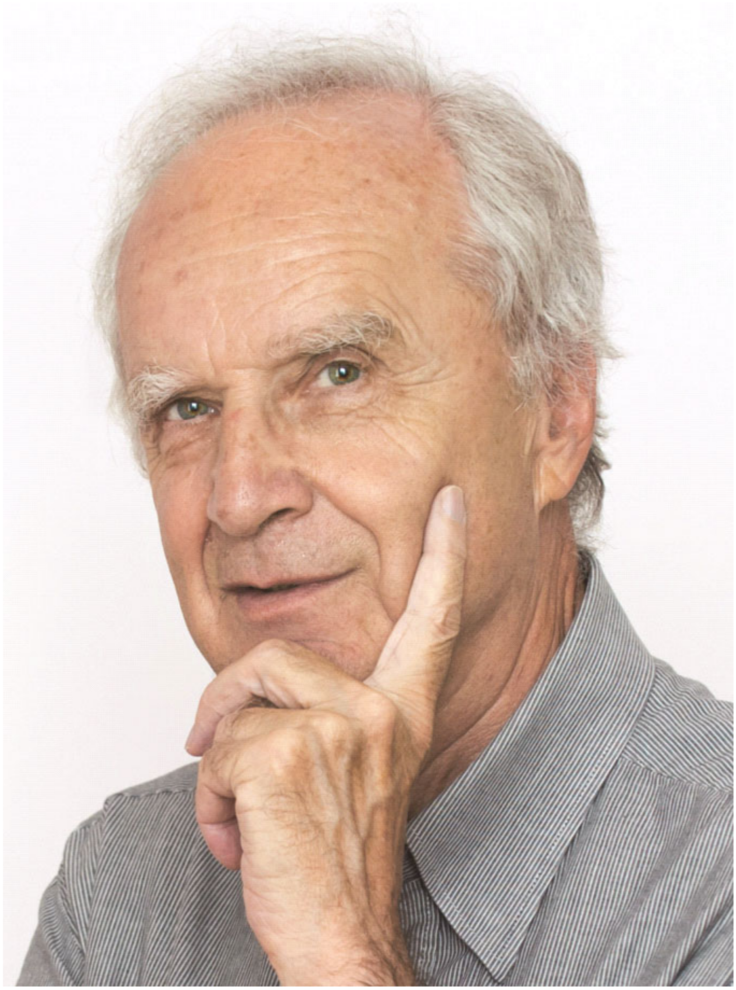
Computers and Structures, Inc.

International Association for Earthquake Engineering

IAEE Central Office

Kenchiku Kaikan (AIJ Building) 4th Floor
26-20, Shiba 5-chome, Minato-ku, Tokyo
108-0014, Japan

E-mail: secretary@iaee.or.jp
Website: www.iaee.or.jp



PETER FAJFAR

University of Ljubljana, Slovenia

Curriculum Vitae of Peter Fajfar

Education

BS, Civil Engineering, University of Ljubljana, Slovenia	1966
MS, Civil Engineering, University of Ljubljana, Slovenia	1972
Ph.D., Civil Engineering, University of Ljubljana, Slovenia	1974

Professional Appointments (selected)

Engineer at construction site, Ljubljana	1967-1968
Researcher, Lecturer, Professor, University of Ljubljana (UL)	1968-2018
Dean, Faculty of Architecture, Civil and Geodetic Engr., UL	1985-1987
Head, Institute of Structural Engr., Earthquake Engr. and Construction IT, UL	1985-1990 2001-2013
Visiting Researcher, Ruhr University, Bochum, West Germany	1972-1973
Visiting Researcher, UC Berkeley, USA	1980
Visiting Professor, McMaster University, Hamilton, Canada	1994
Visiting Professor, Stanford University, USA	1995
Visiting Professor, Bristol University, UK	2006
Visiting Professor, University of Canterbury, Christchurch, NZ	2009
President, Yugoslav Association of Earthquake Engineering	1984-1988
Founding President, Slovenian Assoc. of Earthquake Engr.	1988-1990
Member, Executive Committee of EAEE	2002-2010
Member, Board of Directors of IAEE	2004-2012
Editor, Structural Dynamics and Earthquake Engineering	2003-2015

Honors and Awards (selected)

Advisory Prof., Chongqing Institute of Arch. and Engr., PR China	1988
Member, Slovenian Academy of Sciences and Arts	1989
Republic of Slovenia Award for Science	1994
Founding Member, Slovenian Academy of Engineering	2001
Engineering Achievement Award, Slovenian Chamber of Engineers	2009
Honorary Member, EAEE	2010
Member, European Academy of Sciences (EURASC)	2011
Lifetime Achievement Award, Slovenian Chamber of Engineers	2013
Republic of Slovenia Award for Lifetime Achievement in Science	2015
Ambraseys Distinguished Lecture, EAEE	2018
Foreign Member, US National Academy of Engineering (NAE)	2018

PREFACE

On behalf of the International Association for Earthquake Engineering (IAEE), I am very pleased to announce that the IAEE launched a new initiative called "Masters Series" in 2018. The objective of this initiative is to connect the legendary figures in our discipline of earthquake engineering with those who will lead our discipline, now and in the future. The initiative consists of three categories, namely "Read the Masters," "Meet the Masters," and "Greet the Masters." Among these, "Read the Masters" is for a legend to write a monograph on the subject of his or her expertise and share his or her efforts and experiences with the next generations. The other two, "Meet the Masters" and "Greet the Masters," will connect our legends with the next generations during the World Conference on Earthquake Engineering, which is to be held once every four years in various parts of the world.

The very first two monographs produced by the "Read the Masters" program were published in 2021. One of the two monographs, published here and entitled "The Story of the N2 Method," is written by Professor Peter Fajfar. He was born and raised in Slovenia, studied at and earned his doctoral degree from the University of Ljubljana, and conducted research and education as a professor of his Alma Mater until he retired in 2018. Throughout his professional career Professor Fajfar led research and education on earthquake engineering, published about 100 technical articles in major archival journals, and has fostered a large number of next-generation researchers and practitioners. Through his extensive interactions with many researchers around the world he has played a major role in strengthening international collaboration among the earthquake engineering community. Professor Fajfar served as IAEE's Director between 2004 and 2012.

This monograph is his own narrative about the inception, development, and expansion of the method that Professor Fajfar nurtured for long years of his professional life. I hope that readers enjoy these writings on the accomplishments of this great master of earthquake engineering.

Masayoshi Nakashima
President of IAEE (2018-2022)

Contents

1 INTRODUCTION

In late summer 2018, Professor Masayoshi Nakashima, current president of the International Association for Earthquake Engineering (IAEE), asked me if I would be willing to contribute to the new IAEE initiative, "Read the Masters", with a short book on the topic of my choice. The invitation was a great honour, and I immediately felt that it was difficult to decline. Nevertheless, I first hesitated to accept it. It was a few months before my official retirement after almost 50 years of service, and my plan was to slow down. After thoughtful consideration, however, I realised that this book would be a unique opportunity to promote pushover-based analysis, which occupied a substantial part of my career. Moreover, it would force me to make an inventory of the main results obtained, and to prepare, in a single volume, a brief summary of the major achievements. Considering that the pushover-based N2 method has been implemented in the European standard for the design of structures for earthquake resistance, Eurocode 8 (EC8), and has been well accepted among many researchers and practitioners who may benefit from a monograph on this method, I decided to accept this professional challenge.

The leading role in this book is that of the N2 method. This is an analysis method, intended to achieve a satisfactory balance between required reliability and applicability for everyday design use. The development of the N2 method, its background, and its extensions are described, together with some insider information and memories, which illustrate the long and exciting path from initial ideas to code applications. The story of the N2 method shows that it is possible, with hard work, with good collaborators, and also with some luck, to contribute to the development of a field, even if one works far from the major research centres.

My home country of Slovenia is a region with moderate seismicity. According to the current seismic hazard map for the return period of 475 years, peak ground accelerations at firm soil are between 0.1g and 0.25g. The last major earthquake occurred in 1895, when about 10% of the buildings in the capital city Ljubljana were destroyed. The memory to this earthquake remained alive in early 1930s, when an 11-storey reinforced concrete building was built there. It was one of the first base-isolated buildings in the world. After the Second World War, however, the seismic problem was forgotten, until a few engineers drafted a seismic code that was adopted in Slovenia (at that time a part of former Yugoslavia) a month before the disastrous 1963 Skopje earthquake. After this earthquake, the Slovenian code was, with some changes, adopted as the Yugoslav code. The Skopje earthquake triggered some very limited work in earthquake engineering. However, it took considerable time before this work entered university curricula.

During my undergraduate studies in Ljubljana, I received very little information about earthquakes and their consequences. I became interested in seismic design in 1967, during my first job as an engineer at the construction site of a multistorey building, when I had a chance to see a simple seismic analysis for the building. Later, when I started my academic career at the University of Ljubljana at the invitation of Miloš Marinček, Professor of Steel Structures, I first worked on the inelastic behaviour of steel structural elements, which was essentially a continuation of the work that I had done for my undergraduate thesis. Only in the early 1970s did I have a chance to start doing some research related to earthquake engineering, the topic in which I was the most interested. The problem was that I did not have a proper mentor. Advanced dynamic structural analysis of buildings became feasible only with the appearance of computers, and at that time nobody in Slovenia was an expert in this field. Working in a completely new field was a hard and time-consuming but challenging and rewarding job. Initially, I had to learn everything from books and papers. Later, using both official and private channels, I managed to establish close contacts with colleagues all over the world. However, although the work in a small country distant from the main earthquake engineering centres certainly sets limits in professional development, I have never been tempted to leave my home town Ljubljana for an extended period. This is, or least used to be until very recently, typical for Slovenians, who like to stay at home. I am, in this respect, an extreme case, living from birth in the same house, located a walking distance from my office. Family ties, favourable working conditions, beautiful landscape and a good quality of life in Slovenia prevailed in my thinking over much greater professional opportunities in larger research centres worldwide.

"Choose a job you love, and you will never have to work a day in your life" is a statement, usually attributed to the ancient Chinese sage Confucius. Most of the time, I was fortunate to have a job that I liked. This was a necessary but not sufficient condition for achieving the results described in different sections of this book.

2 ON THE SEISMIC ANALYSIS OF STRUCTURES

2.1 Introduction

Seismic analysis is a tool for the estimation of structural response in the process of designing earthquake-resistant structures and/or retrofitting vulnerable existing structures. In principle, the problem is difficult because the structural response to strong earthquakes is dynamic, nonlinear, and random. All three characteristics are unusual in structural engineering, where the great majority of problems are (or at least can be adequately approximated as) static, linear, and deterministic. Consequently, special skills and data are needed for seismic design, which an average designer does not necessarily have.

After computers became widely available, (i.e., in the late 1960s and in 1970s), a rapid development of procedures for seismic analysis and supporting software occurred. Nowadays, due to tremendous developments in computing power, numerical methods, and software, there are almost no limits related to computation. Unfortunately, knowledge about ground motion and structural behaviour, especially in the inelastic range, has not advanced at the same rate. Also, we cannot expect that, in general, the basic capabilities of engineers will be better than in the past. So, there is a danger, as Mete Sozen wrote in 2002: "Today, ready access to versatile and powerful software enables the engineer to do more and think less." (Sozen, A Way of Thinking, see EERI Newsletter, April 2002.) Two other giants in earthquake engineering also made observations which have remained valid up to now. Ray Clough, one of the fathers of the finite element method, stated: "Depending on the validity of the assumptions made in reducing the physical problem to a numerical algorithm, the computer output may provide a detailed picture of the true physical behaviour or it may not even remotely resemble it. A controlling influence on where the final result lies along this scale is the skill of the engineer who prepares the mathematical idealization" (Clough 1980). Vitelmo Bertero (Reitherman 2009, p.80) warned: "There are some negative aspects to the reliance on computers that we should be concerned about. It is unfortunate that there has been a trend among the young practicing engineers, who are conducting structural analysis, design, and detailing using computers, to think that the computer automatically provides reliability". Today, it is lack of reliable data and the limited capabilities of designers that represent the weakest link in the chain representing the design process, rather than computational tools, as was the case in the past.

An indication of the restricted ability of the profession (on average) to adequately predict the seismic structural response was presented by the results of a blind prediction contest of a simple full-scale reinforced concrete bridge column with a concentrated mass at the top, subjected to six consecutive unidirectional ground motions. A description of the contest, and of the results obtained, described in the following text, has been summarized from (Terzic et al. 2015).

During the first ground motion, the column displaced within its elastic range. The second test initiated a nonlinear response of the column, whereas significant nonlinearity of the column response was observed during the third test. The column was not straightened or repaired between the tests. Each contestant/team had to predict peak response for global (displacement, acceleration, and residual displacement), intermediate (bending moment, shear, and axial force), and local (axial strain and curvature) response quantities for each earthquake. Predictions were submitted by 41 teams from 14 different countries. The contestants had either MSc or PhD degrees. They were supplied with data about the ground motions and structural details, including the complete dimensions of the test specimen, and the mechanical one-dimensional properties of the steel and concrete. In this way the largest sources of uncertainties, i.e., the characteristics of the ground motion and the material characteristics, were eliminated. The only remaining uncertainty was related to the modelling and analysis of the structural response. In spite of this fact, the results showed a very wide scatter in the blind predictions of the basic engineering response parameters. For example, the average coefficient of variation in predicting the maximum displacement and acceleration over the six ground motions was 39% and 48%, respectively. Biases in median predicted responses were significant, varying for the different tests from 5% to 35% for displacement, and from 25% to 118% for acceleration. More detailed results for the maximum displacements at the top of the column and the maximum shear forces at the base of the column are presented in Fig.2.1. A large dispersion of the results can be observed even in the case of the elastic (EQ 1, first test)) and nearly elastic (EQ 2, second test) structural behaviour.

The results of the blind prediction contest clearly demonstrate that the most advanced and sophisticated models and methods do not necessarily lead to adequate results. For example, it was observed that a comparable level of accuracy could be achieved if the column was modelled either with complex force-based fibre beam-column elements or with simpler beam-column elements with concentrated plastic hinges. Predictions of structural response in the contest greatly depended on the analyst's experience and modelling skills. Although both the ground motion and the material characteristics were known and thus the major uncertainties were eliminated, some of the results were completely invalid and could lead to gross errors if used in design. A simple check, e.g., with the response spectrum approach applied for a single-degree-of-freedom system, would indicate that the results were nonsensical.

While the most advanced analytical, numerical and experimental methods should be used in research aimed at the development of new knowledge and in the design of some special structures, the methods intended for everyday

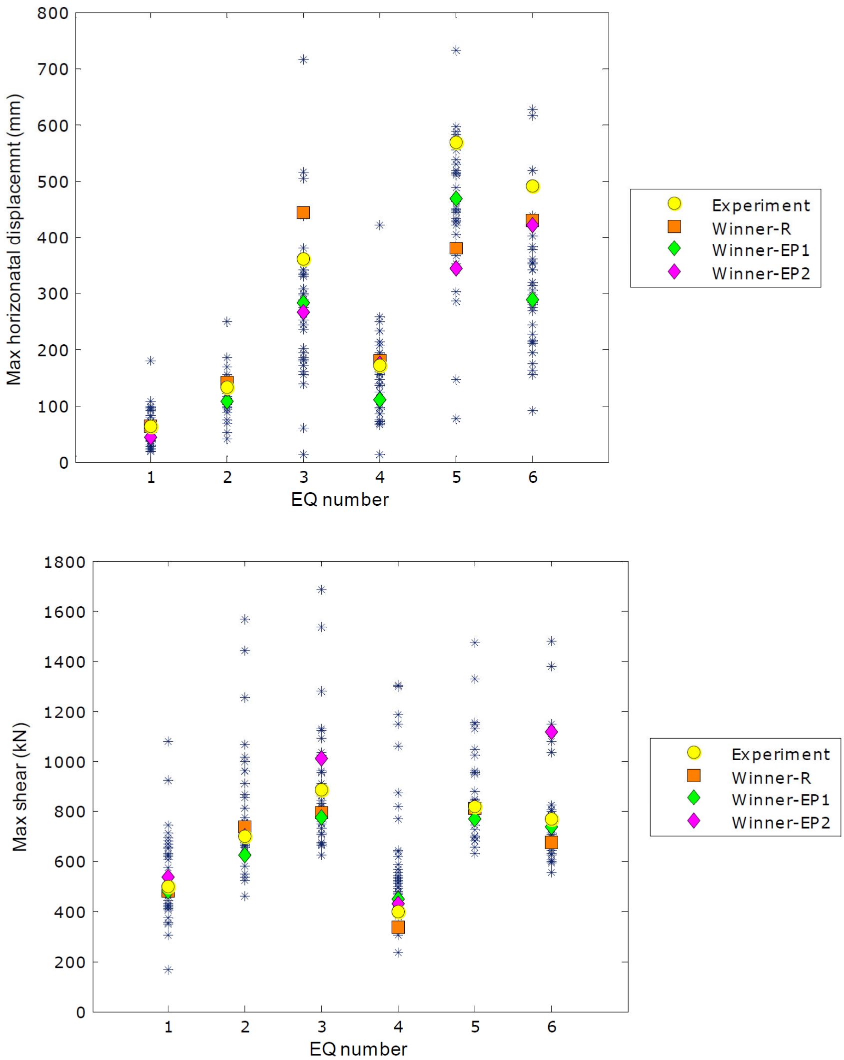


Figure 2.1. Predictions of maximum horizontal displacements at the top of the column and maximum base shears versus measured values. The predictions of the contest winners are indicated (from Terzic et al. 2015).

practical applications should, as Albert Einstein said, be "as simple as possible, but not simpler". A balance between the required accuracy and complexity of analysis should be found, depending on the importance of a structure and on the aim of the analysis. It should not be forgotten that the details of the ground motion during future earthquakes are unpredictable, and the details of the dynamic structural response, especially in the inelastic range, are highly uncertain. According to Aristotle, “It is the mark of an educated mind to rest satisfied with the degree of precision which the nature of the subject admits and not to seek exactness where only an approximation is possible.” (Nicomachean Ethics, Book One, Chapter 3). Of particular concern is the extremely high uncertainty in ground motion prediction, which does not decrease with increasing amounts of data. Very recently, I was upset by a statement by one of the top specialists in seismic hazard analysis (Abrahamson 2018): “Changes to the ground-motion models will likely lead to the largest changes in seismic hazard in the next five years. 2500-yr ground motions may go up or down by up to a factor of two.”

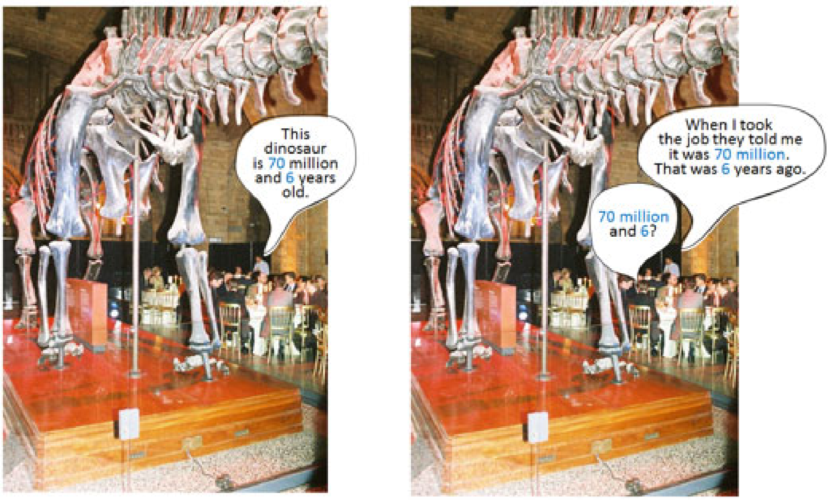


Figure 2.2. The joke about the age of dinosaurs.

I will conclude this introductory part of the chapter on analysis with a joke (see Fig. 2.2) that clearly demonstrates the nonsense of trying to be too “accurate”. Originally, I found this joke about the age of dinosaurs in a publication related to the US 2000 Bush-Gore presidential election (Bush won Florida’s electoral votes by a margin of only a few hundred votes out of almost six million cast and, as a result, became the president-elect.) I liked the joke, and I have occasionally included it in my lectures, hoping that some listeners would recall it when trying to perform a very “accurate” seismic analysis. For the first time, I used the joke in my keynote lecture at the 12th European Conference on Earthquake Engineering (ECEE) in London (Fajfar 2002). I asked a friend of mine to prepare three plots, and I used them as the last slide in my presentation. It was a pure coincidence that the conference dinner was in London's Natural History Museum, where the conference participants sat below a dinosaur. My lecture was the day after the dinner, and I told the joke as if it had happened last night. A large part of the audience believed that the story was true, and the joke actually proved to be a great success. Later, when the conference photos became available, I changed the background of the slide, using a photo taken at the London ECEE dinner (Fig. 2.2).

2.2 Nonlinear Analysis

Most buildings experience significant inelastic deformations when affected by strong earthquakes. However, there was a long way to go before the explicit nonlinear analysis found its way into practice and more advanced seismic codes. Initially, the most popular approach was the use of force reduction factors, and this approach remains popular today (Chapter 3). Although this concept for taking into account the influence of inelastic behaviour in linear analysis has served the profession well for several decades, a truly realistic assessment of structural behaviour in the inelastic range can be made only through nonlinear analysis.

For nonlinear analysis, data about the structure have to be known, so it is very well suited for the analysis of existing structures. In the case of newly designed structures, a preliminary design has to be made before starting a nonlinear analysis. Typical structural response measures (also called “engineering demand parameters”) that form the output from such an analysis are: the storey drifts, the deformations of the “deformation-controlled” components, and the force demands in “force-controlled” (i.e., brittle) components that, in contemporary buildings, are expected to remain elastic.

Nonlinear response history analysis (NRHA) is the most advanced deterministic analysis method available today. It represents a rigorous approach with a sound theoretical background and is irreplaceable for the research and for the design or assessment of important structures. However, due to its complexity, it has, in practice, rarely been used for common structures. NRHA is not only computationally demanding (a problem whose importance has been gradually reduced due to the development of advanced hardware and software), but also requires additional data, which are not needed in pushover-based nonlinear analysis: a suite of accelerograms, and data about the hysteretic behaviour of structural elements. A consensus about the proper way to model viscous damping, in the case of inelastic structural response, has not yet been reached. A wide range of assumptions is needed in all steps of the process, from ground motion selection to nonlinear modelling. Many of these assumptions are based on the analyst’s judgement. Moreover, the complete analysis procedure is less transparent than in the case of simpler methods. For all these reasons, the great majority of codes that permit the use of NRHA require an independent review of the results of such analyses.

According to Krawinkler (2006), “In concept, the simplest method that achieves the intended objective is the best one. The more complex the nonlinear analysis method, the more ambiguous the decision and interpretation process is. . . . Good and complex are not synonymous, and in many cases they are conflicting.”

Analysis procedures, intended to achieve a satisfactory balance between required reliability and applicability for everyday design use, are pushover-based methods (Section 2.3), among them is the N2 method.

2.3 Pushover-based Analysis

Sigmund Freeman, a practising engineer from the USA, can be considered the "father" of pushover-based seismic analysis which was first introduced in the 1970s as a rapid evaluation procedure (Freeman et al. 1975). In the 1980s, it was named the "Capacity Spectrum Method" (CSM). The method was also developed into a design verification procedure for the Tri-services (Army, Navy, and Air Force) "Seismic design guidelines for essential buildings" manual (Army 1986). In order to account for the nonlinear inelastic behaviour of a structural system, effective viscous damping values were applied to the linear-elastic response spectrum (i.e., an "overdamped spectrum") in all CSM formulations. In the N2 method, developed in late 1980s, inelastic spectra were used instead of overdamped elastic spectra. An important milestone was the paper by Mahaney et al. (1993, Freeman was a co-author) in which the acceleration-displacement response spectrum (ADRS, called "AD" in this book) format was introduced, enabling the visualisation of the assessment procedure. In 1996, CSM was adopted in the ATC 40 guidelines "Seismic evaluation and retrofit of concrete buildings" (ATC 1996). In FEMA 273 (FEMA 1997), the target displacement was determined by the "Coefficient Method". This approach, which has remained in all following FEMA documents, and has also been adopted in the ASCE 41-13 standard (ASCE 2014), resembles the use of inelastic spectra. In the United States and elsewhere, the use of pushover-based procedures has accelerated since the publication of the ATC 40 and FEMA 273 documents. A comprehensive summary of pushover analysis procedures was provided by Aydinoğlu and Önem (2010). Nowadays, the popularity of pushover-based methods seems to be declining in the US, whereas it is increasing in many other parts of the world.

A simple pushover approach, which could be applied at the storey level and used for the analysis of the seismic resistance of low-rise masonry buildings, was developed in the late 1970s by Tomaževič (1978). This approach was also adopted in a regional code for the retrofitting of masonry buildings after the 1976 Friuli earthquake in the Italian region of Friuli-Venezia Guilia (Regione Autonoma Friuli-Venezia Giulia 1977).

Pushover-based methods combine nonlinear static (i.e., pushover) analysis with the response spectrum approach. Seismic demand can be determined for an equivalent single-degree-of-freedom (SDOF) system from an inelastic response spectrum (or an overdamped elastic response spectrum). A transformation of the multi-degree-of-freedom (MDOF) system to an equivalent SDOF system is needed. This transformation, which represents the main limitation of the applicability of pushover-based methods, would be exact only in the case that the analysed structure vibrated in a single mode with a deformation shape that did not change over time. This condition is, however, fulfilled only in the case of a linear elastic structure with the negligible influence of higher modes. Nevertheless, the assumption of a single time-invariant mode is used in basic pushover-based methods for inelastic structures, as an approximation.

Pushover-based analyses can be used as a rational practice-oriented tool for seismic analysis. Compared to traditional elastic analyses, this kind of analysis provides a wealth of additional important information about the expected structural response, as well as a helpful insight into the structural aspects that determine performance during severe earthquakes. Pushover-based analyses provide data on the strength and ductility of structures, which cannot be obtained by elastic analysis. Furthermore, they are able to expose design weaknesses that could remain hidden in an elastic analysis. This means that, in most cases, they are able to detect the most critical parts of a structure. However, particular attention should be paid to potential brittle failures, which are usually not simulated in the structural models. The results of pushover analysis must be checked in order to determine if a brittle failure controls the capacity of the structure.

For practical applications and educational purposes, graphical displays of the procedure are extremely important, even when all the results can be obtained numerically. Pushover-based methods experienced a breakthrough when the acceleration-displacement (AD) format was implemented (Mahaney et al. 1993).

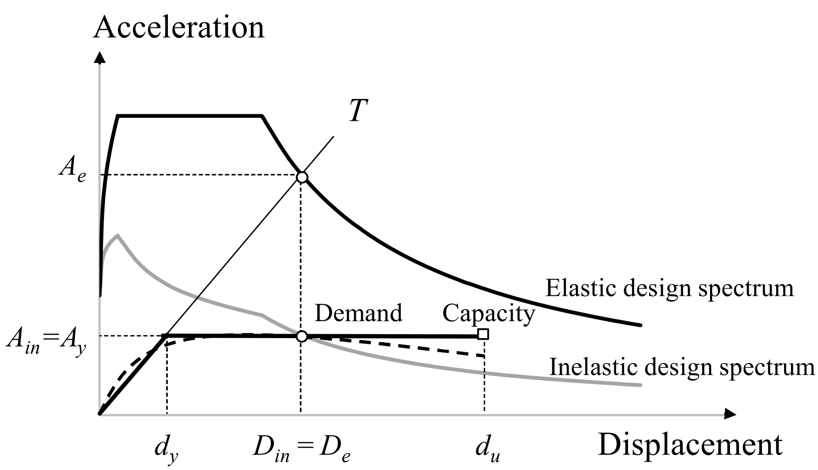


Figure 2.3. Comparison of demand and capacity in the acceleration – displacement (AD) format. Equal displacement rule is assumed.

Presented graphically in AD format (Fig. 2.3), pushover-based analyses can help designers and researchers to better understand the basic relations between seismic demand and capacity, and between the main structural parameters determining the seismic performance (i.e., stiffness, strength, deformation and ductility). They are a very useful educational tool for the familiarising of students and practising engineers with general nonlinear seismic behaviour, and with the seismic demand and capacity concepts. A graph like the one in Fig. 2.3 is, in my opinion, one of the most important and useful graphs in earthquake engineering. (Note that, in this book, the lower-case letter d is used to indicate displacements if they relate to a particular system, e.g., in the case of yield displacement or ultimate displacement capacity. In the case of spectral displacements, e.g., for seismic demand in terms of spectral displacements, the capital letter D is used.)

Pushover-based methods are usually applied for the performance evaluation of a known structure (i.e., an existing structure or a newly designed one). However, other types of analysis can, in principle, also be applied and visualised in the AD format, as discussed in Section 4.3.

Compared to NRHA, pushover-based methods are a much simpler and more transparent tool, requiring simpler input data. The amount of computation time is only a fraction of that required by NRHA, and the use of the results obtained is straightforward. Of course, the above-listed advantages of pushover-based methods have to be weighed against their lower accuracy compared to NRHA, and against their limitations. It should be noted that pushover-based analyses are approximate in nature, and based on static loading. They have no strict theoretical background. Like any approximate method, they are based on a number of assumptions. It cannot be expected that they will accurately predict the seismic demand for any structure and any ground motion. In spite of extensions like those discussed in Chapter 12, they may not provide acceptable results in the case of some building structures with important influences of higher modes, including torsion. For example, they may detect only the first local mechanism that will form, while not exposing other weaknesses that will be generated when the structure's dynamic characteristics change after formation of the first local mechanism. Despite these shortcomings, a pushover-based analysis is an excellent tool for understanding inelastic structural behaviour. When used for quantification purposes, the appropriate limitations should be observed. Additional discussion on the advantages, disadvantages and limitations of pushover analysis is available in, for instance, the publications of Krawinkler and Seneviratna (1998), Fajfar (2000), and Krawinkler (2006).

2.4 Equal Displacement Rule

The so-called "equal displacement rule" is basically an assumption based on empirical observations. It enables the development of several simplified procedures in nonlinear analysis and also allows their very clear graphic representation. Therefore, it greatly facilitates the understanding of the nonlinear response of structures.

In their seminal paper presented at the 2nd World Conference on Earthquake Engineering (WCEE), Veletsos and Newmark (1960) studied how inelastic structural behaviour can effectively reduce the lateral force coefficients that may be used in seismic design. They stated "one of the possibilities is to relate the spectrum for the elasto-plastic system to that for the corresponding elastic system by considering the maximum relative displacements for the two systems to be equal". This statement can be considered to be the birth of the equal displacement rule. According to this empirical rule, displacement of an inelastic SDOF structure is approximately equal to the displacement of the corresponding linear elastic structure with the same period (i.e., the same stiffness and mass). Since 1960, this empirical observation has been repeatedly confirmed as a reasonable approximation for a large number of structures. Nevertheless, some researchers question the validity of the equal displacement rule, claiming that, in general, it is impossible to construct two structures with the same stiffness but with different strengths. The fact is that this is absolutely not needed. The equal displacement rule should be regarded as a computational tool for determining the displacement of an inelastic structure.

As an example, let us consider an elastic design spectrum in acceleration-displacement (AD) format (Fig 2.3). We would like to determine the displacement of an inelastic structure with an idealised force-deformation diagram with strength $F_y = A_y m$, when subjected to the design ground motion. First, we will determine the displacement of a fictitious structure with the same period, but with unlimited linear elastic behaviour. This displacement d_e is defined by the crossing point of the period line and the elastic spectrum. Assuming the equal displacement rule, the displacement of the real inelastic system d_{in} is the same as the displacement of the fictitious elastic system.

Experience has shown that using the equal displacement rule is a viable approach for many structures with the fundamental period in the medium or long-period range, especially if the structure is located on firm sites and has relatively stable and full hysteretic loops. It cannot be applied for short-period structures, for which the inelastic displacement is larger than the corresponding elastic displacement. Also, the equal displacement rule may yield too small inelastic displacements in the case of near-fault ground motions, hysteretic loops with significant pinching or significant stiffness and/or strength deterioration, and for systems with low strength (i.e., with a yield strength to required elastic strength ratio of less than 0.2). Moreover, the equal displacement rule seems to be not satisfactory in the case of extremely narrow-band ground motions, like those recorded on very soft soil deposits.

In the N2 method, as implemented in Eurocode 8 (EC8, CEN 2004), the equal displacement rule is used for structures with the fundamental period in the medium- and long-period range (see Section 5.4).

3 FORCE REDUCTION FACTOR

Experience has shown that the great majority of well-designed and -constructed buildings survive strong ground motions, even if they were designed for only a fraction of the forces that would develop if the structure behaved as if it was linearly elastic. A reduction of seismic forces is possible mainly due to the beneficial effects of energy dissipation in ductile structures and inherent overstrength. The influence of the structural system and its capacity for energy dissipation was recognized in the late 1950s (e.g., Housner 1956). In their paper presented at the 2nd WCEE, Veletsos and Newmark (1960) wrote "It is the purpose of this paper to indicate . . . how inelastic behavior can effectively reduce the lateral force coefficients that may be used in design to values of the order of one-fourth or less of those which would be applicable for elastic systems."

However, it was only in 1978 that the force reduction factor (or simply *R* factor) in the current format was first proposed in ATC-3-06 document "Tentative provisions for the development of seismic regulations for buildings" (ATC 1978). Since then, the *R* factor has been present, in various forms, in all seismic regulations (in EC8, it is called the behaviour factor *q*).

R factor allows, in a standard linear analysis, an approximate consideration of the favourable effects of the nonlinear behaviour of the structure, and, therefore, presents a very simple and practical tool for seismic design. However, it is necessary to bear in mind that describing a complex phenomenon of inelastic behaviour for a particular structure, by means of a single average number can be confusing and misleading. For this reason, the *R* factor approach, although it is very convenient for practical applications and has served the professional community well over decades, is able to provide only very rough answers to the problems encountered in seismic analysis and design. Also, it should be noted that "the values of *R* must be chosen and used with judgement", as stated in the Commentary to the ATC 03-6 document in Sec. 3.1. According to ATC-19 (ATC 1995),

> The R factors for the various framing systems included in the ATC-3-06 report were selected through committee consensus on the basis of (a) the general observed performance of like buildings during past earthquakes, (b) estimates of general system toughness, and (c) estimates of the amount of damping present during inelastic response. Thus, there is little technical basis for the values of R proposed in ATC-3-06.

Nevertheless, the order of magnitude of *R* factors (1.5 to 8, related to design at the strength level) has been widely used in many codes and has remained more or less unchanged.

Although the R factor approach is much simpler (and cruder) than a pushover-based analysis, there are some similarities, especially in the graphical representation of both approaches. This was one reason for our interest in the background of the R factors. Understanding the pushover-based methods aids in comprehending the physical basis of the R factors and vice versa. An important conceptual difference between the two approaches is the notion of overstrength. In pushover analysis, the actual strength, which is practically always larger than the design strength, is estimated by analysis. There is no need to care about the influence of the overstrength. In the R factor approach, the overstrength has to be explicitly taken into account. Bertero called overstrength "a blessing because of which structures designed according to presently specified design seismic forces are able or would be able to withstand maximum credible earthquake shaking safely" (Bertero 1986). Only with overstrength was it possible to justify the quite large values of R factors in the US codes, which could not be attributed solely to ductility. Considering this fact, in the late 1980s, Bertero and other Berkeley researchers proposed splitting R into three factors that account for contributions of overstrength, ductility, and viscous damping. Later, in (ATC 1995), a new formulation was proposed, in which the viscous damping factor was replaced by the redundancy factor.

In the late 1980s, much research on overstrength was done in our research group. In our view, redundancy is a part of overstrength, so our proposal (Fischinger and Fajfar 1990) was to define the R factor as a product of only two factors, the ductility dependent factor R_μ and the overstrength factor R_s:

$$R = R_\mu R_s \tag{3.1}$$

The same formulation was independently proposed by Uang (1991), one of Bertero's students.

The advantage of Eq. 3.1 is a simple graphical representation that clearly shows the two influences (Fig. 3.1). Let us consider two idealized SDOF structural systems with the same mass and stiffness, i.e., with the same natural period. One system shows unlimited elastic behaviour, whereas the other has limited strength. The yielding point of the latter (inelastic) system is defined by the yield strength f_y and the yield displacement d_y. The corresponding idealised force-displacement relationships are shown in Fig. 3.1. The systems in Fig. 3.1 can accommodate the imposed seismic demand d either by large strength f_e (elastic system) or by a combination of smaller strength f_y and inelastic deformation capacity, defined by a ductility factor $\mu = d/d_y$ (yielding system). Note, however, that the reduction of strength may be conditioned not only by the available inelastic deformation capacity but also by the intent to limit damage in more frequent earthquakes.

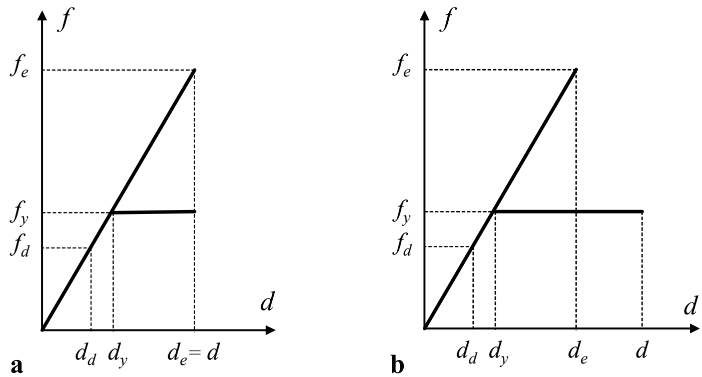


Figure 3.1. Idealised force-displacement (f-d) relationships. (a) Equal displacement rule applies. (b) Equal displacement rule does not apply.

The ductility-dependent reduction factor R_μ, which determines the extent of possible reduction of the strength due to the inelastic deformation capacity, is defined as:

$$R_\mu = \frac{f_e}{f_y} \tag{3.2}$$

The problem can also be stated in a different way. Assuming that an inelastic deformation capacity defined by the ductility factor μ is provided and tolerated, the strength of the system should be equal at least to the required strength f_y, which represents the inelastic strength demand. This approach is used in the design and can be written in the form:

$$f_y = \frac{f_e}{R_\mu} \tag{3.3}$$

where f_e is the elastic strength demand, i.e., the strength required for a structure that would remain in the elastic region during earthquake ground motion with a displacement demand d_e. The displacement demand d_e, i.e., the maximum relative displacement of the system with unlimited elastic behaviour, and the related elastic strength demand can be obtained from the elastic acceleration spectrum as described in Section 5.2.

Expressions similar to Eq. 3.3 can be found in various seismic standards and codes. However, an important difference should be noted between Eq. 3.3 and the expressions in the standards and codes. In Eq. 3.3, f_y represents the actual strength, whereas the seismic forces in standards and codes correspond to the design strength f_d which is, as a rule, lower than the actual strength. This difference is mainly due to overstrength, which is an inherent property of properly designed, detailed, constructed, and maintained highly redundant structures. Taking into account the overstrength factor:

$$R_s = \frac{f_y}{f_d} \tag{3.4}$$

Eq. 3.1 can be derived:

$$R = \frac{f_e}{f_d} = \frac{f_e}{f_y}\frac{f_y}{f_d} = R_\mu R_s \tag{3.1}$$

Thus the total force reduction factor R, which is equal to the elastic strength demand f_e divided by the code prescribed seismic design action (force) f_d, can be defined as the product of the ductility dependent factor R_μ and the overstrength factor R_s. The seismic design force f_d can be obtained from the elastic strength demand as:

$$f_d = \frac{f_e}{R} \tag{3.5}$$

where R is the reduction factor defined in Eq. 3.1.

Using Fig. 3.1b (which represents a general case) and the relation $f_e/f_y = R_\mu = d_e/d_y$, the inelastic displacement demand d can be determined from elastic displacement demand d_d as:

$$d = \frac{\mu}{R_\mu} d_e \tag{3.6}$$

An alternative form of Eq. 3.6 is:

$$d = \mu R_s d_d \tag{3.7}$$

where d_d is the maximum relative displacement of the system obtained by linear analysis under the design loads f_d.

For the determination of inelastic strength demand f_y (Eq. 3.3) and inelastic displacement demand d (Eq. 3.6), the ductility dependent reduction factor R_μ has to be known. If the equal displacement rule is assumed to apply, i.e., the displacements of the elastic and yielding systems are equal ($d = d_e$, Fig. 3.1a), the ductility dependent reduction factor R_μ is equal to the ductility factor μ (see Fig. 3.1a and Eq. 3.2):

$$R_\mu = \mu \tag{3.8}$$

In this case, Eq. 3.7 can be written as:

$$d = Rd_d \tag{3.9}$$

In the more general case, when the equal displacement rule does not apply, e.g., for short-period structures (Fig. 3.1b), a more general relation between the ductility factor μ and the reduction factor R_μ can be developed. Such a relationship is typically dependent on the period T and is often called the R_μ-μ-T relation (see Chapter 5).

The overstrength factor R_s is defined at the level of the whole structure, as the ratio between the actual strength and the code prescribed strength demand arising from the application of prescribed loads and forces. It results from several sources (see, e.g., Fardis et al. 2015). Some of them, e.g., redistribution of internal forces in the inelastic range in ductile, statically indeterminate (redundant) structures, can easily be at least approximately quantified by a nonlinear pushover analysis, whereas some others (e.g., higher material strength than the nominal one specified in design) are uncertain and difficult to be quantified.

Our work on R factors was presented in (Fischinger and Fajfar 1990, 1994). The spectra for the ductility dependent part of the R_μ factor, i.e., the R_μ-μ-T relations, were proposed in (Vidic et al. 1994) and some subsequent publications (see Chapter 5). Due to different interpretations and some misunderstandings related to overstrength factors, "Notes on definitions of overstrength factors" were prepared for the 2nd Bled workshop (Fajfar and Paulay 1997).

Unfortunately, Eq. 3.1 has often been misused by different researchers attempting to determine the R factors experimentally and/or numerically. When determining the value of a force reduction factor to be used in a code, it is of paramount importance to take into account an appropriate value of the displacement and ductility, which control the ductility dependent part of the reduction factor R_μ. The difference between the return period of the design ground motion and the target return period of failure has to be taken into account, as well as uncertainties. It is certainly not correct to consider, for example, the mean value of ultimate ductility obtained by experiments or numerical analyses, and to use it in combination with the design demand related to significant damage (SD) limit state. A quantification of R factors is possible by using the concept of "risk-targeted" safety and reduction factor, which was introduced and elaborated by Dolšek and his doctoral students (Dolšek et al. 2017, Žižmond and Dolšek, 2017, 2019, see also Section 14.2). A brief summary is also presented in (Fajfar 2018). A quite complex probabilistic procedure for determining values of reduction factors was used in FEMA P-695 (2009).

4 DEVELOPMENT OF THE N2 METHOD

4.1 Beginnings

After initial work on the elastic analysis of multi-storey buildings (see Chapter 8), I became interested in nonlinear analysis. With my first master's (and later PhD) student, Matej Fischinger, we started working in this field in the late 1970s. Matej's master's thesis, entitled "Computation of inelastic structural response", completed in 1980, contained an overview of our initial research on nonlinear analysis, which was strongly influenced by the work of Graham Powell from UC Berkeley and Shunsuke Otani, a PhD student of Mete Sozen from the University of Illinois. For analysing the inelastic response of reinforced concrete (RC) structures, we mostly used DRAIN-2D, whereas we developed our software for computing inelastic spectra. Our research was directed at the parametric studies of inelastic response of SDOF structures (see Chapter 5), mathematical modelling, and the possibilities for practical applications of nonlinear analysis of structures. For example, we applied nonlinear analysis for simulating the structural response of a building damaged during the 1979 Montenegro earthquake (Fajfar et al. 1981). At that time, fortunately, there was not yet the pressure to publish or perish. We presented the results at several international conferences, but we believed that our work in the early 1980s was not yet sufficiently developed to be published in an international archival journal. The story of the N2 method started with the paper that Matej and I published within the newly established journal European Earthquake Engineering (EEE) (Fajfar and Fischinger 1987). It appeared in the very first issue of the new journal; the start of a new international journal was a good opportunity to get a paper published quickly. EEE, published by an Italian company, was, from 1992 to 2002, the official journal of the European Association for Earthquake Engineering (EAEE). Unfortunately, it did not receive enough attention outside of Europe and ceased publication in 2007.

The idea for the N2 method came from the paper published by Saiidi and Sozen (1981). In this paper, the so-called Q model was proposed. Two types of simplification were involved in the model: "(1) reduction of a MDOF model of a structure to a SDOF oscillator, and (2) approximation of the varying incremental stiffness properties of the entire structure by a single nonlinear spring." Pushover analysis was used to determine the backbone of the force-displacement relationship of the nonlinear spring. Special hysteretic rules were developed for simulating the cyclic behaviour of the spring. Later, the name "Q model" was often used in a narrower sense only for the hysteretic rules. The displacement history of the structure was estimated by performing a nonlinear response history analysis of the SDOF system. The Q model has been frequently applied in research within the

Sozen group at the University of Illinois at Urbana-Champaign.

In our 1987 paper, all the main ideas of the simple nonlinear pushover-based procedure, which we called the "N2 method", were provided. The letter **N** came from **N**onlinear, and the number **2** came from **two** mathematical models. Unfortunately, the proposal for the new method was hidden under the title "Non-linear seismic analysis of RC buildings: implications of a case study". In this paper, the non-linear seismic response of a seven-storey RC frame-wall building was investigated analytically and compared with the experimental results obtained within the framework of a joint US-Japan research project in Tsukuba. This building was quite often used in our early research. I became familiar with the US-Japan research project and with this building during my visit to the Building Research Institute (BRI) in Tsukuba in 1982. Two methods of analysis were applied: a) the nonlinear dynamic analysis of an MDOF system, and b) the N2 approach including the nonlinear static analysis of the same MDOF system and the nonlinear dynamic analysis of an equivalent SDOF system. It was stated that the dynamic analysis of the SDOF system could be additionally simplified by using inelastic response spectra.

A part of the Introduction of the paper, reproduced below, which explains the need for a simplified nonlinear procedure and outlines the analysis procedure, remains fully relevant today, more than three decades after the paper was published:

> For the rational aseismic design of buildings, a procedure is needed which would, firstly, yield an adequate estimate of the structural stiffness, strength, and ductility supply, as well as of the ductility demand during an expected earthquake, and would also, secondly, not be more complicated than necessary regarding the uncertainties connected with the input data. A promising method which seems to fulfil both requirements is a non-linear procedure using two different mathematical models (N2). It is applicable for structures oscillating predominantly in a single mode. In the first step of the N2 method, stiffness, strength and supplied ductility are determined by the non-linear static analysis of an MDOF system under a monotonically increasing lateral load. Then, in the second step, an equivalent SDOF system is defined. In the third step of N2, maximum displacements (and the corresponding ductility demand) are determined by carrying out non-linear dynamic analysis of the equivalent SDOF system. Dynamic analysis, in its simplest way, can also be performed by using inelastic response spectra. In addition to the maximum displacement and/or displacement ductility demand, other important parameters can be determined, e.g., parameters connected with energy. The non-linear characteristics of the equivalent system are based on the base shear (or base moment) - top displacement relationship, obtained by the non-linear static analysis in the first step. By comparing ductility supply and demand,

> global structural behaviour during an earthquake can be estimated. Sometimes the storey drifts alone provide the designer with sufficient information to judge the acceptability of a structure. Some more details of the structural response (e.g., formation of plastic hinges, inelastic behaviour of different structural elements) can be obtained by following the inelastic static response up to the maximum displacements determined by the non-linear dynamic analysis of the SDOF model.

Further, it was stated in the Introduction:

> Note the analogy with the commonly used elastic procedure for seismic analysis of buildings, where two different mathematical models are usually used, too. Spectral modal analysis is typically performed by using a much simpler mathematical model than that used for the static analysis at the beginning (computation of the stiffness matrix) and end of the complete analysis procedure (computation of member forces corresponding to the maximum displacements determined by modal analysis).

In the year following the first publication related to the N2 method, Matej and I prepared a shorter version of the paper for the 9th WCEE which was held in Tokyo and Kyoto (Fajfar and Fischinger 1989). This time we put the name of the method in the title, which was "N2 – a method for non-linear seismic analysis of regular buildings". This paper received much greater attention than the original paper in EEE, most probably also because the WCEE proceedings were and are more widely available than the EEE journal. In the monograph "The IAEE at fifty" (Gülkan and Reitherman 2012), this paper was the only paper explicitly mentioned in relation to the 9th WCEE: "Reliable tools needed to be developed for building performance assessment. The N2 method of Fajfar for that purpose made a debut during this conference."

In both early papers on the N2 method, we also used, in addition to the response history analysis of the equivalent SDOF system, early displacement spectra developed in our research group.

In the doctoral thesis of Matej Fischinger entitled "Inelastic dynamic analysis of reinforced concrete structures – Development of design methods", defended in 1989, a summary of our early work on inelastic analysis was made. A special chapter was devoted to the N2 method.

4.2 "Mature" Version

After 1988, we continued with work on the N2 method, in which the major emphasis was given to the development of general inelastic spectra (see Chapter 5). Only with reasonably reliable and simple inelastic spectra could the N2 method have a chance to be accepted in practice. In addition, many efforts were made to incorporate the possibility of considering cumulative damage, in addition to the standard parameter measuring inelastic capacity and demand, i.e., displacement (see Chapter 7). Considerable work was accomplished by Peter Gašperšič in his master's and doctoral theses. He was one of my best PhD students, distinguished by an analytical mind and sound engineering judgement. Unfortunately, after obtaining his PhD, Peter was not interested in continuing academic work. He preferred working in industry and later in state administration, and became the Minister for Infrastructure in the Slovenian government from 2014 to 2018.

A "mature" version of the N2 method was presented in our paper in Earthquake Engineering and Structural Dynamics (EESD, Fajfar and Gašperšič 1996). The spectra, developed in (Vidic et al. 1994 and Fajfar and Vidic, 1994, see Chapters 5 and 7) were used. The method, as described in our 1996 paper, was still limited to planar building models, although a simplified pushover analysis of the spatial model had already been developed (Kilar and Fajfar 1997, see Section 8.4).

Capacity issues were not discussed in our paper. However, some references were provided for resources on the ultimate rotation capacity of RC structural elements. The rotation and rotation ductility demands in different structural members were assumed to be approximately equal to the corresponding demands in the MDOF model at a static top displacement corresponding to the maximum displacement of the equivalent SDOF model. The energy demand was included implicitly in parameter γ (Chapter 7). With knowledge of the seismic demand and capacity for each structural member, a damage index can be computed for each member. If needed, a single damage index can be determined for the structure as a whole, based on the weighted average of damage indices for its structural elements.

Several important papers related to the N2 method published prior to and including 1996 were collected in the publication entitled "Towards a new seismic design methodology for buildings" (Fajfar 1996) and distributed among my colleagues around the world, including the participants of the second Bled workshop. In the foreword, I wrote:

> This publication contains a selection of papers published by the research group working in the field of earthquake engineering at the University of Ljubljana . . . The papers have contributed, directly or indirectly, to the development of a rational methodology which should be applicable to the seismic design of new structures, as well as to the seismic evaluation of existing structures. . . . The research work of the group at the University of Ljubljana, although to a large extent fundamental, has always been performed having in view a final practical application. It seems that this research, which began in the early seventies with studies of the linear analysis of multistorey buildings, has now entered its age of maturity, and some of the results which have been obtained might be of interest to the research community and advanced practical engineers."

4.3 Acceleration-Displacement (AD) Format – Two Key Papers

A breakthrough of pushover-based methods was possible after the acceleration-displacement (AD) format was proposed in (Mahaney et al. 1993). A spectrum plotted in the AD format was originally called "ADRS", which stands for Acceleration-Displacement Response Spectrum. Using the AD format, in which acceleration is on the vertical axis and displacement is on the horizontal axis, the capacity of an SDOF structure can be visually compared with the demands of earthquake ground motion on the structure. The capacity of the structure is represented by a force-displacement curve, obtained by non-linear static (pushover) analysis. The base shear forces and roof displacements are converted to the accelerations and displacements of an equivalent SDOF system, respectively. These values define the capacity diagram. Seismic demand spectrum is plotted in the AD format rather than in the standard acceleration-period format (see Section 2.2).

Presenting the demand spectrum in the AD format is a simple but brilliant idea, which allows the visualisation of the problem, a feature that is of the utmost importance for engineers. The intersection of the capacity curve and the demand spectrum provides an estimate of the inelastic acceleration (strength) and displacement demand. The graphical presentation makes possible a visual evaluation of how the structure will perform when subjected to earthquake ground motion. Moreover, it clearly shows the relations between the basic quantities controlling the seismic response, i.e., stiffness, strength, displacement and ductility. The AD format was the main invention that strongly accelerated practical applications of pushover-based methods.

The AD format was used in the Capacity Spectrum Method (CSM) implemented in ATC-40 (ATC 1996). At that time, the name "Capacity Spectrum Method" was linked with equivalent elastic overdamped spectra, which were employed for defining seismic demand. The use of equivalent elastic spectra with high damping is a controversial approach. Nevertheless, it became quite popular among some researchers and is still used in research and in practice. According to Krawinkler (1995),

> there are two fundamental flaws that render the quantitative use of the capacity spectrum method questionable. First, there is no physical principle that justifies the existence of a stable relationship between the hysteretic energy dissipation of the maximum excursion and equivalent viscous damping, particularly for highly inelastic systems. The second flaw is that the period associated with the intersection of the capacity curve with the highly damped spectrum may have little to do with the dynamic response of the inelastic system.

Due to these deficiencies, it was stated in Vision 2000 (SEAOC 1995) that "the theoretical foundations of the method are open to question". Chopra and Goel (2000) found several shortcomings of the ATC-40 CSM approach.

The lack of consensus on the definition of seismic demand was reflected in the two different approaches used in the US documents. Whereas the CSM with the equivalent elastic spectrum was the main method used in ATC-40, it was the so-called displacement coefficient method that was used in FEMA 273 (FEMA 1997). In this latter method, the displacement demand was essentially determined from the inelastic displacement spectra, which were obtained from the elastic displacement spectra by using a number of correction factors based on statistical analyses. Interestingly, FEMA 273, as well as FEMA 356 published a few years later (FEMA 2000), did not use the AD format.

In order to overcome the deficiencies of the original version of the CSM, Bertero (1995) recommended using "smoothed inelastic design response spectra" as demand spectra. However, to realise this recommendation, the conventional acceleration-period format was applied. Fortunately, the AD format does not need to be linked with the equivalent elastic overdamped spectra. Reinhorn (1997) demonstrated that, as an alternative to the use of elastic spectra with equivalent damping, inelastic demand spectra in the AD format could be applied within the CSM. Thus, the two advantages (i.e., the visual representation in the AD format and the superior physical basis of inelastic demand spectra) can be combined. The new procedure eliminated the controversial part of the original CSM procedure. Neither equivalent viscous damping nor the period associated with the intersection of the capacity curve with the highly damped spectrum was used. Reinhorn's paper was presented at the second Bled workshop in 1997 (Section 17.1), where there were many opportunities for in-depth discussions, also on the possibility of using inelastic demand spectra in CSM.

I liked the Reinhorn's proposal, but I did not find the time to formulate the N2 method in the AD format. However, after attending the 6th US National Conference on Earthquake Engineering in Seattle in 1998 and after a subsequent trip to Japan, I became aware of the rapidly increasing popularity of pushover-based methods. Additional motivations were the paper on the method written by Sigmund Freeman for the 11th ECEE in Paris, which was scheduled for a session that I was supposed to chair, and the rapid development of direct displacement-based methods using equivalent elastic spectra, developed by Nigel Priestley, Michele Calvi, and others. Therefore, I finally formulated the N2 method in the AD format by using Reinhorn's idea as an alternative to the CSM approaches mentioned above. Knowing that the publication procedures in international journals are usually slow, I first prepared the results of my work in the form of an internal report (Fajfar 1998), which was distributed among my colleagues around the world. Later, in November 1998, I submitted a revised and extended version of the text entitled "Capacity spectrum method based on inelastic demand spectra" to EESD. It was published the following year (Fajfar 1999).

Independently, Chopra and Goel (1999a, 1999b) used the same idea of replacing the equivalent elastic spectrum with an inelastic spectrum in the CSM.

In my EESD paper (Fajfar 1999), I clearly stated in the Introduction: "This paper contains no basic original developments. It just synthesises existing information and presents it in an easy to use format, which might be acceptable for practical design and for the development of future design guidelines."

In addition to the detailed explanation of the N2 method in the new format and its application to an illustrative example, some discussion of the possibilities of the applicability of the method to different types of approaches, which can easily be visualised in the AD format, was provided. There are four important parameters that define the seismic performance: strength, displacement, ductility, and elastic stiffness. These characteristics are represented by the acceleration A, the displacement D, the ductility factor μ, and the period T. The four quantities are related by Eq. 5.3 in Section 5.2. An additional relationship defines inelastic acceleration demand as a function of μ and T (R_{μ}–μ–T relation, see Sections 5.3 and 5.4). Seismic demand in terms of the elastic acceleration spectrum A_e is defined in the input data. Consequently, there are two equations (relations) for four unknown quantities. It follows that two quantities need to be chosen, and the other two computed.

Table 4.1. Assumed and determined structural characteristics in different approaches.

	Force-based design	**Displacement-based design**	**Performance evaluation**
Assumed	Period (Stiffness) Ductility	Displacement Ductility	Stiffness Strength
Determined	Strength Displacement	Period (Stiffness) Strength	Displacement Ductility

In different approaches, different quantities are chosen at the beginning (Table 4.1). Let us assume that the approximate mass is known. In the case of seismic performance evaluation, the stiffness (period) and strength of the structure have to be known; the displacement and ductility demands are calculated. In direct displacement-based design, the starting points are typically the target displacement and/or ductility demands. The quantities to be determined are stiffness and strength. The conventional force-based design typically starts from the stiffness (which defines the period) and the approximate global ductility capacity. The seismic forces (defining the strength) are then determined and, finally, the displacement demand is calculated. Capacity in terms of spectral acceleration can be determined from the capacity in terms of displacements. All these approaches can be easily visualised with the help of Fig. 2.3. Note that, in all cases, the strength is the actual strength and not the design base shear according to seismic codes, which is less than the actual strength in all practical cases. Note also that stiffness and strength are usually related quantities.

The review procedure for my paper was smooth. The revised version was accepted four months after the submission of the original version. I was pleased to read that both reviewers suggested accepting the manuscript with some minor modifications. The first review started with: "The author should be congratulated for synthesizing recent information and presenting it in single form with substantiated examples for support." and ended with "In conclusion, this is an important paper which presents a true inelastic approach alternative to the one using equivalent period and damping (that characterizes hysteretic behavior). Important and timely." The second reviewer wrote "Overall, thought the paper to be very interesting and well presented." Among reviewers' comments, there were some suggestions for additional clarifications and references, one of which was a request for the clarification of similarities and differences in respect to the Reinhorn (1997) paper. However, it was stated that "the paper . . . presents the capacity spectrum method in an easier format for the reader/user, perhaps better than in Reinhorn (1997)". The relation to the Reinhorn (1997) paper was an issue also in the case of my next paper published in Earthquake Spectra, and I will comment on it later.

I have been a member of the Earthquake Engineering Research Institute (EERI) since 1981. Considering that the EERI journal, Earthquake Spectra, probably has the largest audience of practising engineers among earthquake engineering journals and that the USA was the country where pushover-based analysis began and was first implemented in regulations, I decided to submit a paper on the N2 method in the AD format to Earthquake Spectra. There was a risk that, in the new paper, there would be too much repetition of the material published in the 1999 EESD paper. In fact, one of the reviewers asked what the difference was between the two papers. My response was:

> The basic objective of the paper published in EESD (Fajfar, 1999) was to demonstrate that it is feasible to replace equivalent elastic demand spectra in capacity spectrum method with inelastic demand spectra. In the paper under review, the basic objective is to present the simplest version of the proposed method together with the basic derivations to a broader, more practically oriented audience. Necessarily, there are many similarities between the two papers, but also many differences. In the paper under review, the simplest version of the inelastic demand spectra is used, which does not require any iteration. Each step of the procedure is explained with more details and a summary of the method is presented in a transparent form. The derivations of basic equations are presented. Many discussions, which are not directly connected with the proposed procedure, are omitted.

> In the revised version of the paper, the limitations of the proposed method are more clearly summarized and discussed. A comparison with nonlinear static procedures in FEMA-273 and ATC-40 is presented. The test example has been changed.

As mentioned in my response, the N2 method has been further simplified by using the simplest variant of the inelastic spectrum, which is based on the equal displacement rule in the moderate- and long-period range (see Section 5.4). The transition period between the short- and moderate-period range was fixed to the characteristic period of the ground motion. These assumptions have only a minor influence on the results of the N2 method but substantially simplify the computational procedure. As a further simplification, the low cycle fatigue effects have been neglected.

As expected, in the case of this paper, the review procedure was quite tough. Three reviewers were engaged, and they provided constructive and fair comments, which certainly helped to improve the quality of the paper. The number of comments made by one of the reviewers was 69 (sixty-nine), and my response to all three reviewers comprised nine single-spaced pages. I responded to all the comments of all the reviewers and explained the changes, which I made, or the reasons that I did not take into account the comment. In order to demonstrate the difficulties related to the introduction of a new method, I will present some of the reviewers' comments and my responses. Since Earthquake Spectra is a US journal and the reviewers were most probably from there, in my response, I relied mostly on US documents.

Two of the reviewers expressed some scepticism about the method and its background. My general response was:

> The author does not share the scepticism of two reviewers about the proposed method and about the basic assumptions. An attempt has been made to demonstrate (in this response and in the revised version of the paper) that the N2 method is not worse than other available methods, intended for PRACTICAL application. The author does hope that it is simpler, more transparent and of similar accuracy, i.e., an accuracy appropriate and reasonable for practice. Nevertheless, a number of relevant comments of the reviewers helped to make a better presentation of the method. Hopefully, the considerable amount of time, which has been spent by the reviewers (their efforts are highly appreciated) and by the author for the response and the preparation of the revised version, has resulted in a better paper.
>
> From the reviews, it is evident that one of the reviewers is concerned about the relations between SDOF and MDOF system, whereas the other one is mainly concerned with inelastic spectra. Maybe it was not evident from the paper that the assumptions, used in the N2 method, are identical

or very similar to the assumptions that have been accepted by a large part of the earthquake engineering community. However, it is also true that they have not been accepted by the other part of the same community. I believe that it is impossible to have an in-depth discussion of controversial issues, on which probably never a consensus will be reached, within the review procedure of the submitted paper. I will rather defend my case in an indirect way.

I assume that the Nonlinear Static Procedure (NSP) in FEMA-273 is a legitimate procedure. If this assumption is correct, it is reasonable to assume that the N2 method, which yields identical or very similar results, should also be an acceptable procedure. Please note that the N2 method had been developed well before the FEMA-273 procedure, as explained later in the text. What is new in the proposed version of the N2 method and what represents an essential difference to the FEMA-273 approach, is the formulation of the method in the acceleration-displacement format which enables a graphical representation of the procedure and results. Again, the format is not new, it is taken from the capacity spectrum method, which is used in ATC-40. The new format does not change any result. However, it is extremely important for practical applications, because it helps designers to better understand the procedure and the relations between basic quantities.

The proposed version of the N2 method is also similar to the capacity spectrum method in ATC-40. However, there is a basic and very important difference. Seismic demand in the N2 method is determined by inelastic spectra and not by equivalent elastic spectra. Inelastic spectra, as determined in the N2 method (and in FEMA-273), may be controversial. However, equivalent elastic spectra are even more controversial. It has been shown (Chopra and Goel, Capacity-Demand-Diagram Methods Based on Inelastic Design Spectrum, Earthquake Spectra 15, No. 4, 1999) that the ATC-40 procedures, based on equivalent elastic spectra, may produce inadequate results.

A new chapter entitled "Comparison with the Nonlinear static procedure in FEMA-273 and the Capacity spectrum method in ATC-40" was added in the revised version, in which the similarities and differences are briefly discussed. It is shown, inter alia, that in the proposed method one of the approximations, used by ATC-40 and FEMA-273, is eliminated.

The third reviewer was my friend Andrei Reinhorn, who signed his review. His review was very positive. Based on one of his valuable comments, a new chapter on the limitations of the proposed method was prepared in the revised version. In that chapter, the basic limitations, which were scattered throughout the text in the original version, were summarised. In another comment, Andrei made a remark that made me feel uncomfortable, although I had a clean conscience. He mentioned that the paper repeated many of the features indicated by Reinhorn (1997) and requested that differences between the papers, which produced some information in slightly different ways, should be addressed. In my response, I explained the historical development of the N2 method and stated:

> I hope that this short history of the N2 method demonstrates that the computational procedure has been derived by the author and his colleagues, and published well (about one decade) before other authors independently developed their (similar) procedures. Of course, some ideas of other authors have been incorporated. A "mature" method …. is usually a result of the convergence of the work and ideas of many different authors.

and,

> Nothing but the excellent idea of the inelastic spectra in acceleration – displacement format was taken from Reinhorn's paper and this was and is acknowledged. The development of the N2 method was the major undertaking of my research group in the last 15 years. In 1996, I edited the book entitled "Towards a new seismic design methodology for buildings" (University of Ljubljana), containing 21 selected papers of my research group related to the N2 method. So, everything was prepared when Reinhorn's idea came. Nothing has been changed in the computational part of the N2 method, only the graphical presentation was added.

In response to Reinhorn's suggestion ("the author needs to comment on the similarities and the differences between the papers"), the following text was added in the Introduction:

> The main difference of the proposed procedure compared to the procedure developed by Reinhorn (1997) is its simplicity. Reinhorn's approach is very general and less restrictive. In the proposed N2 method several simplifications have been implemented. They impose some additional limitations. On the other hand, they allow the formulation of the method in a transparent and easy-to-use format, which is convenient for practical design purposes and for the development of the future design guidelines. Although the computational procedures have been developed independently, the proposed N2 method can, in principle, be regarded as a special case of the general approach presented by Reinhorn (1997).

The reviewers were satisfied with my response, so the revised manuscript was accepted for publication in May 2000, about ten months after submission. This paper became my most cited paper, with about 1400 citations in Google Scholar (January 2020). Interestingly, the citations do not decrease with time since the year of publication. In recent years, almost two decades since publication, the citations are more or less constant at a level of over 100 per year. Unfortunately, in the year 2000, the journal Earthquake Spectra was not included in the Web of Science database, which was and is the most respected database for citations.

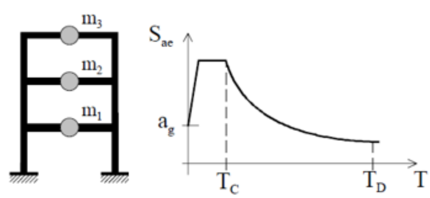
Several reasons contributed to the success of this paper. Firstly and most importantly, the N2 method was consolidated and evaluated by many users more than a decade after its original development. In the paper, the method was clearly described with all the derivations needed for the understanding of the background of the method. In an appendix, reproduced in Fig. 4.1, a transparent summary of the method was provided. Limitations of the method were presented. Very importantly, a step-by-step use of the method was illustrated with a test example. A comparison with well-known analysis procedures was given. All these were provided in 20 pages (in the Earthquake Spectra format), which is still slightly beyond the maximum acceptable length of a reader-friendly paper.

Summary of the N2 method (basic variant)
(Appendix 1 in Fajfar 2000)

I. DATA

a) Structure

b) Elastic acceleration spectrum S_{ae}



II. DEMAND SPECTRA IN AD FORMAT

a) Determine elastic spectrum in AD format

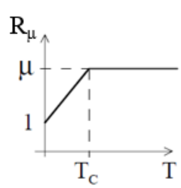
$$S_{de} = \frac{T^2}{4\pi^2} S_{ae}$$

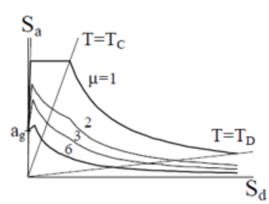
b) Determine inelastic spectra
for constant ductilities

$$S_a = \frac{S_{ae}}{R_\mu}, \quad S_d = \frac{\mu}{R_\mu} S_{de}$$

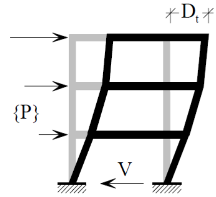
$$R_\mu = (\mu - 1)\frac{T}{T_C} + 1 \qquad T < T_C$$

$$R_\mu = \mu \qquad T \geq T_C$$





III. PUSHOVER ANALYSIS



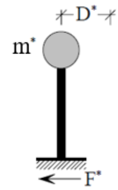
a) Assume displacement shape $\{\Phi\}$

b) Determine vertical distribution of lateral forces

$$\{P\} = [M]\ \{\Phi\}, \quad P_i = m_i\ \Phi_i$$

c) Determine base shear (V) – top displacement (D_t) relationship

IV. EQUIVALENT SDOF MODEL



a) Determine mass m^*

$$m^* = \sum m_i\ \Phi_i$$

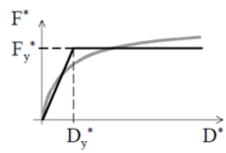
Note: $\Phi_n = 1.0$, n denotes roof level

b) Transform MDOF quantities (Q) to SDOF quantities (Q^*)

$$Q^* = \frac{Q}{\Gamma}, \quad \Gamma = \frac{m^*}{\sum m_i\ \Phi_i^2}$$

c) Determine an approximate elasto-plastic force – displacement relationship

d) Determine strength F_y^*, yield displacement D_y^*, and period T^*

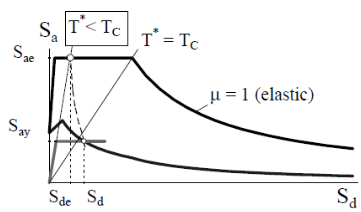


$$T^* = 2\pi\sqrt{\frac{m^*\ D_y^*}{F_y^*}}$$

e) Determine capacity diagram (acceleration versus displacement)

$$S_a = \frac{F^*}{m^*}$$

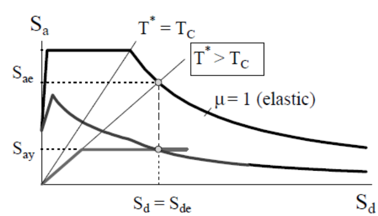
V. SEISMIC DEMAND FOR SDOF MODEL



a) Determine reduction factor R_μ

$$R_\mu = \frac{S_{ae}}{S_{ay}}$$

b) Determine displacement demand $S_d = D^*$



$$S_d = \frac{S_{de}}{R_\mu}\left(1 + (R_\mu - 1)\frac{T_C}{T^*}\right) \qquad T^* < T_C$$

$$S_d = S_{de} \qquad T^* \geq T_C$$

VI. GLOBAL SEISMIC DEMAND FOR MDOF MODEL

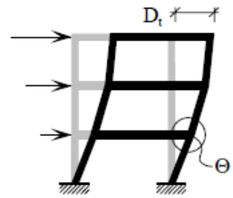
a) Transform SDOF displacement demand to the top displacement of the MDOF model

$D_t = \Gamma\ S_d$

VII. LOCAL SEISMIC DEMANDS

a) Perform pushover analysis of MDOF model up to the top displacement D_t (or to an amplified value of D_t)

b) Determine local quantities (e.g. story drifts, rotations Θ), corresponding to D_t



VIII. PERFORMANCE EVALUATION

a) Compare local and global seismic demands with the capacities for the relevant performance level

Figure 4.1. Summary of the basic N2 method (from Fajfar 2000). Note that, in part, different notation is used than in this book.

4.4 N2 method in Eurocode 8

Eurocodes are ten European standards covering various subjects related to construction. All ten Eurocode standards are consistent with each other. Seismic provisions are included in the standard "Design of structures for earthquake resistance", called Eurocode 8 or EC8 (CEN 2004). It applies to the design of new buildings and engineering works and the assessment and retrofit of existing ones, including geotechnical aspects. The first version of Eurocodes was developed in the form of pre-standards in the 1990-1998 period. In the 1998-2006 period, the pre-standards were converted to standards, which have been gradually adopted in the EU-EFTA countries since 2008.

Considering the popularity of newly developed pushover-based analyses, I raised this issue in the form of Slovenian National Committee comments on the draft standard, when it was already in a quite advanced stage. The response was positive and, in mid-April 2001, I was asked by Michael Fardis, who led the development of the standard, to draft the new clauses related to pushover analysis. With the help of him and Matjaž Dolšek, the draft clauses, which basically represented the codified version of the N2 method (the variant presented in my 2000 Earthquake Spectra paper), were ready in early May 2001. The final text was later implemented in the main body of Part 1 of EC8 (EC8-1), whereas the determination of the target displacement was included in the informative Annex B to EC8-1, entitled "Determination of the target displacement for nonlinear static (pushover) analysis".

Of course, for a code application, some steps in the analysis, which are based on engineering judgement, have to be prescribed.

The results of pushover analysis depend on the distribution of lateral loads. Whereas in the N2 method any reasonable distribution can be used, “at least two vertical distributions of the lateral loads should be applied” according to EC8:
− a “uniform” pattern, in which lateral forces are proportional to mass regardless of elevation (uniform response acceleration);
− a “modal” pattern, consistent with the lateral force distribution determined in an elastic analysis.

The results of the N2 method also depend on the idealisation of the pushover curve. According to EC8-1, an elastic-perfectly plastic idealisation is performed at the SDOF level. The initial (elastic) stiffness of the idealised system is determined by using the equal energy principle (the areas under the actual and idealised curves should be equal) assuming that the target displacement is equal to the displacement at the formation of a plastic mechanism. If the actual displacement demand (target displacement) is much lower than that corresponding to the plastic mechanism, this approach may grossly underestimate the initial stiffness and, consequently, grossly overestimate the displacement demand. In such a case, it is reasonable to apply an iterative procedure (optional in EC8), using the current target displacement, which leads to a higher equivalent stiffness and to a smaller displacement demand.

In 2001, the N2 method was still limited to planar structural models. In EC8, it was simply stated that the analysis may be performed using two planar models, one for each main horizontal directions if the building is regular in plane. In the case of buildings not conforming to the regularity criteria, a spatial model shall be used, in which two independent analyses with lateral loads applied only in one direction may be performed. Note that such an approach was (is) still applicable after the N2 method was extended to spatial (3D) structural models in 2002.

Due to its basic assumption of vibration in a single mode, a simple pushover-based analysis, like the basic N2 method, cannot properly take into account the higher mode effects and the torsional vibration. When EC8-1 was finalised, we worked on this problem. However, the extended version of the N2 method for plan-asymmetric buildings had not been fully developed yet. Nevertheless, based on the preliminary results, the clause “Procedure for the estimation of torsional effects” was added, in which it was stated that pushover analysis may significantly underestimate deformations at the stiff/strong side of a torsionally flexible structure. It was also stated that “for such structures, displacements at the stiff/strong side shall be increased, compared to those in the corresponding torsionally balanced structure”, and that “this requirement is deemed to be satisfied if the amplification factor to be applied to the displacements of the stiff/strong side is based on the results of an elastic modal analysis of the spatial model.”

In the case of higher mode effects along the elevation, a statement was provided in EC8 Part 3 (CEN 2005), which applies to existing buildings: For buildings with long fundamental period and for buildings irregular in elevation, "the contributions to the response from modes of vibration higher than the fundamental one in each principal direction should be taken into account". However, no specific means were provided how to do this. It was just stated that

> this requirement may be satisfied . . . through special versions of the non-linear static analysis procedure that can capture the effects of higher modes on global measures of the response (such as inter-storey drifts) to be translated then to estimates of local deformation demands (such as member hinge rotations). The National Annex may contain reference to complementary, non-contradictory information for such procedures.

The implementation of the N2 method in EC8 was certainly beneficial for the visibility and popularity of the method, and it stimulated further developments. Fortunately, the basic variant of the method was ready and published just in time, when the development of EC8 was in its final stage. In contrast, there was a very long time lag between the first publication of the N2 method in 1987 and its implementation in a regulatory document (officially adopted in 2004).

4.5 Capacity in Terms of Ground Motion

In some cases, e.g., when determining the IN2 curve (Chapter 13), or in a probabilistic approach (Chapter 14), it is necessary to determine the capacity of the structure in terms of ground motion, e.g., spectral acceleration, peak ground acceleration, and/or the ground motion spectrum. The procedure can be visualised in Fig. 4.2, where the equal displacement rule is assumed.

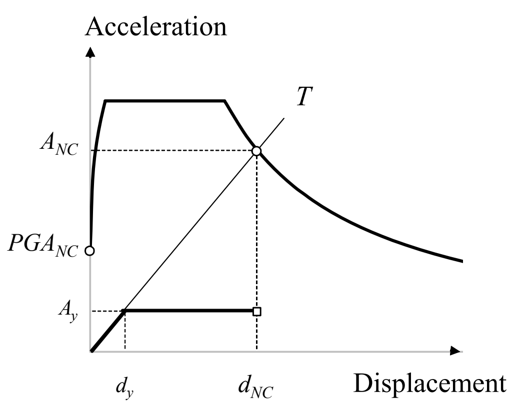


Figure 4.2. NC capacity in terms of ground motion.

The capacity diagram shown in Figure 4.2 represents the idealised pushover curve of an equivalent SDOF model in AD format. Failure occurs at the ultimate displacement, defined as the displacement at the NC limit state d_{NC}. The capacity of the structure in terms of elastic spectral acceleration A_{NC} is defined by the crossing point of the vertical line, representing the displacement at the NC limit state, and the diagonal line, representing the period of the structure. The crossing point is a point on the acceleration spectrum, corresponding to the capacity of the structure in terms of ground motion. If the shape of the spectrum is known or assumed, all spectral values, including the peak ground acceleration PGA_{NC}, are defined.

Analytically, the capacity A_{NC} can be determined as the product of the yield acceleration A_y and the reduction factor R_{μ} (Chapter 3) corresponding to the appropriate R_{μ}-μ-T relation: $A_{NC} = A_y R_{\mu}$. If equal displacement rule applies, $R_{\mu} = \mu_{NC}$, where $\mu_{NC} = d_{NC}/d_y$.

4.6 Extensions

The basic version of the N2 method has been developed for planar structures vibrating predominantly in a single mode. With time, we developed several extensions that allow the use of the N2 method also for an approximate analysis of a broader range of building structures.

The theoretical background for the extension of the applicability of the N2 method to spatial structural models was published in my keynote paper at the 12th ECEE (Fajfar 2002). The extension proved to be straightforward. The planar model was replaced by a spatial model, whereas the lateral loading was applied only in one horizontal direction. Separate 3D pushover analyses were performed in each of the two horizontal directions. Relevant results (displacements, storey drifts, joint rotations, and forces in brittle elements which should remain in the elastic region), obtained by two independent analyses, are approximately combined by the SRSS rule. The derivation of the procedure proved that all formulae developed for the planar system remain valid also for the spatial system. Of course, the problem of torsion has not been solved yet with this extension, which did not consider any dynamic torsional effect that may greatly affect the structural response. The Extended N2 method, also taking into account the dynamic torsional effects, was developed in 2005. In 2011, the N2 method was extended to the structures with a non-negligible influence of higher modes along the height of the building. This extension was combined with the previously developed extension for torsion in a single procedure (see Chapter 12). Frames with masonry infill require special treatment. Our work on this subject is summarised in Chapter 9. Capacity issues are discussed in Chapter 11, whereas the Incremental N2 is described in Chapter 13. Chapters 10, 14, and 15 deal with applications of the N2 method for analysis of bridges, in seismic risk analysis, and for determination of floor spectra, respectively.

5 INELASTIC SPECTRA

5.1 Introduction

Response spectra are the primary computational tool for dynamic analysis in practice. They usually represent the maximum values of relevant response quantities as a function of the structural period. Because, in design, we are mostly interested only in maximum response values, the use of appropriate response spectra reduces the dynamic analysis to free vibration analysis, i.e., the determination of natural periods and mode shapes. Elastic design spectra are usually provided in seismic codes or, in the case of important structures, are determined by a seismic hazard analysis of the site. Inelastic spectra depend not only on the characteristics of the expected ground motion at the site but also on the nonlinear characteristics of the structural system, which significantly complicates the problem.

When using response spectra for the analysis of MDOF systems, a modal analysis is needed, which is based on the superposition of results for individual modes. In the case of elastic structures, the superposition is legitimate. The superposition of maximum values, which generally do not occur at the same time, is approximate. Nevertheless, different superposition rules (the most well-known are the SRSS (Square Root of Sum of Squares) and the CQC (Complete Quadratic Combination) rules have been widely accepted and used. In the case of inelastic structures, however, a superposition of the results for different modes is theoretically not possible. Thus, in theory, the use of inelastic spectra, representing the maximum response of an inelastic structure, is possible only for an SDOF system, whereas it is not applicable to an MDOF system. This is the main reason that simplified methods for the analysis of inelastic structures were initially limited to structures that vibrate predominately in a single mode (typically the fundamental mode) and can be modelled as an equivalent SDOF system. Some extensions of pushover-based analysis, which take into account also the effects of higher modes, use, as an approximation, the mode superposition also in the inelastic range. Similarly, the SRRS rule can be applied as an approximation for combining the results of pushover analysis in two orthogonal directions.

The N2 method uses inelastic spectra. For this reason, in our research group, much work has been done on this topic. Our first papers on inelastic spectra were presented at the 7th ECEE (Fischinger and Fajfar 1982) and at the 8th WCEE (Fajfar and Fischinger 1984). Over approximately the next ten years, extensive parametric studies of SDOF systems were carried out. In these studies, input ground motion, as well as the initial stiffness (expressed by the natural period or frequency), strength, ductility, hysteretic behaviour, and damping of SDOF structural systems, were varied. Seismic demand, expressed in terms of the maximum relative displacement, normalised strength, force reduction factor R_μ, and various energy parameters and ductility factors, was studied. Tomaž Vidic, who was among my best doctoral students, was heavily involved in this research. He had an ability to analyse, synthesise and think critically. He finished his doctorate in 1993 and worked as a postdoc researcher several years in our research group. In this period he also spent some time in Japan and in California. In 1999, Tomaž started a new career in state administration and in 2016 became the chairman of the board of the state-owned motorway company.

The results, obtained at different stages of the research, were published in several publications, e.g., in (Fajfar et al. 1989, 1990). In the end, a procedure for the determination of consistent inelastic design spectra (for strength, displacement, hysteretic and input energy) for systems with a prescribed ductility factor was developed. All the spectra are interrelated and based on the same assumptions. A comprehensive overview of the most relevant results was provided in two highly cited companion papers published in EESD (Vidic et al. 1994, Fajfar and Vidic 1994). A summary of our work on inelastic spectra was presented at several conferences, including the 10th ECEE in Vienna (Fajfar 1995a, 1995b) and the 11th WCEE in Acapulco (Fajfar 1996).

A few years later, Mark Aschheim and one of his PhD students at the University of Illinois at Urbana-Champaign did some work on R factors. They found the R_μ-μ-T relation, developed in our research group, despite its simplicity more accurate than other well-known and commonly used models. They performed a parametric study by using different values of parameters controlling the R_μ-μ-T relation and invited me to contribute to a paper published in Earthquake Spectra (Cuesta et al. 2003).

5.2 Basic Relations

For an elastic SDOF system, Eq. 5.1 applies:

$$D_e = \frac{T^2}{4\pi^2} A_e \tag{5.1}$$

where A_e and D_e are the values in the elastic acceleration and displacement spectrum, respectively, at the fundamental period T for a fixed viscous damping ratio. Strictly, A_e is pseudo-acceleration which is, however, practically equal to acceleration if damping is small. In this book, no distinction will be made between acceleration and pseudo-acceleration. In the AD format, spectral accelerations are plotted against spectral displacements, with the periods T represented by radial lines. Note that Eq. 5.1 is, in practice, not applicable in a very long period range where the acceleration tends to zero, whereas the spectral displacement tends to the maximum ground displacement.

For an inelastic SDOF system with a bilinear force-deformation relationship, the values in the acceleration spectrum (A_{in}) and the displacement spectrum (D_{in}) can be determined by using the ductility dependent reduction factor R_μ (see Chapter 3):

$$A_{in} = \frac{f_y}{m} = \frac{A_e}{R_\mu} \tag{5.2}$$

$$D_{in} = \frac{\mu}{R_\mu} D_e = \frac{\mu}{R_\mu} \frac{T^2}{4\pi^2} A_e = \mu \frac{T^2}{4\pi^2} A_{in} \tag{5.3}$$

where μ is the ductility factor defined as the ratio between the maximum displacement and the yield displacement, f_y is the yield force, and m is the mass of the system.

Eqs. 5.2 and 5.3 indicate that the inelastic spectrum can be determined from the elastic one, considering the reduction factor due to ductility, R_μ, as a function of the ductility μ and period T, i.e., the R_μ-μ-T relation. This relation, which used to be called the R_μ spectrum, was the subject of our research in the early 1990s. Note that the R_μ spectra are just a means to obtain inelastic spectra from elastic spectra. They have been derived using specific elastic spectra and, in principle, they can be applied only in connection with the elastic spectra used in derivation, or with spectra similar to them. In any case, a smooth R_μ spectrum can be applied only to a smooth elastic spectrum.

5.3 Early Proposal

In order to determine simple formulae for R_μ factors, which would be widely applicable, extensive parametric studies have been performed. The results were summarised by Vidic et al. (1994).

The influence of input motion has been studied using five different groups of records representing the ground motions of basically differing types. Two hysteretic models, simulating predominantly flexural behaviour were used: the bilinear model and the stiffness degrading Q-model. In both cases, 10 per cent hardening of the slope after yielding was assumed. Two mathematical models of damping that place the bounds on the dynamic response were investigated. The first, so-called “mass-proportional” damping assumes a time-independent damping coefficient based on elastic properties. In this case, the effective (instantaneous) damping ratio (the actual damping coefficient divided by the damping coefficient at critical damping) increases with decreasing values of the instantaneous (tangent) stiffness. The second model was “instantaneous stiffness-proportional” damping, in which a time-dependent damping coefficient based on tangent stiffness is assumed. In this case, the effective damping ratio decreases with increasing values of the tangent stiffness. In the majority of cases, five per cent damping was assumed. Some structural systems with two per cent damping were also analysed.

Based on the parametric studies, several conclusions on the characteristics of R_μ factor were drawn. The reduction factor R_μ is, in the medium- and long-period region, only slightly dependent on the period T, and is roughly equal to the prescribed ductility μ. In the short-period region, however, the R_μ factor depends strongly on both T and μ. The moderate influence of hysteretic behaviour and damping can be observed in the whole period region. Peaks in the R_μ spectrum correspond to peaks in the elastic pseudo-acceleration spectrum (for this reason, smooth R_μ spectra are applicable in combination with smooth elastic spectra). The transition period from the period-dependent part to the more or less period-independent part of the R_μ spectrum is near to the period T_C, which approximately represents the predominant (also termed characteristic) period of the ground motion. A more precise value of the transition period depends on the prescribed ductility factor. The influence of other characteristics of the ground motion on the R_μ factor has not been considered in our study.

Considering the above conclusions, we decided to consider the dependence of the R_μ factor on the natural period of the system, the prescribed ductility factor, the hysteretic behaviour, damping and the predominant period of the ground motion. Having in mind the large uncertainties inherent in parameters involved in earthquake-resistant design, and the relatively small samples used in our study, no attempt has been made to perform rigorous statistics in the derivation of the simplified expressions. The main idea was to develop formulae that were, on the one hand, as simple as possible and, on the other, produced appropriate results for all the derived basic parameters, including energies which were discussed in the companion paper (Fajfar and Vidic 1994). Consequently, trial-and-error procedures, combined with some simple statistical methods, have been used.

A bilinear curve was proposed for the R_μ spectrum. The spectrum is divided into two period regions. In the first region, which roughly corresponds to the short period region, R_μ increases linearly with increasing period from $R_\mu = 1$ to a value that is equal to the value of the ductility factor ($R_\mu = \mu$) or near to it. In the remaining part of the spectrum (roughly in the medium- and long-period range) the R_μ factor maintains a constant value.

$$R_\mu = c_1(\mu - 1)^{c_r}\frac{T}{T_0} + 1 \qquad T < T_0 \tag{5.4}$$

$$R_\mu = c_1(\mu - 1)^{c_r} + 1 \qquad T \geq T_0 \tag{5.5}$$

$$T_0 = c_2\mu^{c_t}T_C \tag{5.6}$$

The predominant period of the ground motion, T_C, is defined as the period at the transition between the constant acceleration and constant velocity regions of a 5% damped smoothed elastic design spectrum. T_C corresponds to the period T_s used in FEMA 356 (2000), and T_C in EC8 (CEN 2004):

$$T_C = 2\pi \frac{V_{e,max}}{A_{e,max}} \tag{5.7}$$

where $A_{e,max}$ and $V_{e,max}$ are the maximum values in the smoothed elastic (pseudo)acceleration and (pseudo)velocity ground motion spectra, respectively, for linear elastic systems with 5% damping.

The constants c_1, c_2, c_R and c_T depend on the hysteretic behaviour and damping. Their values were proposed for combinations of two hysteretic models and two damping models (Table 5.1). Five per cent damping applies in all cases. Some examples of R_μ spectra are shown in Fig. 5.1.

Table 5.1. Values of the constants in Eqs. 5.4 – 5.6 for 5% damping.

Model					
Hysteresis	Damping	c_1	c_R	c_2	c_T
Q	Mass	1.0	1.0	0.65	0.30
Q	Inst. stiff.	0.75	1.0	0.65	0.30
Bilinear	Mass	1.35	0.95	0.75	0.20
Bilinear	Inst. stiff.	1.10	0.95	0.75	0.20

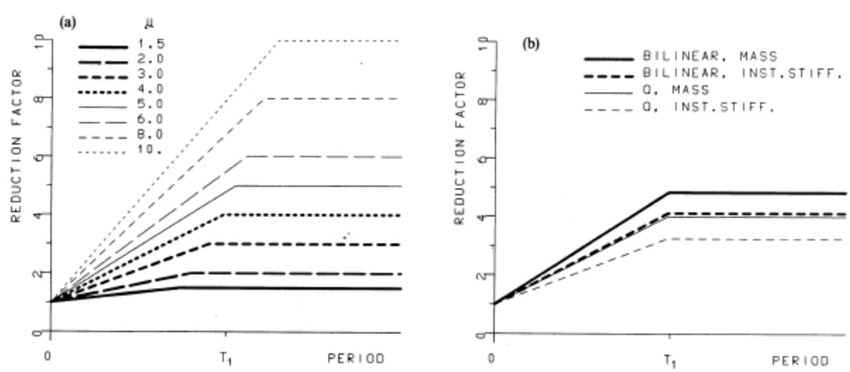


Figure 5.1. Proposed spectra for R_μ factor (for 5% damping): (a) the influence of ductility (Q-model, mass-proportional damping); (b) the influence of hysteretic model and damping model (μ=4) (from Vidic et al. 1994).

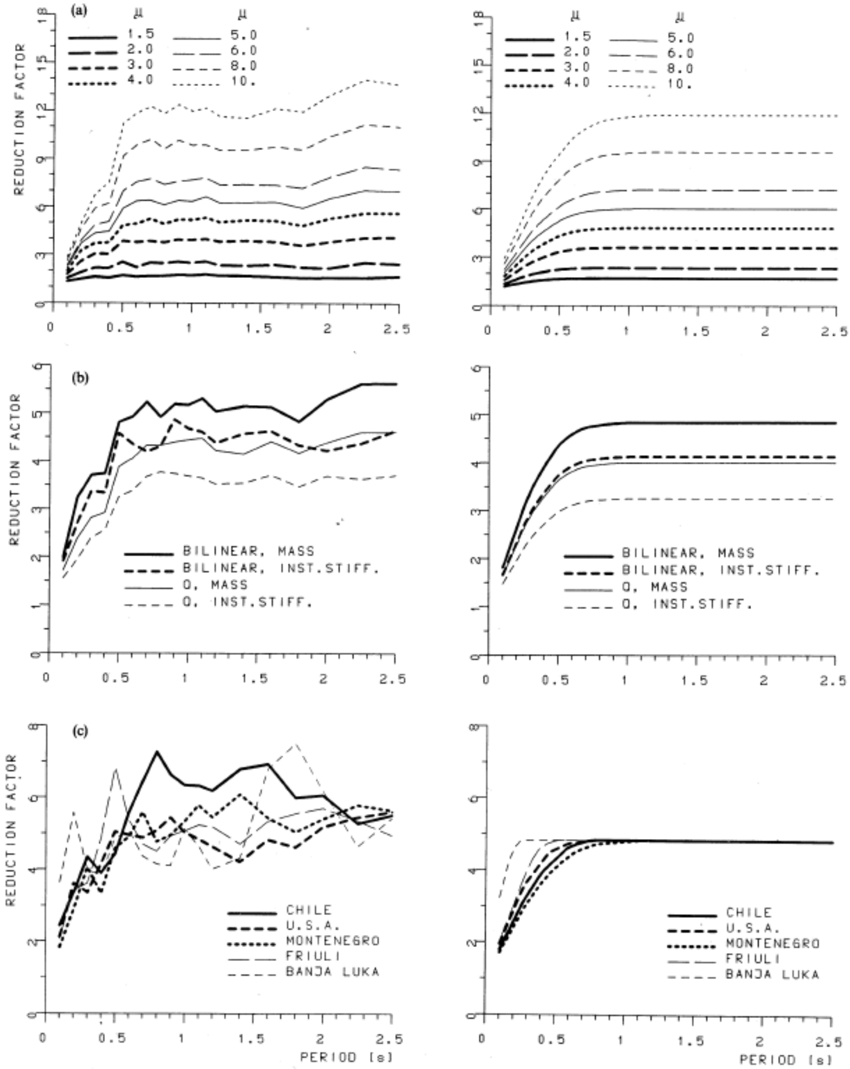


Figure 5.2. Comparison of "exact" and approximate mean inelastic spectra for reduction factor R_μ (5% damping): (a) the influence of ductility (bilinear model, mass-proportional damping); (b) the influence of hysteretic model and damping model (μ=4); (c) the influence of ground motion (μ =4, bilinear model, mass-proportional damping) (from Vidic et al. 1994).

A comparison between the proposed R_μ factors and mean values obtained by nonlinear response history analysis is shown in Fig. 5.2.

As far as damping is concerned, the R_μ factor depends not only on the mathematical modelling of the damping but also on the level of the assumed damping coefficient. The quantitative influence of this parameter has not been included in the formulae. Some data on the influence of the damping coefficient can be obtained from Figure 5.3, where a comparison of R_μ factors for 2 and 5 per cent damping is presented. It can be seen that the R_μ factors increase with decreasing damping. In the case of the Q-model, the R_μ factor corresponding to 2 per cent damping is about 10 per cent higher for the mass-proportional model and about 20 per cent higher for instantaneous-stiffness-proportional damping. As a conservative approximation, the R_μ factor for 5 per cent damping can be used instead of an R_μ factor corresponding to a lower damping percentage.

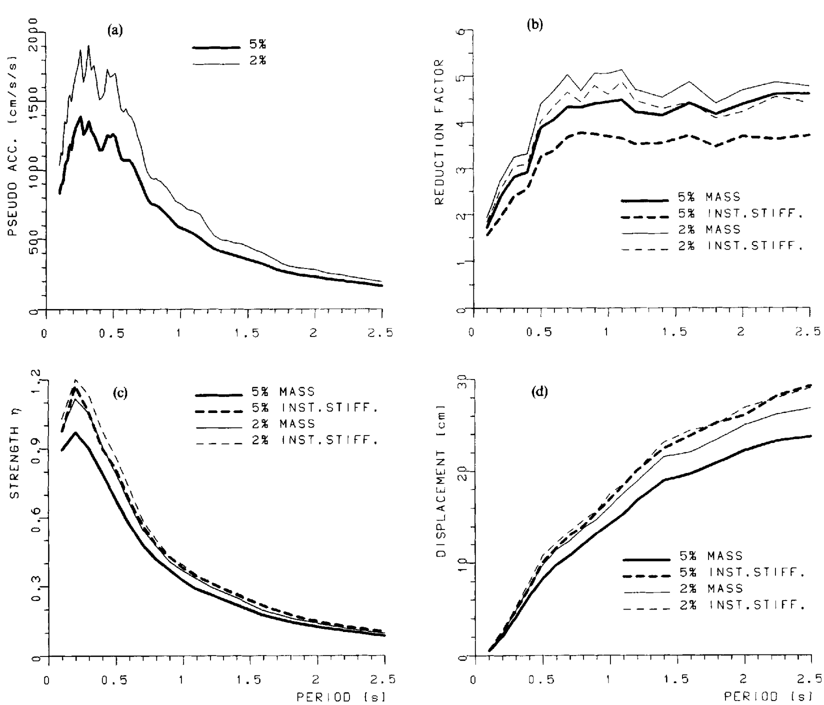


Figure 5.3. Influence of damping on mean elastic and inelastic spectra (μ =4, Q model): (a) elastic spectra for the group of standard records (normalised to peak ground velocity 50cm/s); (b) spectra for reduction factor R_μ; (c) spectra for strength; (d) spectra for displacement D (from Vidic et al. 1994).

The influence of damping on the strength and displacement spectra (Figures 5.3(c) and (d)) is a combination of damping effects on the elastic spectrum (Figure 5.3(a)) and on the R_μ factor (Figure 5.3(b)). In the case of the Q-model and mass-proportional damping, the approximate ratio between the values corresponding to 2 and 5 per cent damping is 1.15, for both strength and displacement. A lower ratio (1.05) was obtained in the case of instantaneous-stiffness-proportional damping.

In (Vidic et al. 1994), the non-dimensional strength η was used, which was defined as:

$$\eta = \frac{f_y}{m\ PGA} = \frac{A_e}{R_\mu\ PGA} \tag{5.8}$$

where *PGA* is peak ground acceleration.

The average coefficient of variation for the R_μ factors, observed in parametric studies, was about 0.3, which is a value similar to typical values observed in earthquake-resistant design. For strength and displacement spectra, the scatter of the inelastic spectra was similar to the scatter corresponding to elastic spectra.

5.4 Simplified Version

The results, presented for example in Figure 5.2(b), demonstrate that the R_μ factors determined for the stiffness degrading hysteretic Q model with mass proportional damping and for the bilinear hysteretic model with the instantaneous stiffness proportional damping are very close to each other and roughly correspond to the average R_μ factors for all investigated cases. Moreover, in the medium- and long-period range, the results demonstrate the validity of the equal displacement rule, i.e., the values of the R_μ factors are close to the values of the corresponding ductility. Based on these observations and considering the significant inherent uncertainties in seismic design and assessment, as well as the crudeness of the simplified nonlinear analysis, I neglected (Fajfar 1999) the dependence of the R_μ factor on the hysteretic behaviour (tacitly assuming that the formulas will not be used for extreme cases of systems with very low energy dissipation capability), and on the modelling of damping (a problem that has not yet been satisfactorily solved). The Q model with mass-proportional damping was chosen as representative for a large variety of typical RC and steel building structures. Also, a further simplification was made by assuming that the transition period T_0 (Eq. 5.6) is equal to the predominant (characteristic) period of ground motion T_C ($T_0 = T_C$). Considering these simplifications, the final form of the formulae for the reduction factor R_μ is:

$$R_\mu = (\mu - 1)\frac{T}{T_C} + 1 \quad T < T_C \tag{5.9}$$

$$R_\mu = \mu \qquad T \geq T_C \tag{5.10}$$

Eqs. 5.10 and 5.9 consider the equal displacement rule in the medium- and long-period range, and assume a linear relationship between the R_μ factor and the period T in the short period range, respectively (Figure 5.4). Of course, due to its simplicity, the formula is quite crude, especially in the short period range. Several researchers have suggested improvements that apply to specific cases, e.g., for masonry buildings. However, it is questionable how much improvement is justified considering the nature of the problem.

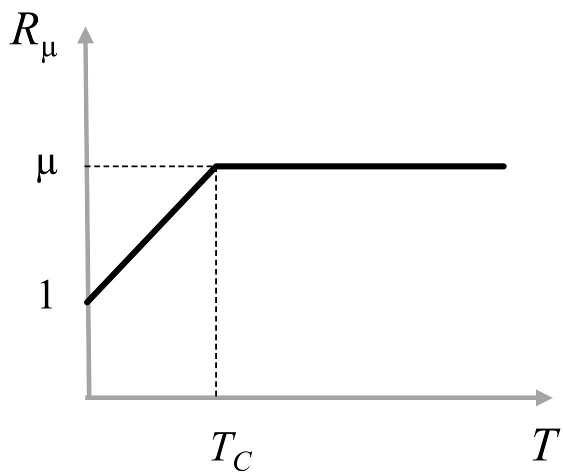


Figure 5.4. Simplified spectrum for reduction factor R_μ (R_μ-μ-T relation).

An advantage of the Eqs. 5.9 and 5.10 is the consideration of the frequency content of the ground motion (by T_C), which typically depends on the soil conditions and on the characteristics of the earthquake. The majority of equations for R_μ factors, provided in the literature, lack this feature.

The extremely simple R_μ-μ-T relation, defined by Eqs. 5.9 and 5.10, is used in the N2 method, as implemented in EC8. Note, however, this relation is not a mandatory part of the N2 method. Any R_μ-μ-T relation (i.e., any spectrum for reduction factor R_μ) can be used for the determination of the seismic demand in the N2 method, for example, the relation developed for infilled frames discussed in Chapter 9.

Starting from the elastic design spectrum and using Eqs. 5.9 and 5.10, the spectra for constant ductility factors μ can be obtained. As an example, the inelastic spectra in AD format corresponding to the EC8 elastic response spectrum for ground type B are shown in Fig. 5.5.

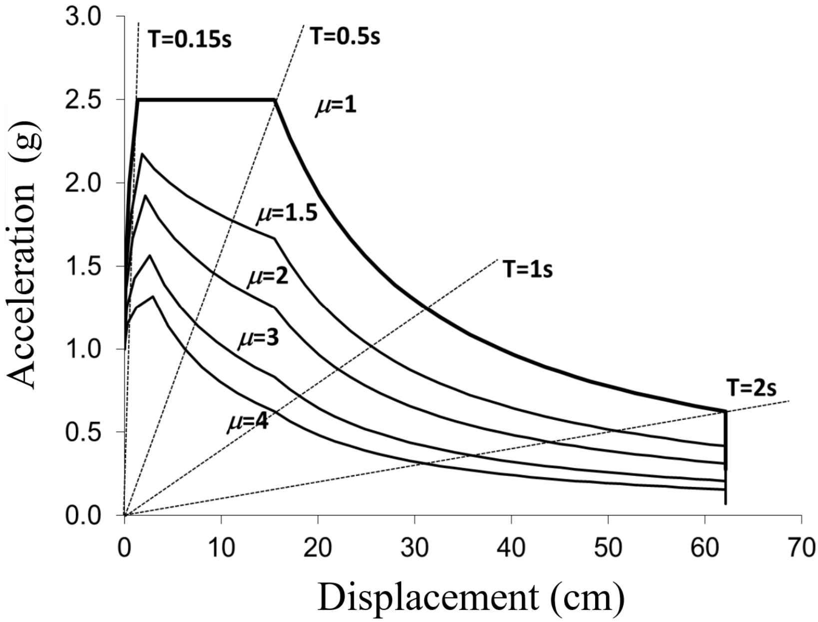


Figure 5.5. Elastic EC8 spectrum (type B soil, *PGA* = 1 g) and corresponding inelastic spectra for constant ductility.

6 SUMMARY OF THE BASIC N2 METHOD

All steps of the basic N2 method are presented in Fig. 4.1 in Section 4.3. In this chapter, the formulae needed for the application of the N2 method, using the notation of this monograph, are summarised. Some additional information that may facilitate the application is provided. The complete derivation is presented in (Fajfar 2000 and Fardis et al. 2015). All equations in this chapter apply to a planar model. In Section 6.6, the application to a spatial (3D) model is discussed.

6.1 Pushover Analysis

A nonlinear static (pushover) analysis is performed by subjecting a structure to a monotonically increasing pattern of lateral forces, representing the inertial forces that would be experienced by the structure when subjected to ground shaking. Gravity loads are kept constant. Under incrementally increasing lateral loads, various structural elements yield sequentially. Consequently, at each event, the structure experiences a loss of stiffness.

The vector of lateral loads **F** is determined as:

$$\mathbf{F} = \alpha \ \mathbf{m} \ \mathbf{\Phi} \tag{6.1}$$

where **m** is a diagonal mass matrix. The magnitude of the lateral loads is controlled by the scale factor α. The distribution of the lateral loads is related to the assumed displacement shape **Φ**, i.e., it represents the displacement shape weighted by the masses. The procedure can start either by assuming the displacement shape **Φ** and determining the lateral load distribution or by assuming the lateral load distribution and determining the displacement shape **Φ**. Note that Eq. 6.1 does not present any restriction regarding the distribution of lateral loads. The expressions provided in this chapter can be applied to any displacement shape and/or for any related distribution of lateral loads.

If the fundamental mode shape of the linear elastic structure is used as the assumed displacement shape, and if it remains constant during ground shaking, then the distribution of lateral forces is the same as the distribution of "seismic forces" which correspond to the fundamental mode in elastic modal analysis. In this case, the transformation factor Γ, determined according to Eq. 6.3, represents the mode participation factor in the modal analysis. In the inelastic range, the displacement shape changes over time. Eq. 6.1 represents an approximation of the "seismic forces". By assuming related lateral forces and displacements according to Eq. 6.1, the transformation from the MDOF to the equivalent SDOF system and vice-versa (see next section) follows from simple mathematics. No additional approximations are required, as in some other simplified procedures.

The selection of an appropriate vertical distribution of lateral loads is an essential step in pushover analysis. A unique solution does not exist. Fortunately, the range of reasonable assumptions is usually relatively narrow and, within this range, different assumptions produce similar results. One practical possibility is to use two different displacement shapes (load patterns) and to envelope the results.

Using a pushover analysis, a characteristic nonlinear force-displacement relationship of the MDOF system, also called the pushover curve, can be determined. In the case of buildings, base shear V and roof (top) displacement d_t are usually chosen as representative forces and displacements, respectively. The idealisation of the pushover curve can be performed either at the level of the MDOF system or at that of the SDOF system. In order to determine an idealised (typically bilinear) force-displacement relationship, engineering judgement has to be used. Some guidelines may be given in regulatory documents. For the graphical representation of the N2 method, the most convenient approach is to assume that the post-yield stiffness is equal to zero. It should be emphasised that moderate strain hardening does not have a significant influence on displacement demand and that the inelastic demand spectra presented in Chapter 5 apply approximately to systems with zero or small strain-hardening.

6.2 Equivalent SDOF System

In the N2 method, seismic demand is determined by using inelastic response spectra. Inelastic behaviour is taken into account explicitly. Consequently, the structure should, in principle, be modelled as an SDOF system.

The force-displacement relationship determined for the MDOF system (the V-d_t diagram) can be transformed to that of the equivalent SDOF system (the f-d diagram) with the help of the transformation factor Γ:

$$f = \frac{V}{\Gamma}, \qquad d = \frac{d_t}{\Gamma} \tag{6.2}$$

The transformation factor Γ is determined as (the value of $\mathbf{\Phi}$ at the roof should be equal to 1.0):

$$\Gamma = \frac{\mathbf{\Phi}^T \ \mathbf{m} \ \mathbf{1}}{\mathbf{\Phi}^T \ \mathbf{m} \ \mathbf{\Phi}} \tag{6.3}$$

where the numerator represents the mass of the equivalent SDOF system m^*:

$$m^* = \mathbf{\Phi}^T \ \mathbf{m} \ \mathbf{1} \tag{6.4}$$

Γ controls the transformation from the MDOF to the SDOF model and vice-versa. The same Γ value applies to the transformation of both displacements and forces (Eq. 6.2). The shape of the pushover curve and the initial stiffness are the same for both the MDOF and SDOF systems.

The period T^* of the idealised SDOF system is computed as:

$$T^* = 2\pi \sqrt{\frac{m^* d_y}{f_y}} \tag{6.5}$$

where d_y and f_y are the yield displacement and the yield strength of the idealised SDOF system, respectively.

The so-called capacity diagram in AD format is obtained by dividing the forces in the force-deformation (f - d) diagram by the equivalent mass m^*, i.e., as f/m^*. Note that f/m^* can be transformed into V/M^*, where M^* is the effective mass for the fundamental mode of the MDOF system: $M^* = m^{*2} / (\mathbf{\Phi}^T \mathbf{m}\, \mathbf{\Phi})$.

6.3 Seismic Demand for the SDOF System

Inelastic demand spectra in AD format can be determined from elastic demand spectra (see Chapter 5). The procedure for determining seismic demand for the equivalent SDOF system is illustrated in Figs. 6.1a and 6.1b.

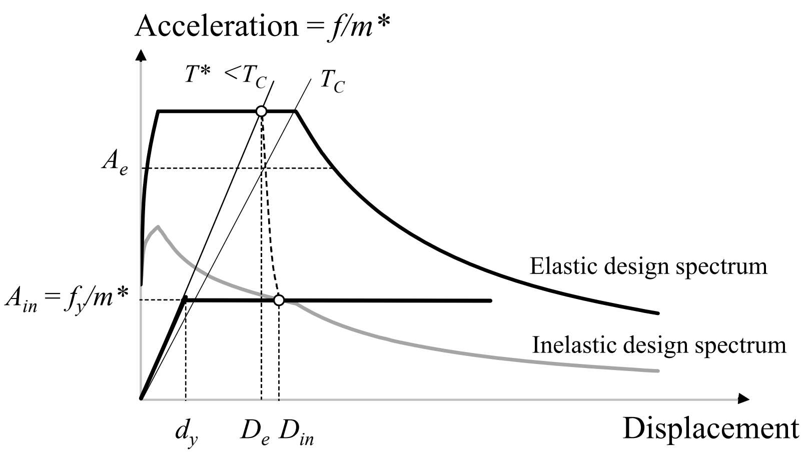


Figure 6.1a. Determination of the seismic demand for the equivalent SDOF system with the period in the short-period range.

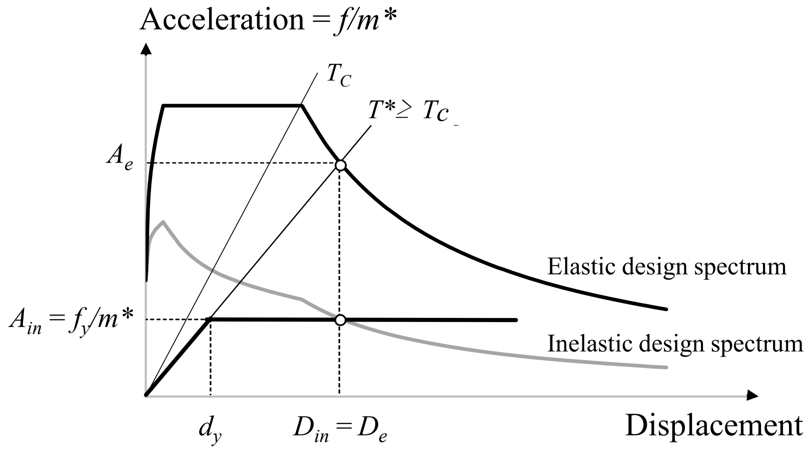


Figure 6.1b. Determination of the seismic demand for the equivalent SDOF system with the period in the medium/long-period range.

Both the demand spectra and the capacity diagram appear in the same graph. The intersection of the radial line corresponding to the elastic period of the idealised bilinear system, T^*, with the elastic demand spectrum in AD format defines the acceleration demand A_e, i.e., the acceleration (strength) capacity required for elastic behaviour, and the corresponding elastic displacement demand, D_e. The yield acceleration represents both the acceleration demand, A_{in}, and the strength capacity of the inelastic system, f_y/m^*. The reduction factor R_μ can be determined as the ratio between the forces corresponding to the elastic and inelastic systems (Eq. 3.2), which can also be expressed in terms of accelerations:

$$R_\mu = \frac{A_e(T^*)}{A_{in}} \tag{6.6}$$

If the period T^* is longer than or equal to the characteristic period of the ground motion T_C, the equal displacement rule applies, the ductility demand is equal to the reduction factor due to ductility ($\mu = R_\mu$), and the inelastic displacement demand D_{in} is equal to the elastic displacement demand D_e (see Eq. 5.10 and Fig. 6.1b).

If the period of the system is shorter than T_C (Fig. 6.1a), the ductility demand can be calculated from the rearranged Eq. 5.9:

$$\mu = (R_\mu - 1)\frac{T_C}{T^*} + 1 \qquad T^* < T_C \tag{6.7}$$

The inelastic displacement demand D_{in} can be determined either from the definition of ductility or from Eqs. 5.3 and 6.7 as:

$$D_{in} = \mu d_y = \frac{D_e}{R_\mu}\left(1 + \left(R_\mu - 1\right)\frac{T_C}{T^*}\right) \qquad T^* < T_C \qquad (6.8)$$

In both cases (i.e., $T^* \geq T_C$ and $T^* < T_C$), the inelastic demand in terms of accelerations and displacements corresponds to the intersection point of the capacity diagram with the demand spectrum corresponding to the ductility demand μ. At this point, the ductility factor determined from the capacity diagram and the ductility factor associated with the intersecting demand spectrum are equal.

All the steps in the procedure can be performed numerically without using a graph. However, visualisation of the procedure may help in better understanding the relations between the basic quantities.

At this stage, the displacement demand can be modified if necessary, e.g., to take into account larger displacements in the case of systems with narrow hysteresis loops or negative post-yield stiffness.

6.4 Seismic Demand for the MDOF System

The displacement demand for MDOF systems (i.e., the target displacement), d_r, is obtained from Eq. 6.2 by multiplying the displacement demand of the equivalent SDOF system, $D_{in} = d$, with the transformation factor Γ. Under monotonically increasing lateral loads with a fixed pattern per Eq. 6.1, the structure is pushed (at least) to the target displacement, d_r. Relevant local quantities (e.g., storey drifts, chord rotations, internal forces) corresponding to d_r are determined, assuming that the distribution of deformations throughout the structure in the static (pushover) analysis approximately corresponds to that which would be obtained in the dynamic analyses.

If mean demand spectra and mean value properties of the materials are used in analysis, the target displacement d_r represents a mean value for the applied earthquake loading. There is a considerable scatter about that mean. Consequently, it is appropriate to investigate the likely building performance under extreme load conditions that exceed the design values, e.g., to carry out the analysis to at least 150% of the calculated top displacement.

6.5 Performance Evaluation

The expected performance can be assessed by comparing the seismic demands, determined in the previous section, with the capacities. Both sets of quantities should correspond to the same limit state (performance level), e.g., to the NC limit state. Comparisons can be made both at the global and at the local level. In the case of inelastic behaviour, the relevant quantities are the roof displacement and the storey drifts, whereas at the local level a convenient quantity is chord rotation. Forces and accelerations are relevant for brittle elements and for equipment that is sensitive to accelerations.

Collapse prevention is the main objective of any design. An adequate safety margin against collapse under the expected maximum seismic load needs to be assured. However, it is extremely difficult to predict a physical collapse that involves large deformations, significant second-order effects, and complex material degradation due to localised phenomena. Despite considerable research efforts, methods for the reliable assessment of collapse are not yet available. In practice, the Near Collapse (NC) limit state is often used as a conservative approximation of structural collapse.

No guidance is provided as to how capacity at the NC limit state could be determined. The NC limit state of an individual structural element is usually defined as the point on its pushover curve at which the horizontal resistance drops by 20%. At the level of the structure, a commonly accepted quantitative definition of the NC limit state does not exist. An option is a similar definition as in the case of individual elements, e.g., at a 20% drop of the lateral resistance of the structure. However, this definition, which seems to be the most appropriate, cannot be applied in nonlinear response history analysis or in pushover analyses with simplified models, e.g., in the case of models without strength-degradation. A more practical definition is based on the assumption that the NC limit state of the structure is reached when the first important vertical element (i.e., a column or a wall) reaches the NC limit state. Note, however, that this definition may be non-conservative in the case of a structure with significant second-order (P-Δ) effects.

The capacities of structural elements (beams, columns, walls) are empirically based. Seismic codes may provide information for the quantification of the capacity of components and/or mechanisms. Some work on capacity of reinforced concrete members has also been done by our research group. It is summarised in Chapter 11. In any case, the capacity of structural members and whole structures is a problem which still requires extensive research.

6.6 Spatial (3D) Building Model

For a 3D building model, separate analyses are performed in each of the two orthogonal horizontal directions. The lateral loads (determined according to Eq. 6.1) are applied at the mass centres of different storeys in one direction only. This is a special case, which also requires that the assumed displacement shape $\mathbf{\Phi}$, has non-zero components in one direction only. In such a case, all the equations derived for the planar system can be directly used for the 3D system, by considering only the direction under investigation. Note that even in this special case of assumed uncoupled displacement shape, the displacements determined by pushover analysis of an asymmetric structure are generally coupled, i.e., they have components in three directions (two translations and torsion). The relevant results (e.g., the displacements, storey drifts, joint rotations, and forces in brittle elements, which should remain in the elastic region), obtained by two independent pushover analyses, are combined through the SRSS rule. Static torsional effects are included in results. The dynamic torsional effects may, however, be quite different from the static ones. For a better estimation of torsional effects, the Extended N2 method can be used (see Chapter 12).

7 ENERGY

The aseismic design philosophy for ordinary building structures relies strongly on energy dissipation through large inelastic deformations. Structures have adequate seismic resistance if their limit deformation capacity exceeds the seismic demand in the case of severe earthquakes. It has been widely recognised that the level of structural damage due to earthquakes does not depend only on maximum displacement, which is the structural response parameter most widely employed for evaluating the inelastic performance of structures, and that the cumulative damage resulting from numerous inelastic cycles is also important. Dissipated hysteretic energy is the structural response parameter that is supposed to be correlated with cumulative structural damage. On the other hand, the input energy is related to the cumulative damage potential of ground motions; it is relatively insensitive to structural characteristics. Energy parameters incorporate the influence of both strength and deformation and include the effect of the duration of strong ground motion. An important advantage of energy formulation is also the replacement of vector quantities (displacements, velocities and accelerations) by scalar energy quantities.

As early as in the mid-1950s, Housner (1956) proposed "a limit design type of analysis to ensure that there was sufficient energy-absorbing capacity to give an adequate factor of safety against collapse in the event to extremely strong ground motion". However, the energy concept was long ignored in earthquake-resistant design because of the apparent complexities in the quantification of energy demands and capacities and their implementation in the design process. It was only in the 1980s that the energy concept attracted extensive attention in the research community. Notable contributions were made by Akiyama (1985) and Bertero and co-authors (e.g., Uang and Bertero 1988), among others. Considering the importance of energy, one of two topics discussed at the first Bled Workshop in 1992 was "Energy concepts and damage models" which included 10 papers related to energy.

A considerable part of the work of our research group in the 1980s and '90s was devoted to determining input and hysteretic energy demand, and to using them in seismic analysis. Later, we also did some limited work on the energy dissipation capacity of RC elements.

In the early 1980s, with Matej Fischinger, we computed and analysed the inelastic spectra for the ground motion records obtained during the 1979 Montenegro earthquake. In addition to displacement and displacement ductility spectra, we also determined spectra for hysteretic and input energy and published the results at different conferences. Among other places, I presented the results at the IAEA (International Atomic Energy Agency) Meeting on Earthquake Ground Motion and Seismic Evaluation of Nuclear Power Plants in Moscow in 1986 (Fajfar and Fischinger 1989). At this meeting, I met Professor Hiroshi Akiyama, who had just published the English version of his seminal book on design based on the energy concept (Akiyama 1985). This book is the primary reference for all researchers working on energy issues in earthquake engineering. He kindly gave me his book with a dedication, and this was the start of our wonderful friendship and a strong motivation for my further interest in energy.

An important contribution to inelastic spectra, including energy parameters, was made by Tomaž Vidic within his master's thesis in 1989. We presented the results in two journal papers (Fajfar et al. 1989, 1990). Recognising the importance of the duration of ground motion on the energy demand of inelastic structures, a measure of earthquake ground motion capacity to damage structures with fundamental periods in the medium-period (velocity-controlled) range was proposed (Fajfar et al. 1990):

$$I = v_g t_D^{0.25} \tag{7.1}$$

where v_g is the peak ground velocity and t_D is the significant duration of ground motion. Moreover, simple formulae for roughly determining the seismic demand in terms of relative displacement D (for 5% damping) and input energy per unit mass E_I/m in SDOF systems with natural periods T in the medium-period range were proposed (Fajfar et al. 1989):

$$D = 0.17\, t_D^{0.25} v_g T \qquad T_1 \le T \le T_2 \tag{7.2}$$

$$\frac{E_I}{m} = 2.2\, t_D^{0.5} v_g^2 \qquad T_1 \le T \le T_2 \tag{7.3}$$

The medium period range comprises the periods between T_1 and T_2. Empirical formulas were used for both corner periods as a function of peak ground acceleration, velocity and displacement.

An approach including cumulative damage indicators, e.g., hysteretic energy, is not without difficulties in terms of its practical application. Designers are reluctant to change the state-of-the-practice radically. Therefore, a new procedure has a better chance of being accepted if it represents only a minor change in a concept that is well understood and already widely employed in practice. A promising technique fulfilling this condition seems to be the concept of equivalent (reduced) ductility factors, which take into account the influence of cyclic load reversals (Fajfar and Fischinger 1990, Fajfar 1992). Such an approach is a minor adjustment to a concept that is well understood and commonly used in practice.

The relation between the equivalent ductility factor μ and the ultimate monotonic ductility factor μ_u depends on the damage model used. For reinforced concrete structures, the Park-Ang model (Park et al. 1984, 1987) has been widely employed:

$$DM = \frac{d}{d_u} + \beta \frac{E_H}{f_y d_u} = \frac{\mu}{\mu_u} + \beta \frac{E_H}{f_y d_y \mu_u} \tag{7.4}$$

where d and d_u are the actual and ultimate displacement, respectively, and μ and μ_u are the corresponding ductilities. Parameter β depends on structural characteristics, mainly on detailing. The experimental values of β reported in the literature are in a very wide range, between about -0.3 to 1.2, with a median of about 0.15. E_H is the dissipated hysteretic energy, d_y is the yield displacement, and f_y is the yield strength. DM is the damage index. DM = 1 represents failure.

In (Fajfar 1992), the non-dimensional parameter γ was defined:

$$\gamma = \frac{\sqrt{(E_H/m)}}{\omega D} = \frac{1}{\mu}\sqrt{\frac{E_H}{f_y d_y}} \tag{7.5}$$

where m is the mass of the system, ω is the natural frequency, and D=d is the actual displacement. (Note that capital D is used for displacement in the first part of Eq. 7.5 because the equation is used for the determination of spectra.) From Eqs. 7.4 and 7.5 the equivalent (reduced) ductility factor can be determined as:

$$\mu = \frac{-1 + \sqrt{1 + 4DM\beta\gamma^2\mu_u}}{2\beta\gamma^2} \tag{7.6}$$

Eq. 7.6 defines the reduction of ductility due to low-cycle fatigue. It is controlled by the parameters β and γ, by the actual amplitude of vibration (expressed in terms of μ) and by the permissible damage index DM. The effect of the low-cycle fatigue can be taken into account in a simple and efficient way if the equivalent (reduced) ductility factor is used instead of the usual ductility factor in different formulae, e.g., in formulae for ductility-dependent reduction factor R_μ (Chapter 3). Using equivalent ductility factor, the explicit use of hysteretic energy is avoided. The hysteretic energy demand and capacity are implicitly included in parameter γ, and parameter β in the Park-Ang damage index, respectively.

In order to obtain qualitative and quantitative data on the parameter γ, a parametric study of the inelastic response of SDOF systems was carried out. The influence of the ground motions of very different duration on parameter γ is shown in Figure 7.1. The results reveal that the γ values for the long duration ground motion (Chile and Mexico group of records) are much larger (indicating more substantial cumulative damage) than those for the short duration ground motions

(Friuli and Banja Luka). In the case of a standard ground motion of usual duration, typical values of γ are between 0.8 and 1.2. Later, in the paper (Fajfar and Vidic 1994), we proposed formulae for estimation of γ values.

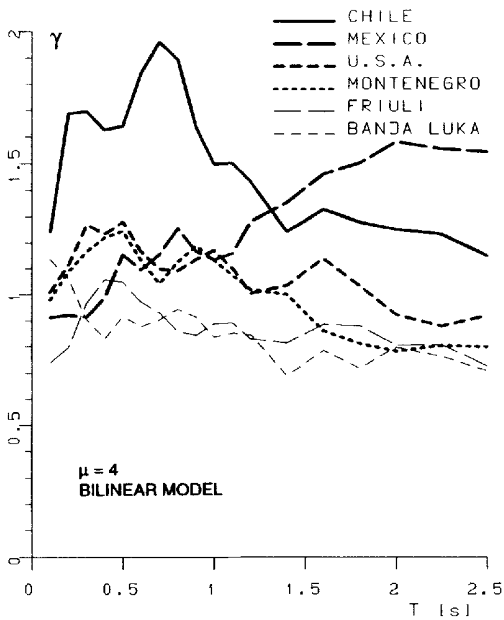


Figure 7.1. Influence of input motion on the mean values of the parameter γ (damping proportional to instantaneous stiffness) (from Fajfar 1992).

In the process of publishing the paper on equivalent ductility factor (Fajfar 1992), I encountered the worst experience with reviewers in my whole professional life. Usually, young researchers publish their first journal papers with their advisors as co-authors. The advisors, typically established researchers, are not only supposed to help young researchers in writing according to international standards, but often, by putting their names on the manuscript, also contribute to an impression of the quality of the work. Since I have never had such an advisor, I wrote and submitted all my papers alone or together with my students. The great majority of reviewers were constructive and fair. Their comments mostly helped to substantial improvements to the manuscripts. Unfortunately, this was not the case with this paper. I cannot resist reproducing the complete review by one of the reviewers:

> The reviewer notes that this paper contains a recasting of some of the work presented in an earlier manuscript by the author, with some new additions.

> Indeed it is of interest to know some of the work underway in eastern Europe, but since only the barest of details is presented, one cannot judge the needed basis for applicability in research or practice. Mere summary is of little value in our western literature. I recommend that the paper be declined with thanks.

This review, which clearly demonstrates the reviewer's prejudices (please note "eastern Europe" and "western literature"), angered me. I complained to the responsible editor, and he hired a third reviewer. Finally, the paper was published after some revision. In fact, it became one of my most cited papers. Although all the reviewers were anonymous, I imagine that Vitelmo Bertero was the third reviewer. He obviously liked the γ factor proposed in my paper and immediately started using it (Bertero and Bertero 1992) and citing it, thus significantly contributing to the visibility of my research. So, in the end, the unfair review was actually of great help for my career.

In 1994, two companion papers were published in EESD, presenting inelastic spectra for strength and relative displacement (Vidic et al. 1994) and hysteretic and input energy (Fajfar and Vidic 1994). Both papers were mainly based on the results obtained within the PhD thesis of Tomaž Vidic. Spectra for strength and relative displacement are discussed in Chapter 5 on inelastic spectra, whereas energy spectra are summarised in this chapter. It is necessary to note that spectra related to energy are consistent with the strength and displacement spectra, i.e., they are interrelated and based on the same assumptions.

In a typical design procedure, the elastic pseudo-acceleration spectrum A_e and the ductility factor μ are prescribed. Starting from these values, spectra for strength and relative displacement, as well as for hysteretic and input energy per unit mass can be easily determined, provided that the spectra for three non-dimensional parameters, R_μ, γ, and E_H/E_I are known. These three parameters have been chosen as being the most convenient for approximate representations by simple formulae.

Hysteretic energy spectra E_H can be obtained from the first part of Eq. 7.5. Considering the relation between relative displacement $D=D_{in}$ and elastic acceleration spectra A_e (Eq. 5.3), which is for convenience repeated here in a slightly different form:

$$D = \frac{\mu A_e}{R_\mu \omega^2} \tag{7.7}$$

the expression for hysteretic energy spectra can be derived:

$$\frac{E_H}{m} = \left(\frac{\gamma \mu A_e}{R_\mu \omega}\right)^2 \tag{7.8}$$

where R_μ is the strength reduction factor due to ductility and γ is the parameter related to hysteretic energy (Eq. 7.5). The input energy E_I can be related to the hysteretic energy by the equation:

$$\frac{E_I}{m} = \frac{1}{E_H/E_I}\frac{E_H}{m} \tag{7.9}$$

Based on the results of the parametric studies, simple empirical formulae for the energy-related non-dimensional parameters γ and E_H/E_I were proposed in (Fajfar and Vidic 1994).

In parametric studies, the coefficients of variation were also determined. The values of the coefficients of variation for γ are mostly in the range from 0.1 to 0.2. They are considerably smaller than those typical for the great majority of response parameters involved in earthquake-resistant design. The coefficients of variation for energies are larger than those in the case of traditional response parameters (e.g., displacements), and amount to about 0.5 to 0.8. The ratio of energies, E_H/E_I, proved to be the most stable parameter involved in earthquake-resistant design, with a coefficient of variation smaller than that of γ.

Parameter γ and the concept of equivalent ductility factors were used in the paper on the N2 method published in EESD in 1996 (Fajfar and Gašperšič 1996, see Section 4.2). Later, energy issues mostly disappeared from our research agenda. Although an approach based on energy is promising and has some potential advantages compared to the standard approach, it has not convinced enough researchers to build a critical mass to realise a breakthrough of such an approach. Some significant contributions have been made on the demand side. On the capacity side, however, not much research has been performed this far, so reliable data on capacities of structural elements and structures for dissipation of energy, which are heavily based on experimental results, are missing. Note that the energy dissipation capacities are influenced not only by structural characteristics but also on the characteristics of ground motion. The whole inelastic deformation time-history, i.e., the number, sequence and relative amplitude of the inelastic excursions, may have a substantial effect on cumulative damage. In practical damage models, however, it is very difficult to take into account the details of a time-history. Even if the explicit use of hysteretic energy is avoided, e.g., by using an equivalent ductility factor, there is still a problem related to the damage model to be used. For example, in the Park-Ang model, the weak point is the parameter β, which is entirely based on relatively rare experimental results. For all these reasons we gave up and, with some exceptions, halted our research on energy issues in the mid-1990s.

Later, I co-authored two papers on energy with Bob Chai from the University of California in Davis as the main author (Chai et al. 1998, Chai and Fajfar 2000). In these papers, mainly the results of his research work were presented. Bob was familiar with our work in Ljubljana, and he used our results in his work. So, he kindly invited me to cooperate as a consultant in writing the two papers.

We have also done some work on the capacity side. We studied energy dissipation capacity and the deterioration of deformation capacity due to cumulative damage by means of a non-parametric empirical approach (see Chapter 11), using empirical data on rectangular reinforced concrete columns that failed in flexure. The results were presented in (Poljanšek et al. 2009).

Let me finish this section on energy on a more optimistic note. With the development and gaining popularity of passive control devices, which directly influence the energy balance in a structure (e.g., energy dissipation devices), the explicit use of energy concepts in seismic design may experience another period of growth. For example, in the revised EC8, the “Energy-balance based analysis” is one of the options for analysis of buildings with energy dissipation systems. Seismic energy formulation presents a natural way to understand the effect of supplemental damping. The use of energy-based analysis for a specific problem may trigger some more research on energy concepts in seismic analysis. Also, in the revised EC8, it is required that the models for the estimation of deformation capacity of structural elements should consider cyclic degradation. This basically applies to existing structures. Equivalent ductility factors can be used in order to comply with this requirement.

8 PSEUDO-3D STRUCTURAL MODEL

8.1 Introduction

For a seismic analysis, the characteristics of a structural model are at least as important as the characteristics of the analysis procedure. This chapter is devoted to the so-called pseudo-three-dimensional (pseudo-3D) structural model, which, for many building structures, represents an excellent option for modelling. By taking advantage of the typical characteristics of building structures, it not only dramatically reduces the computational efforts but also greatly increases the transparency of the results and significantly contributes to a better understanding of the structure and its behaviour.

My initial work in structural analysis was limited to the linear response. The final result of this work was the computer program for elastic analysis of multi-storey structures EAVEK (Fajfar 1976), based on the pseudo-3D structural model of a building structure. The program has had a significant impact on the culture of earthquake engineering in Slovenia and, more broadly, in the former Yugoslavia. Later, my doctoral student Vojko Kilar and I extended the analysis procedure into the nonlinear range and developed a relatively simple and transparent approach for nonlinear static analysis of complex building structures, together with the computer program NEAVEK (Nonlinear EAVEK) (Kilar and Fajfar 1997). In this chapter, the development of the analysis procedure in linear range and the EAVEK program are briefly described. Then the extension to the nonlinear range is summarised.

8.2 Beginnings

The topic of my master's thesis was the static and dynamic analysis of asymmetric multi-storey building structures subjected to horizontal loading. Note that before the implementation of the Bologna system in the late 2000s, the master's degree (called magister's degree) at the University of Ljubljana was a research-oriented degree awarded for 2 or 3 years of study following the bachelor's degree programme (which lasted 4.5 to 5 years) and the defense of a master's (magister's) thesis. In order to be promoted to a doctoral degree after magister's degree, it was required to write and defend a doctoral thesis. In the late 1960s and early 1970s, the use of computers in structural engineering in Slovenia was in its infancy. I was dependent solely on the literature. Browsing through various publications in the library, I found the paper by Winokur and Gluck (1968), in which a structural model consisting of vertical macroelements (e.g., planar frames and walls) connected with rigid floor diaphragms was used for the static analysis of asymmetric multi-storey structures. I liked the model and adopted its basic ideas in my work. In my approach, the condensed stiffness matrices of macroelements were determined by inverting the condensed flexibility matrices of the macroelements, which can be computed for most typical macroelements with closed-form formulae. I established a library of macroelements and extended the approach to the dynamic response spectrum method. As a result of this work, a comprehensive method for linear static and dynamic (response spectrum) analysis of multi-storey building structures was developed together with a computer program. The master's thesis was published in 1972 in the Slovenian language as Report No.1 of the newly established Computing Centre of our faculty. A very short paper (3 pages) was published in the German journal Die Bautechnik (Fajfar 1973). The computer program was successfully used for the seismic analysis of buildings in Slovenia and in teaching and research work at our faculty.

After completing my master studies, I spent 10 months at the Ruhr University in Bochum in Western Germany, where I was able to fully concentrate on the work of my doctoral thesis. The topic was a general elastic analysis of multi-storey building structures, i.e., an extension of the work performed in my master's thesis. I was attempting to develop a solid theoretical background. The dynamic analysis, which was originally limited to the modal response spectrum analysis, was complemented with the response history analysis. Stability analysis and the second-order effects were added. The library of macroelements was extended with several new types of shear walls. During my stay in Bochum, I succeeded in drafting the theoretical part of my thesis together with the corresponding computer program. After some additional work in Ljubljana, I submitted my PhD thesis entitled "Numerical analysis of statics, dynamic and stability problems for multi-storey structures" at the University of Ljubljana at the end of 1973, a few months after my return from Bochum. Formally, my advisor in Ljubljana was Ervin Prelog, who was at that time Professor of Statics of Structures at our faculty. His background was in mechanical engineering, but he specialised in the static analysis of structures. He was heavily involved in consulting work and was considered to be a top specialist for static analysis of buildings in Slovenia, which was at that time typically performed with slide-rule calculations. Later he became Rector of the University of Ljubljana. Prelog and the examination committee were satisfied with my work, so I defended the thesis in the spring of 1974.

Unfortunately, in my surroundings, in the 1970s, it was not usual to publish results in international literature, and there was nobody around with experience in such an undertaking. The first step in my learning process was a paper which I submitted to an ASCE journal. Looking back, it was really very poorly written and, of course, it was rejected. In the following years, I managed to prepare and publish very few publications presenting partial results of the thesis, one at the 5th European Conference on Earthquake Engineering (Fajfar 1975), another one (in German) in Die Bautechnik (Fajfar 1978), and a few in national journals and conference proceedings.

8.3 Linear Analysis - EAVEK Program

The method, developed in my PhD thesis, can be used for linear static, dynamic and stability analysis of quite general asymmetric multi-storey structures. The special feature of the method is the use of a pseudo-3D structural model that takes into account the essential characteristics of the building structures. The model consists of assemblages of two-dimensional macroelements (substructures) such as frames, walls, coupled walls and walls on columns that may be oriented arbitrarily in the floor plan. Each macroelement extends from the foundation to any floor and is assumed to resist load only in its own plane, but the building as a whole can resist load in any direction. Torsional macroelements can be included. The macroelements are connected at each floor level by diaphragms that are assumed 1) to be rigid in their own planes and 2) to have no out-of-plane flexural stiffness. The consequence of the first assumption is that all displacements of the floor slab are known if the horizontal displacements and the torsional rotation of one point of the floor are known. The second assumption takes into account the typical situation in buildings, in which the horizontal stiffnesses of vertical structural elements are substantially larger than the vertical stiffness of the floor slabs. The consequence of these two assumptions is that individual macroelements can be treated independently and connected with floor slabs, through which only horizontal shear forces are transmitted. The third assumption is based on the fact that, in typical building structures, the great majority of the mass is concentrated on the floor levels and on the roof. Thus it is reasonable to assume concentrated masses at the floor levels.

Using the above assumptions, the pseudo-3D model is greatly simplified compared to a full three-dimensional model, which does not take into account specific features of typical building structures. In the case of a spatial structure, the pseudo-3D model has three degrees of freedom for each floor level (two horizontal translations and one rotation about the vertical axis). All other degrees of freedom are eliminated by static condensation on the macroelement level, by assuming rigid links, or by ignoring them. In a special case of a planar structure, there is only one degree of freedom per floor.

In the pseudo-3D model, the compatibility of axial deformations in columns common to more than one frame, or in intersecting shear walls is neglected. This certainly represents a limitation of the model. However, it is considered that for most buildings this is an acceptable approximation, with the possible exception of some tall, slender buildings or tube-type structures. The advantages of a pseudo-3D model over a fully three-dimensional model are easier data preparation, easier interpretation and checking of the results, and much higher computational efficiency. It allows a transparent analysis procedure and a better understanding of the structural behaviour. As all the essential degrees of freedom are included in the model, the results at the global level are sufficiently accurate in most cases. On the level of individual elements, additional static analyses are required, which can be easily performed and automated.

The matrices for the whole structure are formed from the matrices of the individual macroelements. Closed-form formulae were developed for the determination of the flexibility matrices for several standard macroelements. Stiffness matrices for individual macroelements are obtained by the inversion of the flexibility matrices and transformed into the global coordinate system. The structural stiffness matrix is determined by summing the transformed matrices of all macroelements. For more details, see (Fajfar 1978).

With the use of the described model, a user-friendly computer program called EAVEK (the acronym comes from the Slovenian title Elastična Analiza VEčetažnih Konstrukcij, which means "the elastic analysis of multi-storey structures") was developed in the early 1970s within my Master's and PhD theses. The program became standard analysis software in Slovenian design offices.

The EAVEK program was written in FORTRAN. Free-format input was used, similar to those in the most widely used STRESS program (Fenves 1964). The program contained many user-oriented features of value for the building analyst: detailed yet simple documentation, problem-oriented input, automatic error checking, and problem-oriented output in the final report format. A graphical presentation of results on the printer was possible (so-called "print-plot") and on a plotter. The program ran on small IBM 1130 type computers with 64k of memory. Nevertheless, it was possible to analyse buildings up to 30 storeys high. All computations were performed using two fields of the total size of 9000, in which all variables of type REAL and INTEGER were recorded. In the coding of the free format input, Matej Fischinger was involved.

The EAVEK program, together with some other computer programs developed at the computing centre of our faculty, which later expanded and became in 1980 the Institute of Structural Engineering, Earthquake Engineering and Construction IT, has been widely used for teaching and research and has been very well accepted by the practising engineers. For about two decades, it was used for practically all building design projects in Slovenia. It has also been popular in the rest of former Yugoslavia. Thanks to my friend Mingwu Yuan from Peking University in Beijing, whom I met during my stay at the UC Berkeley in 1980, and who after returning to China developed one of the first (if not the first) version of the SAP program running on personal computers, EAVEK program also experienced some applications in China. In the next two decades or so, several updates and improvements of the program have been made. Due to the sound theoretical background, it was straightforward to accommodate all changes of seismic regulations. In the early 1990s, much work on the new versions of the EAVEK program was done by Vojko Kilar within his bachelor's and master's theses.

The key for the success of the program was a simple and transparent structural model, which closely followed the intuitive thinking of structural engineers, required an understanding of the structural system and its main load-bearing elements, was very easy to use, and provided transparent results of adequate accuracy for the vast majority of building structures. Frankly, if I was asked about the major achievement in my professional life, I would choose the EAVEK program, and the structural model and the analysis method behind it. Although the approach and the program have practically not been presented to the international community and have not received international recognition, as the N2 method did, they made a profound impact on the understanding of the structural response of buildings and thus on seismic resilience in my home country Slovenia.

Towards the end of the 20th century, the rapid development of computer software and hardware brought utterly new possibilities, which have led to significant changes in new computer programs intended for commercial use. Highly sophisticated pre- and postprocessors have been developed. Research groups at universities have no longer been able to update application software according to user requirements. This work was taken over by companies specialised in the development and distribution of programs. We stopped updating the EAVEK program and other application programs for structural analysis. In teaching, research and in practice, commercial programs slowly replaced our own programs, including EAVEK, although the theoretical foundations of this program remain valid today. The new software has not only enabled automated input data preparation and advanced presentation of results but also the use of arbitrarily complex structural models, which is a double-edged sword. When using very demanding models with thousands or even hundreds of thousands of degrees of freedom, there is a risk of losing understanding of the structural behaviour. William Hall, with extensive research and design practice, said “A sophisticated analysis is not a substitute for a good engineering understanding of the problem” (Hanson and Reitherman 2015). Due to the large amount of data and results (the vast majority of them are usually entirely irrelevant), there is a much greater risk of error in very complex models than in the case of simpler models. Also, it should be considered that, in the case of seismic analyses, due to the significant uncertainties in determining the seismic action and the non-linear behaviour of the structure, even the most complex models can only give rough approximations to the actual response. Thus, it makes sense to use simplified models that represent an appropriate compromise between “accuracy” and simplicity. For these reasons, my younger colleagues, who had been frustrated when using sophisticated general-purpose programs for simple everyday problems, revived the EAVEK program. An online application of the program has been developed in accordance with current guidelines of the service-oriented architecture, whereas the core of the program with all the theoretical background remained intact. A paper describing the new online version of the EAVEK program was published (in Slovenian) in the leading Slovenian civil engineering journal (Klinc et al. 2016).

8.4 Nonlinear Analysis – NEAVEK Program

The model, described in the previous section, was extended into the inelastic range (Kilar and Fajfar 1997). The work was done within the doctoral thesis of Vojko Kilar, who was a very bright student with a very good sense for engineering applications. After completing his doctoral degree, he continued his academic career and became Professor of Structural Engineering at the Faculty for Architecture in Ljubljana. In parallel, he has been quite active in consulting.

In order to extend the procedure in the nonlinear range, for four standard macroelements an approximate bilinear or multilinear base shear-top displacement relationship was determined based on the initial stiffness, the strength at which the assumed plastic mechanism forms, and assumed post-yield stiffness (Figs. 8.1 and 8.2). For each macroelement, one or more possible plastic mechanisms are assumed.

For frames, three main types of plastic mechanism, as proposed by Mazzolani and Piluso (1996), are anticipated (Fig. 8.2). The global-type mechanism is a special case of the Type 2 mechanism. For three macroelement types (walls, walls on columns and frames), elastic behaviour is assumed until the plastic mechanism is formed. After the formation of the plastic mechanism, the force-displacement relationship is governed by the post-yield stiffness, which is arbitrarily assumed on the macroelement level. So, the base shear-top displacement relationship of a macroelement is bilinear, provided that the vertical distribution of lateral loading is constant. If this distribution changes during the loading history (this always happens, in principle, when one of the elements in the structural system yields) then the slope of the base shear - top displacement line also changes.

In the case of a coupled wall, static analysis of the structural model shown in Fig. 8.1 is needed in order to determine the elastic base shear - top displacement relationship. A gradual formation of the plastic mechanism is assumed. Consequently, the base shear-top displacement relation is piecewise linear; the stiffness changes after the yielding of different elements of the coupled wall (beams, walls).

The formulae for the determination of the condensed flexibility matrices for the three macroelement types (walls, walls on columns and frames) are given in (Kilar and Fajfar 1997).

Pushover analysis of the whole structure is performed as a sequence of linear analyses, using an event-to-event strategy. An event is defined as a discrete change of the structural stiffness due to the formation of a plastic hinge (or the simultaneous formation of several plastic hinges) in a macroelement.

The method was implemented in the computer program NEAVEK (Nonlinear EAVEK). At each step, NEAVEK automatically uses the program EAVEK, which performs a linear elastic analysis.

The described procedure for the pushover analysis of building structures is capable of estimating several important characteristics of nonlinear structural behaviour, especially the real strength and the plastic mechanism of the whole structure. It also provides data about the sequence of yielding of different parts of the structure, and an estimate of the required ductilities of the different macroelements in relation to the target maximum displacement.

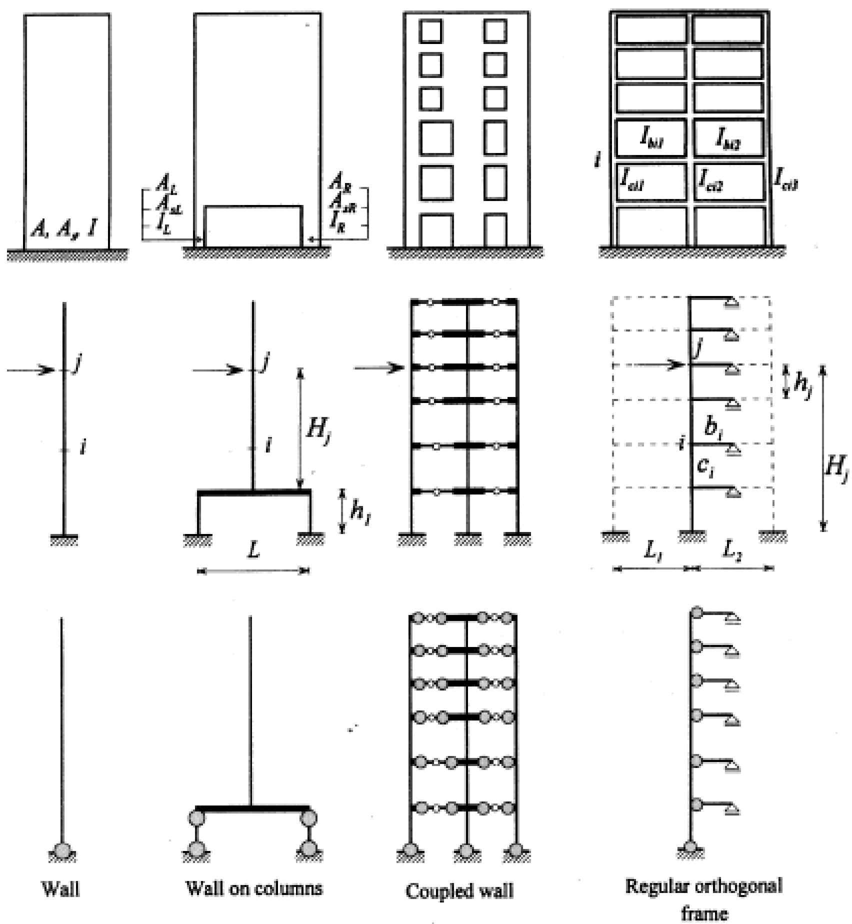


Figure 8.1. Standard macroelements, structural models for elastic analysis, and plastic mechanisms. For the frame, only the global mechanism is shown (from Kilar and Fajfar 1997).

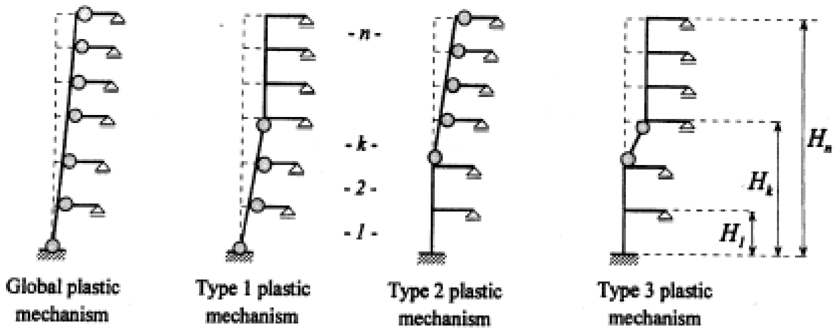


Figure 8.2. Anticipated plastic mechanisms for the frame (from Kilar and Fajfar 1997).

More recently, a slightly modified version of the NEAVEK program was used by my doctoral student Klemen Sinkovič, when he compared different procedures for the assessment of the seismic performance of low-rise reinforced concrete structures (Sinkovič et al. 2016). The simplified approach used in the NEAVEK program provided results which were very similar to those obtained by the more "accurate" analysis with standard building models. Of course, the simplified approach cannot be used for all structural systems.

9 INFILLED FRAMES

Reinforced concrete (RC) frames with masonry infill are a popular structural system in many parts of the world. Heavy damage and even the collapse of infilled RC frames, which has occurred during several earthquakes, calls for more attention for seismic behaviour of infilled frames.

Experience from earthquakes and test results suggest that "non-structural" masonry infills usually exhibit a strong influence on seismic response of frame structures. This influence may be positive or negative. Undesirable effects under seismic loading may occur mostly due to an irregular arrangement on infills in elevation or in plan. An irregular distribution of infills in elevation, typically an open first storey, results in the concentration of damage in this storey, typical for soft-story buildings. The irregularities introduced by the distribution of infills in plan induce torsion and can completely change the seismic response, e.g., from predominantly translational to predominantly torsional. A possibly adverse local effect due to the frame-infill-interaction (e.g., shear failure of columns under shear forces induced by the diagonal strut action of infills) can occur. If infills are not constructed along the entire height of a storey, e.g., in the case of parapet walls, a short-column effect can occur. Out-of-plane collapses of infilled walls often occur if they are not properly connected to the frame. In contrast, regularly distributed infills significantly reduce the deformation and ductility demand in structural elements. In several moderate earthquakes, such buildings have shown excellent performance even though many such buildings were not designed and detailed for earthquake forces. However, even in the case of regularly distributed infills, an undesirable story mechanism can be formed at the bottom of the building, which may represent a potential danger in the case of very strong ground motion, as, for example, observed in the 1999 Kocaeli (Turkey) earthquake, where a large number of multi-storey RC frame buildings with masonry infills collapsed.

In Fig. 9.1, two similar buildings with a uniform vertical distribution of infill, located within the same complex of buildings, are shown after the earthquake. In the building on the left-hand side, which did not collapse, a concentration of damage in the bottom two storeys can be clearly seen. The other building collapsed due to complete failure of the bottom two storeys.

In (Dolšek and Fajfar 2001), an attempt was made to explain why and when a soft storey effect may occur in uniformly infilled frames. The seismic response of structures designed according to EC8 was compared with that of structures with limited strength and ductility, typical for previous codes and for existing structures in many countries, including Turkey. It was demonstrated that, in the latter case, soft storeys can be created at the bottom of the building if the ground motion is

Figure 9.1. Multistorey buildings (RC frames with a uniform vertical distribution of masonry infill) after 1999 Kocaeli earthquake.

strong enough. The soft storey effect occurs even if the duration of ground motion is relatively short. In the former case (i.e., for code-designed buildings), there is also a tendency toward the formation of a soft first-storey mechanism. However, unless the ground motion is much stronger than expected, deformations remain within acceptable limits.

Possible beneficial and adverse effects of infills on the seismic structural response of infill frames can be accounted for only if infills are included in structural models. The analysis and design methods should adequately take into account the highly nonlinear behaviour of this system during strong earthquakes and yet be appropriate for practical applications.

In Ljubljana, much effort has been devoted to researching infilled frames. Considerable work has been performed at the Slovenian National Institute for Research in Materials and Structures (ZRMK, later ZAG), where my colleague Miha Tomaževič was employed. Miha was my schoolmate in secondary school, following which, we both studied civil engineering at the University of Ljubljana. He spent all his professional career at ZRMK and ZAG, working mostly in masonry structures. We have always had a very close relationship, although there was sometimes also some competition between our institutions. At our faculty, the seismic behaviour of infilled frames was the topic of several bachelor theses. A closer connection between the two groups was established when Roko Žarnić, initially employed at ZRMK, did his graduate study at the faculty and prepared his master's and doctoral theses on the seismic response of infilled reinforced concrete frames under my supervision. Miha Tomaževič was a co-advisor. Roko was not only an excellent researcher but also very capable in administration. After obtaining his doctorate in 1992, he became director-general of ZRMK. From this position, he moved back to the faculty. In 1995, he established the Chair for Testing of Materials and Structures and led it until his retirement from teaching in 2015. From 2010 to 2012, Roko took a brief excursion into politics as the Minister of Environment and Spatial Planning of Republic Slovenia.

Another colleague, working in the field of infilled frames, was Janez Reflak. In 1990, he defended his doctoral thesis entitled "Influence of infills on linear static and dynamic behaviour of frames". In the thesis, a structural model based on the finite element idealisation was proposed. Each infill was treated as a substructure and all degrees of freedom corresponding to the infill, with the exception of those in contact with the frame, were eliminated using the static condensation procedure. A paper was presented at the 6th Canadian Conference on Earthquake Engineering (Reflak and Fajfar 1991).

The first publication in which the N2 method was applied for the analysis of infilled frames appeared at the second Bled workshop (Fajfar et al. 1997). The work on infilled frames was performed by my master's student Dušica Drobnič. Infill panels were included in the structural model. They were modelled with equivalent diagonal struts, which carry load only in compression. Equivalent diagonals are the simplest option for modelling infills, which has been used in all our following analyses. Comparisons with test results have shown that equivalent diagonals are able to satisfactorily simulate the influence of the infills, provided that the main characteristics of the diagonal (stiffness and strength) are appropriately chosen. The basic N2 analysis was performed by using the structural model with infills. For the test structures (variants of a four-storey RC frame: bare frame, infilled frame and infilled frame with open first storey) it was possible to demonstrate a strong influence of infill on the structural response and to obtain reasonably close agreement with experimental results. However, these observations could not be generalised.

In order to correctly apply the N2 method to different infilled RC frames, two modifications of the basic version of the N2 method need to be made. (It is assumed that the infill fails before the frame.) First, the pushover curve has to be idealised as a multi-linear force-displacement relationship rather than a simple bilinear elasto-plastic one. Secondly, inelastic spectra have to be determined by using specific reduction factors (i.e., the R_μ-μ-T relation), appropriate for infilled frames. The extension of the applicability of the N2 method to infill frames was the topic of the doctoral thesis of Matjaž Dolšek (finalised in 2002) and of his work as a postdoc. The results were published in two papers in EESD (Dolšek and Fajfar 2004a, 2005). Later, two additional papers appeared in the journal Engineering Structures (Dolšek and Fajfar 2008a, 2008b).

The pushover curve of an infilled frame exhibits greater stiffness and strength than that of the corresponding bare frame. Its primary characteristic is a substantial decrease in strength after the infill has begun to degrade. This feature has to be taken into account in analyses. A typical idealised force-displacement envelope corresponding to an infilled RC frame is shown in Fig. 9.2. It can be divided into four parts. The first, equivalent elastic part, represents both the initial elastic behaviour and the behaviour after cracking has occurred in both the frame and the infill. The second part, between points P1 and P2, represents yielding. This part is typically short, due to the low ductility of infilled frames. In the third part, strength degradation of the infill governs the structural response until the point P3 is reached, where the infill fails completely. After this point, only the frame resists the horizontal loads.

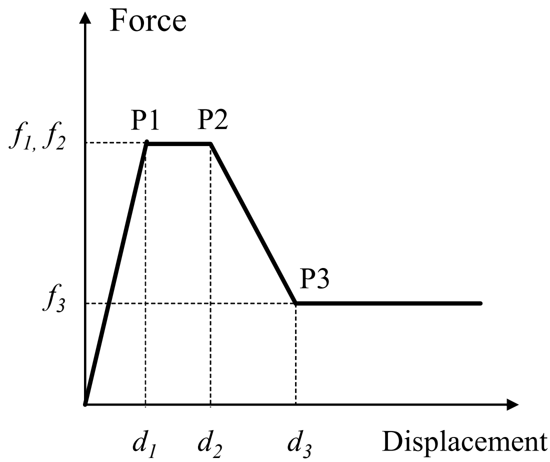


Figure 9.2. The idealised force-displacement relationship for an infilled RC frame.

Based on an extensive statistical study of an SDOF mathematical model with a four-linear backbone curve and hysteretic behaviour typical for infill frames, a specific R_μ-μ-T relation was determined (Dolšek and Fajfar 2004a). The structural parameters determining this relation, which are employed in addition to the parameters used in a usual elasto-plastic system (i.e., the initial period and global ductility), are ductility at the beginning of strength degradation $\mu_s = d_2/d_1$, and the reduction of strength after the failure of the infills $r_u = f_3/f_1$ (Fig. 9.2). R_μ also depends on the corner periods of the elastic demand spectrum (T_C and T_D according to EC8).

As an example, the R_μ-μ-T relations for a specific idealised system representing an infilled frame, are presented in Fig. 9.3. For comparison, the relations for a related elasto-plastic system without strength degradation are also shown.

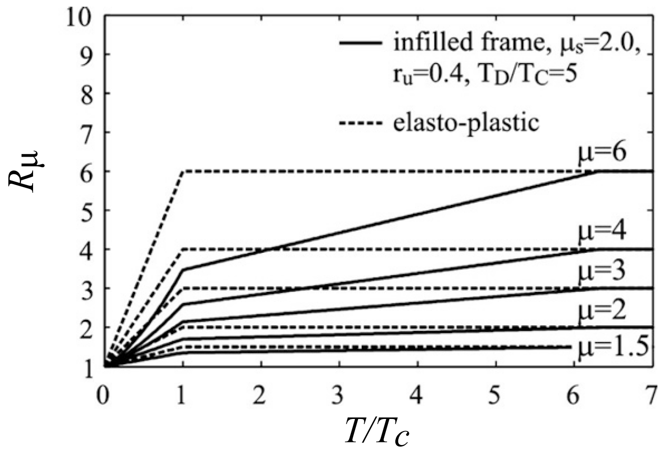


Figure 9.3. The R_μ - μ - *T* relations for an infilled RC frame and for an elasto-plastic system without strength degradation (from Dolšek and Fajfar 2008a).

An extension of the N2 method, similar to that for infilled frames, can be made to any structural system, provided that an appropriate specific R_μ-μ-T relation is available.

10 BRIDGES

Pushover-based procedures were initially developed for buildings. Later, attempts were made to use them for bridges and viaducts ("bridge" will be used in this chapter for both bridges and viaducts). This should be done with care, since structural systems of bridges and, consequently, their seismic responses differ significantly from those of buildings.

The research related to the application of the N2 method to bridges started in the mid-1990s in the doctoral thesis of Peter Gašperšič entitled "A method for seismic damage analysis of building and bridge structures". The results were published in (Fajfar et al. 1997) and (Fajfar and Gašperšič 1998). As in the case of buildings at that time, studies on bridges were restricted to two-dimensional (planar) problems. In the longitudinal direction, the determination of an SDOF model is usually straightforward. Consequently, we studied only the response in the transverse direction. Using the N2 method, we performed analyses of several bridges of different types and compared the results with the results of nonlinear response history analyses. We concluded that the N2 method was able to adequately predict the main seismic response parameters for regular bridge structures. In the case of irregular structures, the N2 method could, at least, detect the weak points in the structure.

Personally, I have not been involved in further research related to the use of the N2 method for bridges since our 1998 publication. One reason for this was that many different variants of the structural system can appear in the case of bridges. More often than in the case of buildings, several vibration modes are significant. Moreover, it can happen that the inelastic displacement shape is qualitatively different from the elastic one. For these reasons, for many bridges, the main assumptions of the basic version of the N2 method are violated. Another reason that contributed to my loss of interest in extending the N2 method to general bridges was the fact that a bridge is usually a simpler and "cleaner" structure than a building, whereas the investment in a bridge may be higher than in a building. Consequently, a nonlinear dynamic analysis may be in practice more feasible for a bridge than for a building.

Major work on bridges in our research group has been performed by Tatjana Isaković, a very intelligent, committed and tremendously hard-working person. She did her doctoral degree, with Matej Fischinger as the advisor, on the seismic design of reinforced concrete bridges in 1996. After completing her doctoral degree, she got a permanent position at the faculty and finally became the first female full professor in the history of the faculty. Presently (2020), Tatjana is the head of the institute and of the Chair for structural and earthquake engineering.

The work on bridges, performed by Tatjana Isaković, Matej Fischinger and co-authors, also included pushover-based methods (see Isaković et al., 2003, Isaković et al., 2008, Isaković and Fischinger, 2006, 2011). Inter alia, the limits of the applicability of the basic N2 method were examined. It was found that the applicability of the N2 method depends on (a) the ratio of the stiffness of the superstructure to that of the bents, and (b) the strength of the bents. The stiffer the superstructure and the more flexible the bents, the more regular the bridge is and less important its higher modes are. The value of the effective mass of the predominant mode could be used as a criterion of the efficiency of the N2 method. It was considered to be sufficiently accurate if the predominant mode did not significantly change with the changing of the intensity of the ground motion, and if this mode had an effective mass of at least 80% of the total mass. For short bridges with few columns, the accuracy of the N2 method increases with the seismic intensity. In long bridges (e.g., with a total length of 500 m or more), the stiffness of a typical superstructure is relatively low due to its great total length. Consequently, the higher modes significantly influence the response, regardless of the seismic intensity (or the strength of the bents). The N2 method is less accurate in the case of such structures.

In general, the scope of the applicability of the N2 method can be extended if two or three different distributions of the lateral forces are considered, and the envelope of the corresponding results is used, as in the case of buildings in EC8.

Isaković and Fischinger also studied pushover-based methods which consider the higher modes of vibration. Among them, they also applied the Extended N2 method, developed for buildings, in which the higher modes are taken into account by enveloping the results of the basic pushover analysis and the normalised results of the elastic modal analysis (see Chapter 12). The results were compared with results of a shaking table test of a single bridge. For this example, a considerable difference between the numerical and test results was observed (Isaković and Fischinger 2011).

11 CAPACITY

The primary research efforts of our research group were directed toward seismic demand, i.e., to one side of the capacity versus demand (in) equation. Nevertheless, we also did some work on the seismic capacity, which, generally, heavily relies on experimental data. We did not have our own experimental data, so we used the data available elsewhere.

For the analysis of data, we applied a multidimensional non-parametric regression method, called Conditional Average Estimate (CAE), which is used for the estimation of unknown quantities as a function of the known data. The first (unknown) and the second (known) set of parameters are called the output and input parameters, respectively. Unlike a pre-selected parametric model that could be too restrictive or too low in the number of input parameters to fit unexpected features, this non-parametric smoothing approach offers a flexible tool for analysing unknown regression relationships. This has proved to be a beneficial attribute of the method when studying problems for which many details of the physical background remain uncertain.

To determine unknown output parameters from known input parameters, a database containing sufficient well-distributed and reliable empirical data is needed. One particular observation that is included in the database can be described by a sample vector, whose components are the input and output parameters. The database consists of a finite set of such sample vectors. According to the CAE method, the unknown output parameters are determined in such a way that the computed vector, composed of the given and estimated data, is most consistent with the sample vectors in the database. The output parameters can be estimated by the formulae:

$$\hat{c}_k = \sum_{n=1}^{N} A_n C_{nk} \tag{11.1a}$$

$$A_n = \frac{a_n}{\sum_{i=1}^{N} a_i} \tag{11.1b}$$

$$a_n = \frac{1}{(2\pi)^{D/2}\; w_1 \,\dots\, w_D}\; \exp\left[-\sum_{i=1}^{D} \frac{(b_l - b_{nl})^2}{2{w_l}^2}\right] \tag{11.1c}$$

where $\hat{c}_k$ is the estimated value of the k-th output parameter, c_{nk} is the same output parameter corresponding to the nth vector in the database, N is the number of vectors in the database, b_{nl} is the lth input parameter of the nth vector in the database, b_l is the lth input parameter corresponding to the vector under consideration (with unknown output parameters), and D is the number of input parameters. The parameter w_l is the width of the Gaussian function, which is called the smoothness parameter (different values of w_l correspond to different input parameters). It determines how fast the influence of data in the sample space decreases with increasing distance from the point whose coordinates are determined by the input parameters of the prediction vector. Choosing the value of this parameter is based mainly on the analyst's judgement. This may be a weak point in routine practical applications.

The CAE method was developed by Igor Grabec, my colleague from the Faculty for Mechanical Engineering (Grabec and Sachse 1997). The method is applicable to any prediction problem if an appropriate database is available. We have used it for predicting seismic capacity and for ground motion predictions. The work was mostly performed by Iztok Peruš.

As the first application of the CAE method, we predicted the seismic capacity of RC structural walls. The paper was published in 1994, with Igor Grabec as a coauthor (Peruš et al. 1994). After some time, when Karmen Poljanšek joined our research team, three journal papers on the capacity of rectangular RC columns followed. They dealt with flexural capacity (Peruš et al. 2006), the force-drift envelope (Peruš and Fajfar 2007), and hysteretic energy dissipation capacity (Poljanšek et al. 2009). Karmen was a highly competent and dedicated doctoral student. After completing her doctoral degree, Karmen stayed a short time as a postdoc in our research group. Then she moved with her family to Ispra, Italy, where she is employed at the European Joint Research Center (JRC).

To illustrate the CAE approach, let us estimate the flexural deformation capacity of a rectangular RC column in terms of the ultimate member drift, sometimes called the drift ratio, which is equal to the chord rotation at the fixed end. The ultimate drift, representing the near collapse (NC) limit state, is commonly related to the displacement at a 20% drop below maximum strength (i.e., when the restoring force reaches 80% of its maximum value) in the force-displacement diagram. A database is needed, containing experimental data for different columns (data on ultimate drifts together with the corresponding characteristics of columns, which are related to input parameters). Let us choose four characteristics of the column as the input parameters: the axial load index, P^*, an index related to confinement, $\alpha\rho_s{}^*$, the concrete compressive strength, $f_c{}'$, and the shear span index, L^*. Using the selected input parameters, the output parameter, i.e., the ultimate drift for the investigated column, can be predicted with Eq.11.1.

In Fig. 11.1, the predicted isolines for the constant ultimate drift, presented as a function of two input parameters (P^* and $\alpha\rho_s{}^*$), are plotted for two different databases and using the formula provided in EC8. Constant values were assigned

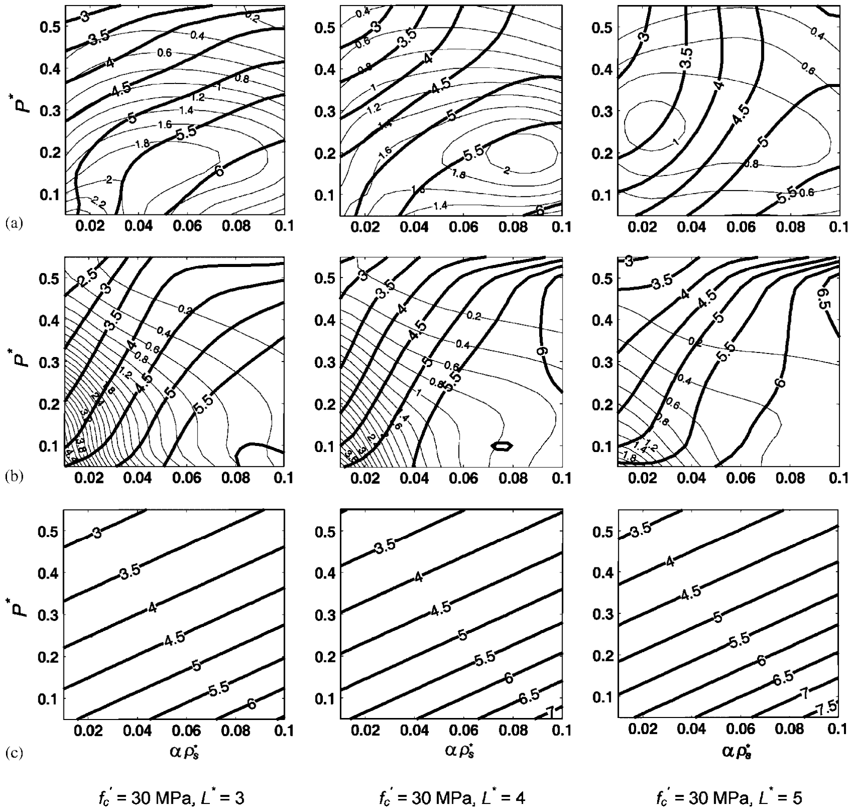


Figure 11.1. Ultimate drift determined by (a) CAE, using PEER database; (b) CAE, using the Fardis database; and (c) EC8 formula. The thin isolines are related to the reliability of results (from Peruš et al. 2006).

to the other two input parameters (f_c' and L^*). Thin lines represent the isolines of ρ, which is a measure that helps to detect the possible less accurate predictions due to the data distribution in the database and due to local extrapolation outside the data range. The higher the ρ value is, the more column specimens with input parameters similar to the input parameters of the investigated column exist in the database. The results presented in Figure 11.1 indicate the important influence of axial load and confinement on the ultimate drift. As expected, lower axial loads and better confinement increase the deformation capacity. There are some differences between the results obtained by two databases and by the EC8 formulae, which are based on the Fardis database. For more details, see (Peruš et al. 2006).

Frankly, our publications on capacity have not received much attention. One reason for this is undoubtedly the fact that we have provided a tool rather than final results that could be directly applied. In standards and codes, closed-form empirical formulae that enable an easy estimation of seismic capacities are typically given. The problem with these formulae is their dependency on the database used for their development. When new data become available, the formulae may become out-of-date. A typical example is that of ground motion prediction equations in which new formulae appear practically every year. In contrast, in the CAE approach, it is always possible to take into account the most recent version of the database. I am convinced that CAE is a simple and handy tool, at least in research.

12 EXTENDED N2 METHOD

12.1 Introduction

The main assumption in basic pushover-based methods is that the structure vibrates predominantly in a single mode. This assumption is sometimes not fulfilled, especially in high-rise buildings, where higher mode effects may be important along the height of the building, and/or in plan-asymmetric buildings, where substantial torsional influences can occur. In such cases, some corrections have to be applied to the basic procedure. In order to allow a more general use of pushover-based methods, several approaches have been proposed for taking into account the higher modes in elevation, torsional effects, or both influences. Unfortunately, many of the proposed approaches require quite complex analyses and are thus not appropriate for practical applications. The problem was excellently described by Baros and Anagnostopoulos (2008):

> The nonlinear static pushover analyses were introduced as simple methods … Refining them to a degree that may not be justified by their underlying assumptions and making them more complicated to apply than even the nonlinear response history analysis … is certainly not justified and defeats the purpose of using such procedures.

In our research, we have always had in mind the danger of developing a procedure which would not be accepted in practice. I hope that the Extended N2 method, which is the outcome of our research, is simple enough to be useful for practical applications. The extension is based on the assumptions that the structure remains in the elastic range in higher modes and that the amplification of deformations determined by elastic dynamic analysis can be used as a rough, mostly conservative estimate also in the inelastic range. In other words, it is assumed that the higher mode effects in the inelastic range are the same as in the elastic range, and that an estimate of the distribution of seismic demand throughout the structure can be obtained by enveloping the seismic demand in terms of the deformations obtained by the basic N2 (pushover-based) analysis, which neglects higher mode effects, and the normalised (the same roof displacement as in pushover analysis) results of elastic modal analysis, which includes higher mode effects. (Note that, in the Extended N2 method, torsion is considered as a "higher mode" effect, although, in a torsionally flexible structure, the first mode is torsional. A torsionally flexible structure is, by definition, a structure in which the predominantly torsional mode has a higher period than one (or both) predominantly translational mode(s)). Both methods (pushover and elastic modal analysis) are standard procedures in seismic analysis. Thus, the approach is conceptually simple, straightforward, and transparent. Of course, for widespread use in practice, the approach should be automated and provided as an option in computer codes. Since the approach has been implemented in the revised EC8, it is expected that some of the commercial programs will introduce the Extended N2 method in the near future.

The work on torsional influences on inelastic building structures started in our research group in the late 1990s. The N2 method presented in (Fajfar and Gašperšič 1996) was still limited to planar structures where torsion is not an issue. In the following years, first, a simplified pushover analysis was developed for a 3D building model (Kilar and Fajfar 1997). Later, the complete N2 method was extended to 3D structural models. Equations were adopted to such a model (Fajfar 2002), and the influence of the excitation in two horizontal directions was investigated. Several papers were published, mainly as conference proceeding. Initially, Vojko Kilar was involved in this research. The two researchers who continued the work on torsion were two other doctoral students of mine, Iztok Peruš and Damjan Marušić. Iztok finished his PhD in 1994 on a knowledge-based system for the assessment of seismic resistance of RC structures. After that, he stayed for an extended time in our research group as a postdoc, engaged in research on many different topics. Iztok was distinguished by his dedication, responsibility and modesty, and was a highly respected member of our research group. Later he became an Assistant Professor at the University of Maribor (the second Slovenian university with a civil engineering program). Damjan graduated from the Faculty of Architecture at the University of Ljubljana. It should be noted that at the University of Ljubljana, the Faculty of Civil Engineering and Geodesy covers both buildings and civil engineering structures, whereas at the Faculty of Architecture the emphasis is on the architectural design of buildings, while structural issues are in the background. Nevertheless, Damjan did an outstanding job studying the torsional response of multi-storey structures. After completing his PhD in 2001, he continued working for some years as a postdoc, before he moved to a state surveying company. Later, he founded a small company involved in the research, development, design, and production of furniture.

The results of the work on torsional influences were summarised in three journal papers. In (Peruš and Fajfar 2005) and (Marušić and Fajfar 2005) the findings of research on single-storey and multi-storey buildings, respectively, were presented. In (Fajfar et al. 2005) the main results were summarised, and an extension of the N2 method was proposed, in which the influence of torsion was taken into account by enveloping the results of basic pushover analysis and the results of standard elastic modal analysis.

The influence of higher modes along the height of the building was the topic of the doctoral thesis of Maja Kreslin. She was one of the very few female PhD students in our research group, very bright, well-organised and highly efficient, concentrated on important issues. She did an excellent job. Unfortunately, after finishing her doctorate, we could not offer her a permanent position at the faculty, so she moved to the Slovenian National Building and Civil Engineering Institute, where she works as a researcher in the field of bridge engineering.

In order to take into account the higher mode effects in elevation, the same idea as in the case of torsion was used: The seismic demand in terms of displacements and storey drifts can be obtained by enveloping the results of basic pushover analysis and the results of standard elastic modal analysis. The approach was presented in (Kreslin and Fajfar 2011).

After that, we combined two earlier approaches, taking into account both torsion and higher mode effects in elevation, into a single procedure, called Extended N2, enabling the analysis of plan-asymmetric medium- and high-rise buildings (Kreslin and Fajfar 2012). This method was implemented in the draft revised EC8 (see Section 12.5).

12.2 Torsion

Based on the results of our studies, we concluded that, in general, the inelastic torsional response is qualitatively similar to elastic torsional response and that, quantitatively, the torsional effects depend on the ductility demand and thus on the intensity of ground motion. This influence may be substantial. The amplifications determined by a usual linear dynamic (spectral) analysis represent an upper bound of the torsional amplifications in the majority of cases. The torsional effects generally decrease with increasing plastic deformations. This is manifested mainly in smaller amplification of displacements due to torsion on the flexible side (i.e., on the side that develops larger displacements in the case of static loading). For the opposite, stiff side, it was more difficult to make general conclusions. The response on the stiff side generally strongly depends on the effect of several modes of vibration and on the influence of the ground motion in the transverse direction. These influences depend on the structural and ground motion characteristics in both directions. If the building is torsionally stiff, usually a de-amplification occurs at the stiff side, which mostly decreases with increasing plastic deformations. In contrast, if the building is torsionally flexible, displacements at the stiff side are generally amplified. They usually decrease with increasing plastic deformations, although, in some cases, they may be larger in the case of inelastic behaviour than in the case of elastic response.

Considering the obtained results, the following conclusions relevant for the development of simplified analysis methods and code procedures were drawn:

1. The amplification of displacements determined by elastic dynamic analysis can be used as a rough, mostly conservative estimate also in the inelastic range.

2. Any favourable torsional effect on the stiff side, i.e., any reduction of displacements compared to the counterpart symmetric building, which may arise from elastic analysis, will probably decrease or may even disappear in the inelastic range.

These conclusions were used for the extension of the applicability of the N2 method to asymmetric building structures. In the Extended N2 analysis, independent standard pushover analyses in two horizontal directions are first performed. Displacement demand (amplitude and the distribution along the height) at the mass centres (CM) is determined using the basic N2 method. In order to take into account the torsional influence, a linear spectral modal analysis of the 3D mathematical model is required. This analysis is performed independently for excitations in two horizontal directions, where the results are combined according to the SRSS rule. The correction factors, to be applied to the relevant results of pushover analyses, are defined as the ratio between the normalised roof displacements obtained by elastic modal analysis and by pushover analysis. The normalised roof displacement is the roof displacement at an arbitrary location divided by the roof displacement at the CM. De-amplification due to torsion is not taken into account. Correction factors, which depend on the location in the plan, are defined for each horizontal direction separately. Finally, all relevant quantities obtained by pushover analyses are multiplied with appropriate correction factors. For example, in a perimeter frame parallel to the X-axis, all quantities are multiplied with the correction factor determined with pushover results obtained for loading in the X-direction and for the location of this frame. The relevant quantities are, for example, deformations for the ductile elements that are expected to yield, and the stresses for brittle elements that are expected to remain in the elastic range. For elements that yield, the upper limits for stresses and forces should be taken into account.

As an example, the torsional response of the original SPEAR (Test) building (Chapter 16) will be presented. The building is asymmetric. The eccentricities between the mass centres and approximate stiffness centres amount to about 10% and 14% in the X- and Y-directions, respectively. The first two modes are predominantly translational, whereas the third mode is predominantly torsional, so the building is torsionally stiff.

In a parametric study, nonlinear response history analyses (NRHA) were performed, using bi-directional ground motion, scaled to different intensities defined by peak ground accelerations of 0.05g, 0.1g, 0.2g, 0.3g, 0.5g, and 1.0g in order to obtain results from practically elastic to different levels of inelastic response. For peak ground accelerations PGA= 0.3g, 0.5g, and 1.0g, the global ductilities amount to about 2.8, 5.4, and 11.7 for the X-direction, and 2.5, 5.2, and 11.2 for the Y-direction, respectively.

Normalised roof displacements are presented in Figure 12.1. A decrease of torsional influences with increasing intensity of ground motion can clearly be seen. In the case of elastic behaviour, the torsional amplification on the flexible side and the torsional de-amplification on the stiff side are the largest. The Extended N2 method conservatively takes into account the elastic amplification, whereas the de-amplification is neglected. Note that there is a difference between the results of elastic response history analysis and modal spectral analysis. This difference, which is due to the approximation related to the SRSS combination of different modes in the modal analysis, has been widely accepted in practice. In the figure, it can also be seen that the standard pushover analysis completely fails in the prediction of torsional effects.

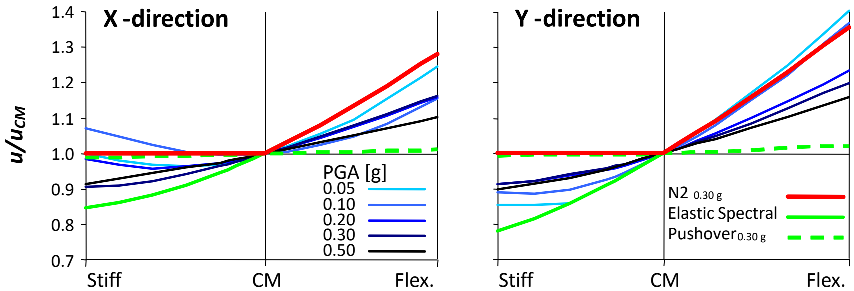


Figure 12.1. Comparison of normalised roof displacement in plan obtained by NRHA analysis (mean values) for different intensities, elastic response spectrum analysis, pushover analysis (for *PGA* = 0.3g), and the Extended N2 method (for *PGA* = 0.3g) for the SPEAR building (adapted from Fajfar et al. 2005).

Additional examples can be seen in references (Marušić and Fajfar 2005, and Fajfar et al. 2008) where torsionally flexible structures are included. As an example of the application of the N2 method for the seismic evaluation of a plan-asymmetric actual building, the irregular building structure of our faculty was analysed (Kreslin and Fajfar 2010).

12.3 Higher Modes in Elevation

For extending the applicability of the N2 method to medium- and high-rise buildings, where higher mode effects are important along the elevation of the structure, practically the same idea has been used as in the case of torsion. It is assumed that the structure remains in the elastic range when vibrating in higher modes and that the seismic demands can be estimated as an envelope of demands determined by a pushover analysis, which does not take into account the higher mode effects, and normalised demands determined by an elastic modal analysis, which includes higher mode effects. The latter results are normalised to the target roof displacement obtained in the basic N2 method. Usually, the pushover analysis controls the behaviour of those parts of the structure where the major plastic deformations occur, i.e., typically at the lower part of a building, whereas the elastic analysis determines seismic demand at those parts where the higher mode effects are important, i.e., typically in the upper part of the building. Due to the similarity of the approaches, basically the same procedure as in the case of torsion can be applied. The influence of higher modes is determined by standard elastic modal analysis and used for the adjustment of the results obtained by the basic pushover analysis.

An important difference between the torsion and higher mode effects in elevation should be noted. In the case of torsional influences, the displacement is the relevant demand parameter. In the case of higher modes in elevation, their influence on displacements is small and can be neglected in most practical applications. The relevant demand parameter is storey drift. In any case, the higher mode effect on displacements is neglected at the roof, i.e., it is assumed that the higher modes do not increase the target roof displacement determined in the basic N2 method.

As a test example, a nine-storey steel frame "Los Angeles building", which was investigated within the scope of the SAC steel project, performed in the United States after the 1994 Northridge earthquake, is used (for details on the structure, mathematical model, and ground motions, see Kreslin and Fajfar 2011). In our study, a planar mathematical model was used. The first three natural periods of the building are 2.27 s, 0.85 s, and 0.49 s.

For a nonlinear response history analysis (NRHA) two sets of ground motions, containing 44 and 20 accelerograms taken from two different strong-motion databases were used. The second set was used for investigating the influence of ground motion intensity. The normalised records were therefore scaled to four different intensities in order to obtain results from the elastic range to a high level of inelastic response. These intensities were defined by peak ground accelerations of PGA = 0.10g, 0.50g, 0.75g, and 1.00g. The structural response to PGA = 0.10g (intensity I1) is elastic. For three higher intensities, I2, I3 and I4, the global displacement ductilities amount to about 1.4, 1.9 and 2.5, respectively.

The influence of ground motion intensity can be seen in Fig.12.2, where the normalised storey drifts are shown for different ground motion intensities. The normalisation of the storey drifts was performed in such a way that the roof displacement d_t, which corresponds to the mean values of the roof displacements obtained by NRHA, is equal to 1.0 for each ground motion intensity. The storey drifts were therefore multiplied by a factor of $1/d_t$, which is different for different intensities. The results obtained by NRHA clearly show that the elastic results (intensity I1) represent the upper bound for the normalised storey drifts in the upper part of the building, and a lower bound in the storeys where major plastic deformations occur, mostly at the bottom of the building. With increasing intensities, the normalised storey drifts in the upper part of the building decrease, whereas they increase in the bottom storeys. These observations represented the basis for the proposed Extended N2 method. The predictions based on elastic analysis represent a conservative estimate in the upper part of the structure, whereas the pushover results control the predictions in the lower part of the structure.

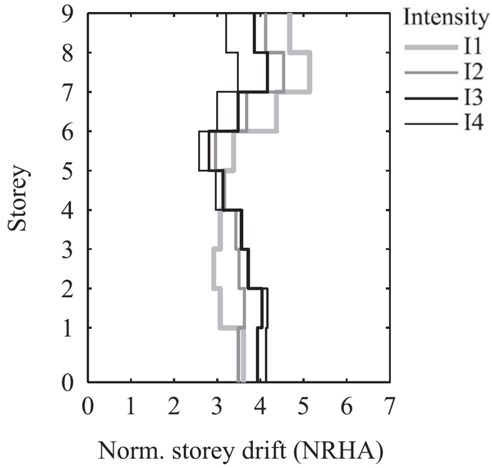


Figure 12.2. Influence of ground motion intensity on the normalised storey drifts obtained by NRHA (mean values) (adapted from Kreslin and Fajfar 2011).

A comparison of results in terms of displacements and storey drifts obtained by different procedures is shown in Figure 12.3. In the case of NHRA, where the first set of ground motions with 44 accelerograms scaled to intensity I3 was used, mean values, standard deviations, and envelopes are presented. A very large dispersion of results, typical for problems in earthquake engineering, can be observed.

In the case of the N2 method, there are two essential independent parts of the procedure, i.e., the determination of the target displacement and of the distribution of seismic demand. In order to eliminate the possible difference in the determination of target displacements and to enable the comparison of the higher mode effects determined by using different methods, the target displacement was set equal to the mean value of the 44 roof displacements, calculated by NRHAs. The results in Figure 12.3 show that the influence of higher modes on displacements is mostly small. In the case of storey drifts, however, the basic N2 method, which does not take into account the higher mode effects, grossly underestimates storey drifts in the top stories, whereas the normalised results of elastic analysis underestimate storey drifts in the lower part of the building. The results of the Extended N2 method represent the envelope of the results obtained by the basic N2 method and the usual elastic modal analysis scaled to the target roof displacement. The results fit very well to the mean results of the NRHA. Of course, such an excellent agreement cannot be expected in all cases.

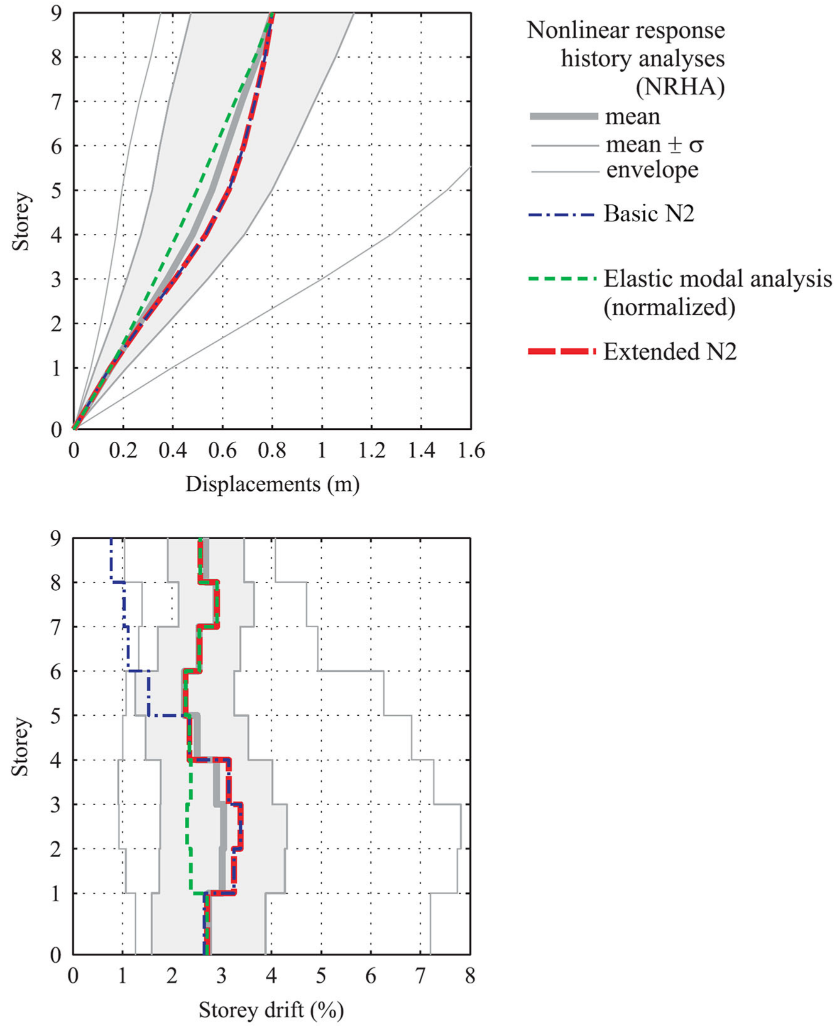


Figure 12.3. The displacements and storey drifts (%) for the nine-storey LA building, obtained by different analyses (adapted from Kreslin and Fajfar 2011).

The US standard for existing buildings ASCE 41-13 (ASCE 2014), basically uses the same idea of enveloping the results of the two analysis procedures in order to take into account the higher modes in elevation. In C7.3.2.1 it is stated "Where the NSP [Nonlinear Static Procedure] is used on a structure that has significant higher mode response, the LDP [Linear Dynamic Procedure, typically the modal response spectrum analysis] is also used to verify the adequacy of the evaluation or retrofit." The same recommendation is included in recent New Zealand guidelines (NZSEE 2017).

12.4 Torsion and Higher Modes in Elevation

The procedures for taking into account torsion and higher mode effects in elevation are consistent and compatible. Both effects can be simultaneously considered, as shown in (Kreslin and Fajfar 2012).

In order to predict the structural response for a building with a non-negligible effect of torsion and higher modes in elevation, the following procedure can be applied:

1. Perform the basic N2 analysis. In the case of a plan-asymmetric building, either two 2-D (planar) models can be used, one for each horizontal direction, or a 3-D structural model. Loading is applied at the mass centres (CM), independently in each of the two horizontal directions, in each direction with the + and - sign. The target displacement (the displacement demand at the CM at roof level) is determined for each of the two horizontal directions (the larger value of the two values, obtained for the + and – sign). It is assumed that the effect of higher modes on the target roof (top) displacement is negligible.

2. Perform the standard elastic modal analysis (Chopra 2017) of the 3-D structural model independently for excitation in two horizontal directions, considering all the relevant modes (using, e.g., the SSRS or the CQC rule), and combine the results for both directions according to the SRSS rule. Determine the displacements and storey drifts at the CM for each storey. Determine the roof displacements for each frame or wall in the plan. Normalise the results in such a way that the roof displacement at the CM is equal to the target displacement.

3. Determine the seismic demand by using the results obtained in steps 1 and 2. This can be achieved by applying two sets of correction factors, one for displacements (in plan) and the other for storey drifts (along the elevation). The set determined for displacements also applies to the storey drifts. So, the resulting correction factor for the storey drift in a particular storey, and at a particular position in the plan, is obtained as a product of two correction factors. The correction factors are defined for each horizontal direction separately. They are applied to the relevant results of the pushover analyses.

3.a. The correction factor for displacements due to torsion is defined as the ratio between the normalised roof displacements obtained by elastic modal analysis (step 2) and by pushover analysis (step 1). The normalised roof displacement is the roof displacement at an arbitrary location divided by the roof displacement at the CM. If the normalised roof displacement obtained by elastic modal analysis is less than 1.0, then the value 1.0 is used, i.e., no de-amplification due to torsion is taken into account. These correction factors depend on the location in the plan.

3.b. The correction factor for storey drifts due to higher mode effects in elevation is defined as the ratio between the normalised storey drifts obtained by elastic modal analysis (step 2) and the results obtained by pushover analysis (step 1). As in the case of torsion, no de-amplification is taken into account, i.e., if the ratio is less than 1.0, the value 1.0 is used. One correction factor is determined for each storey in the two horizontal directions.

The resulting correction factors for storey drifts (obtained as a product of two correction factors as described above) apply to all local deformation quantities (e.g., total joint rotations consisting of both elastic and plastic parts). They also apply to the internal forces, provided that the resulting internal forces do not exceed the load-bearing capacity of the structural member. If the capacity is exceeded, internal forces can be estimated from the deformations by taking into account the relevant force-deformation relationship.

In the case of a planar (2-D) structural model, the results obtained with the Extended N2 method represent an envelope of the results obtained by two analyses. In the case of a plan-asymmetric (3-D) model, the seismic demand at different locations at the roof and at the mass centres along the elevation, determined according to the proposed procedure, represents such an envelope. At other locations, the results are mostly close to the envelope. Thus, conceptually, the Extended N2 method can be explained as a procedure enveloping the results of two standard methods: the basic N2 method and the elastic modal spectral analysis. The target roof displacement in the mass centre is determined using the basic N2 method, whereas the results of the elastic analysis are normalised to the target roof displacement from the basic N2 analysis. The internal forces are limited by the upper bound imposed by the force-deformation relationship.

The two essential parts of the proposed procedure, i.e., the determination of the target displacement and of the distribution of seismic demand, are not coupled and are performed independently. For this reason, the procedure which is used in the Extended N2 method for the distribution of seismic demand can be applied together with any procedure for the determination of the target displacement.

A test example, showing the effects of both torsion and higher modes along the elevation, is presented in (Kreslin and Fajfar 2012).

12.5 Extended N2 Method in Eurocode 8

The basic version of the N2 method has been implemented in EC8. When Part 1 of EC8 was finalised, the extended version of the N2 method for plan-asymmetric buildings had not been fully developed yet. Nevertheless, based on the preliminary results, a clause on torsion was added in line with the procedure used in the Extended N2 (See Section 4.4). In EC8, Part 3 (CEN 2005), a requirement related to higher mode effects in elevation is provided.

The Extended N2 method has been implemented in the draft revised version of EC8 Part 1-2 (CEN 2019c). The relevant part of the standard was prepared by a project team set by CEN TC 250, where Matjaž Dolšek was in charge of drafting the clauses related to pushover analysis.

According to the revised EC8, it is required that the results obtained by the pushover-based (N2) analysis at different locations in the building be multiplied by the correction factors. Displacements are multiplied by $c_{P,j}$, whereas other seismic action effects, such as (generalised) deformation and (generalised) stresses, are multiplied by the product of correction factors $c_{P,j}$ and $c_{E,i}$. The correction factors $c_{P,j}$ and $c_{E,i}$ account, respectively, for the torsional effects and higher mode effects in elevation. The values of $c_{P,j}$ vary in plan (j is the index denoting the location of the structural member in plan), while the values of $c_{E,i}$ vary in elevation of the building (i is the index denoting the storey of the structural member). The effect of higher modes along the elevation on the displacements is neglected. If some specific criteria are fulfilled, $c_{P,j}$ and/or $c_{E,i}$ are equal to 1.0, i.e., it is not required to take into account the torsional effects and/or higher mode effects in elevation.

The values of the corrections factors $c_{P,j}$ and $c_{E,i}$ should be calculated as the ratio between normalised deformations obtained from the linear elastic analysis and the pushover-based (N2) analysis. The normalised deformations from linear elastic analysis should be calculated by the response spectrum method with consideration of the effects of torsion and the effects of the combination of horizontal components of the seismic action. If conditions for the use of the lateral force method are met, it may be used for the calculation of the correction factors. For each structural member, the correction factors $c_{P,j}$ and $c_{E,i}$, should be calculated for each direction of lateral forces for pushover analysis:

$$c_{P,j} = \frac{d_{et,j}}{d_{t,j}} \frac{d_t}{d_{et}} \geq 1.0 \tag{12.1}$$

$$c_{E,i} = \frac{d_{ret,i}}{d_{rt,i}} \frac{d_t}{d_{et}} \geq 1.0 \tag{12.2}$$

d_t and $d_{t,j}$ are the target displacement, i.e., the displacement of the control node (control displacement, typically displacement at the centre of mass of the slab at the top of the building) determined by the pushover-based (N2) analysis associated with the considered limit state, and the corresponding displacement at location j in plan, respectively,

d_{et} and $d_{et,j}$ are the values of the control displacement from the linear elastic analysis for the design seismic action, and the corresponding displacement at location j in the plan, respectively,

$d_{rt,i}$ is the interstorey drift at the centre of mass of the i-th storey corresponding to d_t,

$d_{ret,i}$ is the interstorey drift at the centre of mass of the i-th storey from the linear elastic analysis for the design seismic action.

13 INCREMENTAL N2 METHOD (IN2)

At the beginning of the new millennium, Incremental Dynamic Analysis (IDA) (Vamvatsikos and Cornell 2002) became a very popular tool for studying structural behaviour under different levels of seismic intensity. IDA is a parametric analysis method for the estimation of structural response under seismic loads. A structural model is subjected to multiple levels of seismic intensity using one or more ground motion records. The objective of an IDA study is the understanding of structural behaviour under different levels of seismic intensity. IDA is also a substantial part of a probabilistic framework for seismic performance assessment, developed at Stanford University (Cornell and Krawinkler 2000, Cornell et al. 2002). The result of IDA is an IDA curve, which represents the relation between a structural response parameter and the intensity level of ground motion, and the corresponding variability.

IDA requires great computational effort. The question arises whether it is possible to determine IDA curves with fewer input data and with less effort but still with acceptable accuracy. An approximation of the IDA curve can be obtained with the N2 analysis. We called this approach the "Incremental N2 (IN2) method". A point of an IN2 curve (approximate IDA curve), which corresponds to a given seismic intensity, is determined with the N2 method, in contrast to an IDA curve, for which each point is determined with nonlinear dynamic analysis. IN2 was presented for the first time in 2004 at the 13th WCEE (Dolšek and Fajfar 2004b) and has been thereafter often used in our research (see Chapter 14).

In order to determine an IN2 (or IDA) curve, first, the ground motion intensity measure and the demand measure have to be selected. The most appropriate pair of quantities is the spectral acceleration and the roof (top) displacement, which also allow the visualisation of the procedure in the acceleration-displacement (AD) format. In such a case, each point of the IN2 (or IDA) curve is defined with the following pair: elastic spectral acceleration, corresponding to the equivalent elastic period T, on the Y-axis, and the corresponding inelastic displacement demand on the X-axis. Other relevant quantities, such as maximum story drift, rotation at the column and beam end, shear force in a structural element and in a joint, and story acceleration, can be employed as secondary demand measures. They are related to roof displacement and can be uniquely determined if roof displacement is known. The secondary demand measures can be used, together with the main demand measure, for performance assessment at different performance levels.

The shape of the IN2 curve depends on the inelastic spectra applied in the N2 method, which are based on the relation between strength reduction factor, ductility and period (the R_μ-μ-T relation). The simplest IN2 curve is applicable to a structure for which the equal displacement rule applies. In such a case (Figure 13.1), the IN2 curve is bilinear and is defined by a single point representing the capacity of the structure in terms of displacements d_u (ultimate displacement) and corresponding elastic spectral acceleration A_u (see Section 4.5). The IN2 curve is linear from the origin to this point. It is assumed that the failure occurs at this point, and the IN2 curve becomes horizontal. For any ground motion intensity, a point on the bilinear IN2 curve applies. For example, for design ground motion intensity presented by elastic design spectrum, the point defined by D_e and A_e applies.

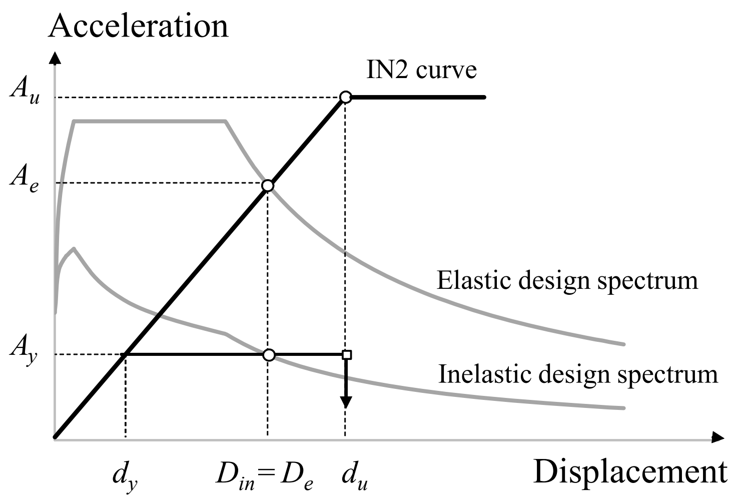


Figure 13.1. IN2 curve (for equivalent SDOF system) for the simplest case (equal displacement rule, bilinear capacity diagram, failure at ultimate displacement d_u).

The main application of IDA is in a probabilistic framework, in which the annual likelihood of the event that the demand exceeds the limit-state or capacity is estimated. A single-record IDA study cannot fully capture the behaviour of a structure in a future event. IDA curves can be highly dependent on the ground motion records chosen, so a sufficient number of records is needed to cover the full range of responses. The suite of IDA curves can be summarised, for example in 16%, 50%, and 84% IDA curves.

In the case of an IN2 analysis, the spectra, used in the N2 method, usually represent mean spectra and thus the IN2 curve represents a mean curve. Information on dispersion is not available from the IN2 analysis. If the IN2 curve is to be used in a probabilistic seismic assessment method, it has to be combined with predetermined generic variability for different structural systems, as discussed in Section 14.2.

A comparison of the IN2 and IDA curves for the original SPEAR building (Chapter 16) is shown in Figure 13.2. IDA curves were determined using a MDOF model and an equivalent SDOF model. Median, 16%, and 84% fractiles are plotted. Conservatively, it was assumed that the NC limit state corresponds to the collapse of the building.

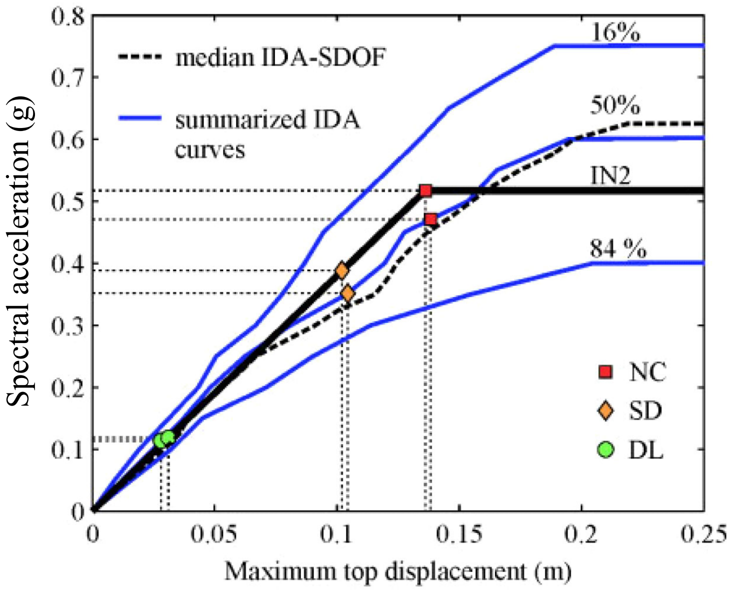


Figure 13.2. Comparison of the summarised IDA curves with the IN2 curve for the SPEAR (Test) building. Markers indicate limit states of damage limitation (DL), of significant damage (SD), and near collapse limit state (NC) (from Dolšek and Fajfar 2007).

14 RISK ASSESSMENT

14.1 Introduction

The seismic response of structures is characterised by large uncertainties, especially with respect to the ground motion, but also in the structural modelling, so that, in principle, a probabilistic approach would be appropriate for seismic performance assessment. However, an average engineer is not familiar with probabilistic methods and is very hesitant to use them. Also, a large part of the research community is sceptical about explicit probabilistic approaches other than those used in seismic hazard analysis. For these reasons, the analysis of structures is typically performed with deterministic analysis, using the ground motion parameters corresponding to a prescribed return period of the ground motion. In this analysis, the uncertainties are implicitly taken into account by means of various safety factors. An explicit probabilistic approach, which allows for the explicit quantification of the probability of exceedance of different limit states, has not yet been implemented in building seismic codes, with the exception of the ASCE-7 standard (ASCE 2017). When using current seismic codes, "at the end of the design process there is no way of evaluating the reliability actually achieved. One can only safely state that the adoption of all the prescribed design and detailing rules should lead to rates of failure substantially lower than the rates of exceedance of the design action" (*fib* 2012, p.3).

In the long term, it will be difficult to completely avoid quantitative determination of risk. Also due to the public pressure on loss minimisation in addition to life safety in most developed countries with high seismicity, the profession will sooner or later be forced to accept some kind of risk-based design and assessment, at least for a better calibration of different safety factors and force reduction factors used in codes. Information on seismic risk would also facilitate discussions of design options between designers, building owners, and other stakeholders. However, the mandatory use of explicit probabilistic approaches in seismic building codes, if it will ever happen, is still very distant. The prerequisites for possible implementation of quantitative risk assessment in the codes are reasonably reliable input data and highly simplified procedures, which are presented in a format that is familiar to engineers, and which require only a small amount of additional effort and competence. Inclusion of optional reliability-based material in the seismic codes would help due to its educational role. In Europe, the very first step has been very recently already taken with an informative annex to draft revised EC8, Part 1-1 (CEN 2019a), entitled "Simplified reliability-based verification format" (CEN 2019b), which has been drafted by Matjaž Dolšek and his doctoral students. It provides a basis for a simplified verification of the performance of a structure in probabilistic terms.

Our research on probabilistic approaches started when Matjaž Dolšek did some work on the evaluation of the probability of exceedance of a damaged state of a building during its expected lifetime, as a part of his doctoral thesis on seismic response of infilled RC frames. Work in Ljubljana was strongly influenced by research performed in Stanford by Allin Cornell and his PhD students. Our official cooperation with universities in Stanford and Berkeley, as well as the special relationship we have had with Helmut Krawinkler, enabled the visits of some members of our research team to Stanford University almost every year. During these visits, there was always a meeting with Professors Krawinkler and Cornell and their students. So, we were kept informed about the recent state-of-the-art in the place where the comprehensive probabilistic framework for seismic performance assessment, known as the PEER probabilistic framework, was born as a result of the close cooperation of a top expert in probabilistics and a top engineer (Cornell and Krawinkler 2000).

Impressed by the engineering approach to probabilistic problems, Matjaž Dolšek became the driving force for research on probabilistic approaches in our research group. He completed his doctoral degree in 2002, continued with research as a postdoc, and quickly advanced to the rank of full professor. With his doctoral students, he did excellent work and published several well-cited papers. Since mine and Fischinger's retirement, he has been teaching the courses related to structural dynamics and earthquake engineering and leads the research group on earthquake engineering. In 2019, he received a national award for his research achievements. Matjaž is a very intelligent, committed, independent, hardworking and ambitious person. In my opinion, he is presently one of the leading researchers in seismic risk analysis worldwide.

Our first journal paper on risk assessment, entitled "Simplified probabilistic seismic performance assessment of plan-asymmetric buildings", was published in EESD (Dolšek and Fajfar 2007). In this paper, a relatively simple approach for the probabilistic seismic performance assessment of structures, which we later called the Pushover-based Risk Assessment Method (PRA – see the next section) was proposed. The approach was applied to the SPEAR building (see Chapter 16). The building is plan-asymmetric, and this was the reason that we put "plan-asymmetric" in the title of the paper. As I look it now, it was an unfortunate decision, since it most probably distracted the attention from the main contribution of the paper, i.e., the considerable simplification of the probabilistic procedure by replacing the IDA with the IN2 analysis. Later, after some further simplifications, we published another paper describing the PRA method, entitled "A practice-oriented estimation of the failure probability of building structures" (Fajfar and Dolšek 2012). The missing part for the applicability of the approach were the default values for the dispersion parameter β. Research on this parameter was done later by my doctoral student Mirko Kosič (see the next section), with the cooperation of Matjaž Dolšek, who was the co-advisor.

14.2 Pushover-based Risk Assessment Method (PRA)

The Pushover-based Risk Assessment Method (PRA) is a simple approach for estimating the annual probability of the failure of a structure. It represents a combination of Cornell's closed-form solution (Cornell 1996) and the N2 method, which is used for the determination of the capacity of the structure. Provided that predetermined default values for dispersions are available, PRA requires only a very minor effort in addition to a standard pushover-based analysis. For more details, see (Dolšek and Fajfar 2007, Fajfar and Dolšek 2012, Kosič et al. 2017). Compared to Cornell's original approach, in the PRA method, a large number of nonlinear response history analyses required for the determination of IDA curves is replaced by a pushover analysis needed for the determination of the IN2 curve (see Chapter 13). Of course, like other simplified methods, the PRA method has limitations, which are basically the same as those that apply to Cornell's closed-form solution and to the basic N2 method.

The "failure" probability of building structures, P_{NC}, i.e., the annual probability of exceeding the near-collapse limit state (NC), which is assumed to be related to a complete economic failure of a structure, can be estimated (Cornell 1996, Fajfar and Dolšek 2012, Kosič et al. 2017) as:

$$P_{NC} = \exp[0.5k^2\beta_{NC}^2]\,H(A_{NC}) = \exp[0.5k^2\beta_{NC}^2]\,k_0 A_{NC}^{-k} \quad (14.1)$$

Spectral acceleration at the fundamental period of the structure, A, is used as the ground motion intensity measure. However, the equation can also be used for other ground motion intensity measures, e.g., peak ground acceleration PGA. A_{NC} is the median NC limit state spectral acceleration at the fundamental period of the structure (i.e., the capacity at failure), and β_{NC} is the dispersion measure, expressed as the standard deviation of the natural logarithm of A_{NC} due to record-to-record variability and modelling uncertainty. The parameters k and k_0 are related to the hazard curve $H(A)$ which is assumed to be linear in the logarithmic domain:

$$H(A) = k_0 A^{-k} \quad (14.2)$$

A seismic hazard curve shows the annual rate or probability at which a specific ground motion level will be exceeded at the site of interest. The reciprocal of the annual probability of exceedance of a specific ground motion level is the return period $T_R = 1/H$.

The capacity at failure A_{NC} is estimated using the N2 method (see Section 4.5), whereas predetermined dispersion values are used for β_{NC}. Note that, in principle, Eq. 14.1 can be applied for any limit state provided that the median value and the dispersion of the selected intensity measure are related to the selected limit state. The NC (near collapse) limit state was selected as representative for failure instead of the C (collapse) limit state, since it is much easier to estimate capacities for the NC limit state than for the C limit state. It should be noted, however, that the tolerable probabilities of exceedance are higher for the NC than for the C limit state.

According to Eq. 14.1, the failure probability is equal to the hazard curve evaluated at the median capacity A_{NC}, multiplied by an exponential magnification factor, which depends on the product of the variability of the A_{NC}, expressed by β_{NC}, and the slope (in log-log terms) k of the hazard curve. For frequently used values ($\beta_{NC} = 0.5$ and $k = 3.0$), the correction factor amounts to 3.1. In such a case, the probability of failure is about three times larger than the probability of the exceedance of the ground motion corresponding to the median capacity at failure in terms of the spectral acceleration A_{NC}. If there were no variability ($\beta_{NC} = 0$), both probabilities would be equal.

Several options are available for the estimation of the parameters k and k_0. The best k and k_0 estimates can be obtained by fitting the actual hazard curve by a linear function in the logarithmic domain. In the absence of an appropriate hazard curve, k can be estimated from seismic hazard maps for two return periods. If hazard maps for two different return periods are not available, the only (very approximate) option is to assume a value of k depending on the geographical location of the structure. Appropriate values of k are usually within the range from 1 to 3 (exceptionally to 4). If the value of k, specific for the region, is not known, a value of k=3.0 has often been used as an option in high seismicity regions. In low seismicity regions, the k values are usually smaller. Note that k also depends on the intensity measure used in Eq. (14.1). In the case of the spectral acceleration A, it depends on the period of the structure. This dependence should be taken into account when a more accurate analysis is being sought (Dolšek et al. 2017).

For the determination of the parameter k_0, at least one value of A, corresponding to a specific return period, needs to be known for the location under consideration, e.g., A_d that corresponds to the return period of the design ground motion $T_d = 1/H(A_d)$, and represents the spectral acceleration in the elastic design spectrum. A_d can be typically obtained from the hazard map and the prescribed shape of the elastic acceleration ground motion spectrum. Knowing the value A_d and the corresponding return period T_d, the parameter k_0 can be obtained from Eq. 14.2, as follows:

$$k_0 = \frac{A_d^k}{T_d} \tag{14.3}$$

Considering Eq. 14.3, Eq. 14.1 can be written in the form

$$P_{NC} = \exp[0.5k^2\beta_{NC}^2]\frac{1}{T_d}\left(\frac{A_d}{A_{NC}}\right)^k \tag{14.4}$$

The determination of typical dispersion values β_{NC} of the capacity at failure for RC building structures using spectral acceleration at the fundamental period of the structure, A, as the intensity measure, was the main topic of the research work of my doctoral student Mirko Kosič. After defending his thesis in 2014, Mirko became a postdoc at our institute. The main findings of his work were published in (Kosič et al. 2014, 2016). The results of these studies showed that the values depend on the structural system and on the period of the structure T. In a simplified approach, it may be reasonable to assume $\beta_{NC} = 0.45$ or $\beta_{NC} = 0.5$ as an appropriate estimate for RC frame structures. This value takes into account both aleatory (mostly related to ground motion) and epistemic (mostly related to structural modelling) uncertainty. According to the annex (CEN 2019b) to draft revised EC8, for all structures and materials $\beta_{NC} = 0.6$ can be used, with the exception of very stiff structures, for which β_{NC} is reduced up to 0.4.

For design purposes, Eq. 14.4 has to be inverted in order to express the ratio between the spectral acceleration at failure A_{NC} and the design spectral acceleration A_d (both values are related to the elastic spectrum) as a function of the return period corresponding to the target probability of failure $T_{NC} = 1/P_{NC}$ and the parameters k and β_{NC}

$$\frac{A_{NC}}{A_d} = \exp[0.5k\beta_{NC}^2]\left(\frac{T_{NC}}{T_d}\right)^{\frac{1}{k}} \tag{14.5}$$

The ratio of accelerations (Eq. 14.5) is the product of two factors. The first factor takes into account the uncertainties both in the ground motion (record to record dispersion) and in the modelling. The value of this term is 1.0 if $\beta_{NC} = 0$, i.e., if there is no uncertainty. The second factor ($(T_{NC}/T_d)^{1/k}$) takes into account the fact that the target probability of failure (the NC limit state) is smaller than the probability of the design ground motion or, expressed in terms of return periods, the target return period of failure is larger than the return period of design ground motion. The value of this factor is equal to 1.0 if $T_{NC} = T_d$.

Eq. 14.5 can be used for the determination of the force reduction factor R (Chapter 3), as proposed by Dolšek and his doctoral students (Dolšek et al. 2017, Žižmond and Dolšek 2017, 2019). They introduced and elaborated the "risk-targeted" safety factor γ, which is equal to the ratio of accelerations A_{NC}/A_d (Eq. 14.5), and the "risk-targeted" force reduction factor, which, by using the safety factor γ, explicitly takes into account uncertainty and the difference between the ground motions related to design and NC limit state.

14.3 Tolerable Probability of Failure

The estimated probability of failure has to be compared with the tolerable (also called acceptable or permissible or target) probability of failure, which has not yet been clearly defined in the earthquake engineering community. When determining the tolerable probability of failure, the possible consequences in terms of risk to life and injury, the potential economic losses, and the likely social and environmental effects need to be taken into account. The choice of the target level of reliability should also take into account the amount of expense and effort required to reduce the risk of failure. The tolerable risk is, of course, a reflection of personal and societal value judgements, as well as disaster-based experience, and differs from one cultural environment to another. It is, therefore, no wonder that generally accepted quantitative values for target structural reliability, which could be used in seismic design, do not exist. Currently, the most popular value for the tolerable annual probability of failure of common buildings is $P_f = 2 \cdot 10^{-4}$ (1% in 50 years), also suggested in the annex (CEN 2019b) to the draft revised EC8 and confirmed in a discussion among European code developers. This value is comparable to the probabilities of failure estimated for buildings compliant with current seismic codes. It seems, however, that both the practising engineers and the general public expect a higher degree of safety, as described below.

An internet-based survey was conducted in 2013 to gather data about the perception of seismic risk in Slovenia. Respondents were differentiated according to their expertise in the field of project design and building construction. The first group of respondents were members of the Slovenian Chamber of Engineers (denoted as "experts"). Their answers were compared to the answers of the lay public sample, which was located using snowball sampling. It should be noted that the sample of lay people was not representative of all inhabitants of Slovenia and was, to a large extent, limited to people with higher levels of education. The results of the survey did not show significant differences between the two samples regarding the tolerable probability of collapse of buildings built according to the current seismic regulations. Both groups were asked how many buildings, on average, can be tolerated to collapse as a direct consequence of an earthquake during their expected working life (i.e., 50 years). Moreover, respondents were asked about the tolerable probability of economic failure (i.e., the building does not collapse physically, but a repair is not economically justified, corresponding to the NC limit state). A significantly higher tolerable probability than in the case of building collapse was expected. However, surprisingly, there was only a moderate difference between the tolerable probability of collapse and the tolerable probability of economic failure. For both groups of respondents, a large scatter of results was observed. The mean values of the tolerable probabilities of collapse and economic failure in a working life of 50 years are presented in Table 14.1. The results suggest that both experts and laypeople expect, on average, a lower (for at least an order of magnitude) probability of failure than that which is suggested in draft revised EC8 and that which has been estimated by different researchers for buildings complying with the current seismic regulations. More details of the survey are available in (Fajfar et al. 2014).

Table 14.1 Mean values of the tolerable probabilities of collapse and economic failure in 50 years for ordinary buildings, built according to the current seismic regulations.

	Prob. of collapse	Prob. of econ. failure
Experts	$1/1780 = 5.62 \cdot 10^{-4}$	$1/1000 = 10.0 \cdot 10^{-4}$
Lay people	$1/1740 = 5.75 \cdot 10^{-4}$	$1/1320 = 7.58 \cdot 10^{-4}$

14.4 Application of the PRA Method

The results of extensive studies have demonstrated that the PRA method has the potential to estimate the seismic risk of low- to medium-rise building structures; therefore, it could become a practical tool for engineers. Typical values of probabilities of exceedance of the NC limit state in a life-span of 50 years are, in the case of buildings designed according to modern codes, about 1%. In the case of older buildings not designed for seismic resistance, the probabilities are usually at least one order of magnitude higher (see, e.g., Kosič et al. 2014, 2016). It should be noted, however, that the absolute values of the estimated failure probability are highly sensitive to the input data and simplifying assumptions, especially those related to the seismic hazard. Comparisons between different structures are more reliable. Comparative probabilistic analyses can provide valuable additional data necessary for decision-making. Due to its simplicity, the PRA method can also serve as a tool for the introduction of explicit probabilistic considerations into structural engineering practice.

As an example, the annual probability of failure will be estimated for two variants of the SPEAR building: Test and EC8-H (see Chapter 16). The test structure was built without consideration of seismic codes, whereas EC8-H structure complies with EC8. It is assumed that both variants of the building are located at the same location with seismic hazard defined with design peak ground acceleration (for return period T_d = 475 years) $PGA_d = PGA_{475}$ = 0.29g and k = 3. The EC8 spectrum for soil type C applies. The corresponding design spectral accelerations are A_d = 0.46g and A_d =0.71g for the Test and EC8 building, respectively. For both cases, β_{NC} = 0.45 is assumed. The data related to the pushover analysis are taken from Chapter 16. The annual probabilities of failure (NC limit state), determined by using Eq. 14.4, are presented in Table 14.2, together with the probabilities of failure in 50 years (the average lifetime of the structures), which is determined as $P_{NC,50} = 1 - (1- P_{NC})^{50}$, and with the corresponding return periods T_{NC}.

The results presented in Table 14.2 indicate a very large probability of failure of the structure that was not designed for seismic resistance. The probability of failure of the structure designed according to a modern code is about 25 times smaller, but still quite considerable (somewhat larger than the tolerable probability according to draft revised EC8). However, it should be noted that, in the analysis, the NC capacity was defined in a quite conservative way, so the computed probabilities may represent an upper bound of possible estimates.

Table 14.2. The peak ground accelerations PGA_{NC} corresponding to the displacement capacities d_{NC} at failure (NC limit state), the corresponding spectral accelerations at the fundamental natural periods A_{NC}, the mean values of the hazard function at A_{NC}, the annual probabilities of failure, the probabilities of failure in 50 years (in %), and the return periods of failure for two variants of SPEAR building.

SPEAR	PGA_{NC} (g)	A_{NC} (g)	$H(A_{NC})$	P_{NC}	$P_{NC,50}$ (%)	T_{NC} (years)
Test	0.25	0.40	$0.32\cdot10^{-2}$	$0.80\cdot10^{-2}$	33	125
EC8-H	0.77	1.89	$1.12\cdot10^{-4}$	$2.79\cdot10^{-4}$	1.4	3600

15 FLOOR SPECTRA

15.1 Introduction

Typically, the structural components of a commercial building account for approximately 15-25% of the original construction cost, while the non-structural components (mechanical, electrical, plumbing, and architectural), which, in this chapter, will be referred to as secondary elements or equipment, account for the remaining 75-85% of the cost (FEMA 2012). Nevertheless, in general, not enough attention has been paid to the seismic behaviour of secondary elements, with the exception of the nuclear industry, where the main concern is the seismic safety of the equipment.

Slovenia has one nuclear power plant (NPP), located in Krško, on the Sava River between Ljubljana and Zagreb. The Westinghouse pressurised water reactor was designed in the 1970s and went into commercial operation in 1983 as a joint venture of Slovenia and Croatia, which were at the time both parts of former Yugoslavia. The location of the NPP in Krško is characterised by relatively high seismicity. My colleagues and I did not take part in the original design. Later, however, in the process of risk re-assessment and reconstruction, we were regularly involved in the studies related to the seismic risk of NPP Krško.

When working with NPP Krško, I became aware of the importance of the floor acceleration spectra that are used for the design and assessment of acceleration sensitive equipment. These spectra can be applied when the mass of the secondary element is significantly smaller than the mass of the primary structure, which is usually the case. Floor spectra depend both on the characteristics of the ground motion and those of the structure and can be "accurately" determined by performing a dynamic response history analysis of the structure. This is a time-consuming approach, which is only used exceptionally, e.g., in the nuclear industry. In everyday design practice, an approximate approach is usually used, in which the floor spectra are determined directly from the design ground motion characteristics. Such a method is called a "direct method".

From the seismic point of view, the buildings within an NPP are, due to their robustness, usually not a problem. A potential problem is the equipment, which may be subjected to very high accelerations. Since seismic damage in the nuclear industry is generally not allowed, the building structures are supposed to remain in the elastic range of behaviour. In resonance, i.e., in the case in which the natural period of a secondary system (equipment) is approximately equal to the natural period of the primary structure, very high amplification of accelerations (compared to the accelerations of the building) may occur, especially if the damping of the equipment is small. Of course, floor spectra are important not only for nuclear industry with extreme safety and related cost requirements, but also for other structures, e.g., industrial buildings and hospitals, which are allowed to be at least partially damaged, and for which analysts cannot afford using the complex procedures for the determination of floor spectra with response history analysis. From limited literature on floor spectra available in 1980s, it has been known that inelastic behaviour may have a beneficial effect by reducing the floor spectral accelerations, especially in the case of tuned equipment. My idea was to develop a direct procedure for the determination of floor spectra that would take into account the inelastic structural behaviour.

The initial work on direct methods was performed by Tomaž Vidic in his bachelor's thesis in 1986. In his further work, he moved to inelastic spectra (see Chapter 5), and the work on floor spectra in our research group was paused until Dejan Novak came in early 1990's to study for his doctoral degree. He had a master's degree in mechanical engineering, and floor spectra seemed to be a natural choice for his thesis topic.

At that time, a direct method was developed by Yasui et al. (1993) for elastic structures. We found this method to be convenient for practical applications and reasonably accurate in the off-resonance regions, so we decided to use it. The method takes into account the dynamic characteristics of the primary structure and the elastic ground acceleration spectrum. Considering that at that time inelastic response spectra had been under development in our research group, our idea was to use an inelastic acceleration spectrum (instead of the elastic one), if the inelastic behaviour of the primary structure was to be taken into account.

Preliminary results of the study of floor spectra were summarised in two conference papers. In the first one, presented in 1995 at 13th SMiRT (Structural Mechanics in Reactor Technology) conference (Fajfar and Novak 1995), we showed the results of a parametric study of floor spectra for an inelastic primary SDOF structure. The conclusions summarised in this paper remain entirely valid. Among others, we found that the amplification of peak acceleration from the primary to the secondary system is a relatively stable parameter, which depends mainly on the damping of the secondary system, and is quite insensitive to the characteristics of the primary system. At the end of the paper, we stated that "a simple procedure for an approximate determination of floor response spectrum is under development". In fact, the procedure was drafted very soon, and included in a paper presented at the Slovenian conference of structural engineers (Novak and Fajfar 1994). (Note that the 1995 paper for the SMiRT conference was written and submitted before the 1994 paper for the Slovenian conference.) We proposed using the Yasui formula, where, for the case of inelastic primary structures, the elastic ground acceleration spectrum was replaced by the inelastic spectrum, developed by Vidic et al. (1994). Moreover, it was proposed to limit the spectral amplification in resonance region to the empirically determined values, since the Yasui formula did not work well in the resonance region. Thus, the draft procedure proposed in 1994, used, for SDOF primary structures, basically the same ideas as the current procedure which was elaborated on much later.

Unfortunately, for some private reasons, Dejan Novak prematurely quit his doctoral studies, and no work on floor spectra was done in our research group for about one-and-a-half decades, until 2010, when Vladimir Vukobratović decided to prepare his doctoral thesis on floor spectra. He started at the point where Dejan Novak finished. The fact that the main idea of the direct method for inelastic primary structures was still appropriate after such a long time suggests that, globally, in that period of time not much had been done on floor spectra.

Vladimir completed his master's degree at the University of Novi Sad (in present-day Serbia). He was a very bright and highly motivated PhD student. During his doctoral study in Ljubljana, he learned Slovenian language very well, so we communicated in Slovenian. After completing his doctorate in 2015, Vladimir returned to Novi Sad, where he became an Assistant Professor at the Faculty of Civil Engineering and a consultant in a well-known design office.

Vladimir Vukobratović has done excellent work, which included, inter alia, an in-depth study of the background of Yusui's formulas, which was not an easy task, as well as extensive parametric studies of floor spectra for SDOF and MDOF elastic and inelastic primary systems. Much time and effort were spent studying modal combination rules, since the standard SRSS and CQC combination rules sometimes do not work well for the combination of absolute accelerations. As a result of Vladimir's work, in addition to his doctoral thesis (in English), two journal papers were published, in which a direct method for the determination of floor acceleration spectra was proposed. The first one was limited to SDOF primary structures (Vukobratović and Fajfar 2015), whereas the second one was dealing with MDOF structures (Vukobratović and Fajfar 2016).

In 2015, the work on the revised EC8 started. EC8, like other seismic codes, does not include an adequate procedure for the determination of floor spectra. Oversimplified formulae that do not take into account all relevant parameters are used. We anticipated that the newly proposed method would be a candidate for the implementation in the revised EC8, so we quickly prepared a simplified code-oriented version. We realised that, for a code application, one potentially important influence was missing in our method, i.e., the inelastic behaviour of the equipment. Based on a quick study, we suggested taking into account this influence, if relevant, by increasing the damping of the equipment. We also provided some numerical values. A short paper, entitled "Code-oriented floor acceleration spectra for building structures", was published in the Bulletin of Earthquake Engineering (Vukobratović and Fajfar 2017). The proposed method was accepted by the project team developing the revised EC8, and implemented in the draft of the revised EC8. The direct method, as defined in our 2017 paper, is included in an annex to the part of EC8 dealing with buildings EC8-1-2 (CEN 2019c). In discussions on drafts of the revised EC8, it was discovered that the proposed procedure does not provide correct results in the bottom-most part of the building if all relevant vibration modes are not taken into account, which is a common case in practice. Also, it does not allow the computation of floor spectra at elevation zero. For this reason, a lower limit for floor spectral values, which applies to the bottom part of a building, was added in the EC8 version (see the next section).

15.2 Direct Method

The direct method for determination of floor acceleration spectra, implemented in draft revised EC8, is based on the method originally developed by Yasui et al. (1993) for elastic primary structures and equipment (elaborated mainly for SDOF primary systems). It also allows the taking into account the inelastic behaviour of the primary structures and equipment using the idea for the extension of this method to inelastic structures proposed by Novak and Fajfar (1994). The method can be used for equipment modelled as an SDOF oscillator. The inelastic behaviour of equipment can be taken into account approximately by increasing its damping. The developed direct method is based on the principles of structural dynamics. Due to its theoretical background, it is considered to be valid for general systems. Empirical values obtained in a parametric study are used only in the resonance region.

In order to determine the floor acceleration spectra, first, an elastic modal response spectrum analysis of the building structure (referred to as the primary structure) has to be performed. If inelastic structural behaviour is taken into account, then a pushover-based analysis of the primary structure, e.g., employing the N2 method, is also needed. If the higher mode effects are neglected, the described direct method can be used by taking into account the fundamental mode only.

The floor acceleration spectrum represents the acceleration of the secondary system A_s as a function of the period of the secondary system T_s. For each mode of the primary structure that is taken into account, the floor acceleration spectra are determined separately and then combined in order to obtain the resulting floor response spectrum.

For mode ‘i’ and floor ‘j’, the value of the floor acceleration spectrum is determined as:

$$A_{s,ij} = \frac{\Gamma_i \varphi_{ij}}{\left|\left(\frac{T_s}{T_{p,i}}\right)^2 - 1\right|} \sqrt{\left(\frac{A_{ep,i}}{R_\mu}\right)^2 + \left(\frac{T_s}{T_{p,i}}\right)^4 A_{es}{}^2} \tag{15.1a}$$

$$\left|A_{s,ij}\right| \leq AMP_i \left|PFA_{ij}\right| \tag{15.1b}$$

$$PFA_{ij} = \Gamma_i \varphi_{ij} \frac{A_{ep,i}}{R_\mu} \tag{15.2}$$

$$AMP_i = 2.5\sqrt{\frac{10}{5+\xi_s}} \qquad T_{p,i}/T_C = 0 \tag{15.3a}$$

$$AMP_i = \text{linear between (15.3a) and (15.3c)} \qquad 0 < T_{p,i}/T_C < 0.2 \tag{15.3b}$$

$$AMP_i = \frac{10}{\sqrt{\xi_s}} \qquad T_{p,i}/T_C \geq 0.2 \tag{15.3c}$$

Eq. 15.1a corresponds to the off-resonance region. The plateau of the floor acceleration spectrum in the resonance region, where the period of the equipment is equal or near to the period of the primary structure, is determined as a product of the peak floor acceleration PFA_{ij} and an empirical amplification factor for the considered mode AMP_i (Eq. 15.1b). Since $A_{s,ij}$ and PFA_{ij} can be either positive or negative, absolute values are used in Eq. 15.1b. The equation for the peak floor acceleration PFA_{ij} (Eq. 15.2) represents a special case of Eq. 15.1a, for $T_s = 0$. Values of AMP_i can be determined from empirical Eq. 15.3.

The indices "*p*" and "*s*" correspond to the primary structure and the secondary element (i.e., equipment), respectively. A_e is a value in the elastic acceleration spectrum, which represents the seismic demand. $A_{ep,i} = A_e(T_{p,i}, \xi_{p,i})$ applies to the ith mode of the primary structure, whereas $A_{es} = A_e(T_s, \xi_s)$ applies to the equipment. The natural periods of the ith mode of the structure and equipment are denoted as $T_{p,i}$ and T_s, respectively, whereas $\xi_{p,i}$ and ξ_s denote the damping values of the structure (for the ith mode) and of the equipment, respectively. In Eq. 15.3, T_C is the characteristic period of the ground motion (which is equal to T_C in Eurocode 8), and ξ_s is expressed as the percentage of critical damping. R_μ is the ductility dependent reduction factor due to the inelastic behaviour of the primary structure (see Chapter 3). The term ($A_{ep,i}$ / R_μ) represents the value in the inelastic acceleration spectrum for the primary structure. It can be replaced by the ratio of the yield force and the mass of the equivalent SDOF primary system. In the case of an elastic structure, $R_\mu = 1.0$. Γ_i is the modal participation factor for the ith mode, whereas φ_{ij} represents the ith mode shape value at the jth floor. In the case of a simple planar structural model with concentrated masses, Γ_i is defined by Eq. 15.4, where m_j is the mass at the jth floor:

$$\Gamma_i = \frac{\sum \varphi_{ij} m_j}{\sum \varphi_{ij}^2 m_j} \tag{15.4}$$

Eq. 15.3c was proposed by Sullivan et al. (2013). According to this formula, the amplification factor, which is defined as the ratio between the peak value (plateau) in the floor acceleration spectrum, A_s, and the peak floor acceleration *PFA* (Eq. 15.2), depends only on the damping of the secondary element ξ_s. The results of the performed parametric studies indicated that this is the most important parameter influencing the amplification factor. The influence of hysteretic behaviour, ductility, and the ratio T_p/T_C is small to moderate, except when this ratio has small values. Thus Eq. 15.3c is a simple and viable option for implementation in codes. It yields very similar results to those of the empirical relations determined in our early studies (Novak and Fajfar 1994, Fajfar and Novak 1995). More elaborate empirical formulae, which also take into account the effects of hysteretic behaviour, ductility and the ratio T_p/T_C, are presented in (Vukobratović and Fajfar 2016).

In the case of the inelastic primary structure, it is assumed that the inelastic behaviour is related only to the fundamental mode, whereas all higher modes are treated as elastic. Thus, R_μ is calculated only for the fundamental mode, whereas for all higher modes (i > 1), R_μ amounts to 1.

A pushover-based procedure, e.g., the N2 method, is needed in order to determine some parameters related to the inelastic structure. These are the inelastic deformation shape, the effective natural period T_p^*, and the ductility demand μ. T_p^* has to be used in Eqs. 15.1-15.3 instead of $T_{p,1}$. Moreover, an inelastic deformation shape, normalised to 1.0 at the control point (usually at roof level), has to be used in Eqs. 15.1a and 15.2 instead of the fundamental mode shape $\boldsymbol{\varphi}_1$. Γ_1 should also be determined from the inelastic deformation shape.

Inelastic behaviour of ductile equipment reduces the floor acceleration spectra. This beneficial effect can be approximately taken into account by increasing the damping of the equipment. Our study indicated that floor acceleration spectra for elastic equipment with 10% and 20% damping approximately correspond to the spectrum for inelastic equipment in the case of a ductility demand μ_s equal to 1.5 and 2.0, respectively, and the actual damping of the equipment equal to 1%. It was also found that, with increasing inelastic behaviour of the equipment, the influence of its damping rapidly decreases. Based on these observations, and considering that, in practice, it is hardly possible to make a reliable estimation of the damping and ductility of equipment, the following suggestion was made: for inelastic equipment, the procedure developed for elastic equipment (Eqs. 15.1-15.3) should be used by taking into account ξ_s = 10%, independently of the actual damping of equipment. This approach generally provides fair estimates of the floor acceleration spectra if the actual ductility μ_s is 1.5, and the actual equipment damping is about 1% (Vukobratović and Fajfar 2017). The results are generally conservative in the case of higher ductility and/or higher damping.

The floor acceleration spectra calculated for individual modes should be combined in order to determine the resulting floor spectra. The modal superposition is a standard procedure in the case of elastic structures. As an approximation, it is often applied also for inelastic structures. In the case of the described direct method, the following modal combination approach is used for both elastic and inelastic primary structure. In the range of the periods of equipment from $T_s = 0$ up to and including the end of the plateau of the resonance region of the fundamental mode ($T_s = T_{p,1}$), the direct floor spectra obtained for individual modes should be combined by using the standard SRSS or CQC modal combination rules. In the post-resonance region of the fundamental mode (the rest of the period range), the algebraic sum should be used, in which the relevant signs of individual modes are taken into account. The upper limit of the resulting floor spectrum calculated as the algebraic sum is represented by the plateau obtained for the resonance region of the fundamental mode by using the SRSS or CQC rules.

The physical lower limit for the accelerations at the very bottom of a structure fixed in the ground is represented by the ground motion acceleration spectrum. Therefore, the ground acceleration spectrum represents the lower limit of **combined** floor acceleration spectra. Note that Eqs. 15.1 and 15.2 are not applicable to the lowest floor of structures with fixed supports ($\varphi_{i0} = 0$). Moreover, at the bottom of a building, higher vibration modes are usually quite important; therefore, the combined acceleration may be underestimated due to an approximate combination rule and because, typically, not all relevant vibration modes are taken into account.

It should be noted that, due to uncertainties in assessing natural periods of both the primary and secondary structures, a broadening of the spectra in the resonance region is required by seismic codes and standards. A broadening is implicitly included in the described direct method, especially in the case of the fundamental mode, as demonstrated in the illustrative example in the next section (Fig. 15.1).

15.3 Example Application

The variant of the SPEAR building designed in compliance with EC8 (see Chapter 16) is used as an illustrative example. The floor spectra are determined for elastic equipment with 5% damping by the direct method and compared with mean results of nonlinear response history analysis (NRHA) for a set of ground motions. (A more detailed analysis for another variant of the same structure is presented in (Vukobratović and Fajfar 2017).) Ground motion is defined by the type 1 elastic EC8 spectrum for soil type B. The peak ground acceleration PGA amounts to 0.35g; the soil factor is already included in this number. The ductility demand is $\mu = 2.4$. The fundamental period of the primary structure T_P is larger than the characteristic period of the ground motion $T_C = 0.5$ s; therefore, the reduction factor R_μ is equal to the ductility demand: $R_\mu = \mu = 2.4$. A hysteretic model with degrading stiffness (Takeda model) was applied. For comparison, floor spectra were also determined for the case in which the inelastic behaviour of the primary structure is not taken into account, i.e., the structural response is elastic. Also, in this case, both the direct method and (linear) response history analysis (LRHA) were used. The floor acceleration spectra are presented in Fig. 15.1 for the bottom (first) and top (third) floor, for X-direction. They apply to the mass centres.

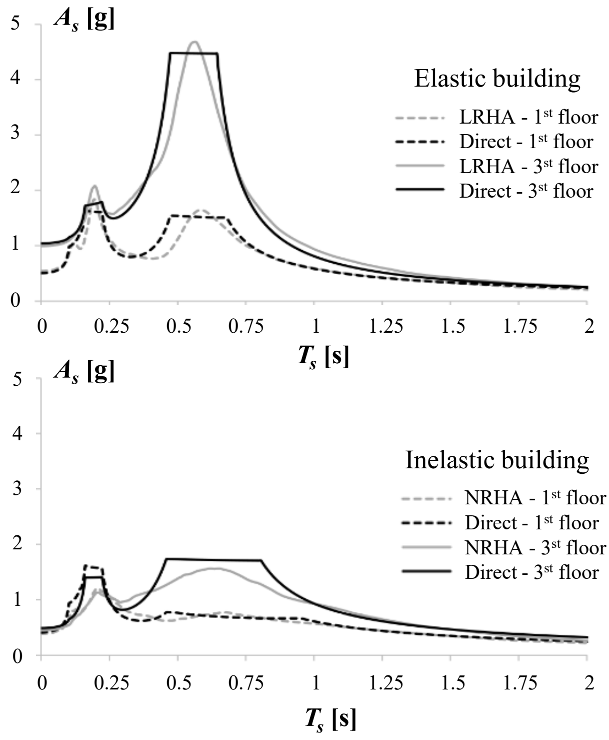


Figure 15.1. Comparison of floor acceleration spectra for the elastic and inelastic variant of the SPEAR EC8-H building obtained by the direct method and response history analysis. Damping of (elastic) equipment is 5%.

There is a good correlation between the results of the direct method and more accurate response history approach. A considerable reduction of the spectral values can be observed if the inelastic structural behaviour of the primary structure is taken into account, mainly in the resonance region related to the fundamental mode.

In the case that the equipment is allowed to deform in the inelastic range and dissipate energy, the same procedure for the determination of floor acceleration spectra can be used, but the damping of the equipment ξ_s is set to 10%, assuming that the ductility capacity of the equipment μ_s amounts to at least 1.5. In such a case, the accelerations shown in Figure 15.1 for elastic equipment are reduced, especially in resonance regions.

16 SPEAR BUILDING

The so-called SPEAR building is an asymmetric three-storey reinforced concrete frame structure. Several full-scale pseudo-dynamic tests of the original and repaired structure were performed at the European Laboratory for Structural Assessment (ELSA), which is a part of the Joint Research Centre (JRC) in Ispra, within a European research project from 2002 to 2005. Our institute was a partner in this project. Like other European researchers, we have extensively used this structure as a test example.

The basic data of the SPEAR building are shown in Figure 16.1. The building was conceived as representative of older construction in Southern European countries, without engineered earthquake resistance. It was designed for gravity loads only, according to the concrete design code implemented in Greece between 1954 and 1995, with the construction practice and materials used in Greece in the early 1970s.

In addition to the original SPEAR building (denoted as "Test"), a variant of the building was designed by my PhD student Matej Rozman in compliance with EC8 (denoted as "EC8 H") (Rozman and Fajfar 2009). Realistic values were used for permanent loads. As a result, the total mass of this variant was 45% greater than the total mass of the Test structure. The geometry of the whole structure remained the same as in the case of the Test structure, but the dimensions of individual load-bearing elements were changed. To comply with EC8 requirements, the dimensions of the columns were increased, and the dimensions of the beams were adjusted. The structure was designed for the EC8 design spectrum for ground type C, with a peak ground acceleration PGA = 0.29g. The high ductility class (DCH) was selected.

In our analyses, we have always applied reasonably simple mathematical models of the structure. For the SPEAR building, different doctoral students used slightly different models, so there are some minor differences in the results reported in various papers. However, in all cases, the basic modelling was the same. A space frame model with one element per member was used. Rigid diaphragms were assumed at the floor levels, due to monolithic RC slabs. Beam and column flexural behaviour was modelled by one-component lumped plasticity elements, composed of an elastic beam and two inelastic rotational hinges (defined by the moment–rotation relationship). The yield and the maximum moment were calculated taking into account the axial forces caused by the vertical loading on the frame. The analyses were performed by Matjaž Dolšek, Aurel Stratan, a visiting researcher from the University of Timișoara, and some of the PhD students (see, e.g., Fajfar et al. 2006). With our model, we were able to provide a reasonably

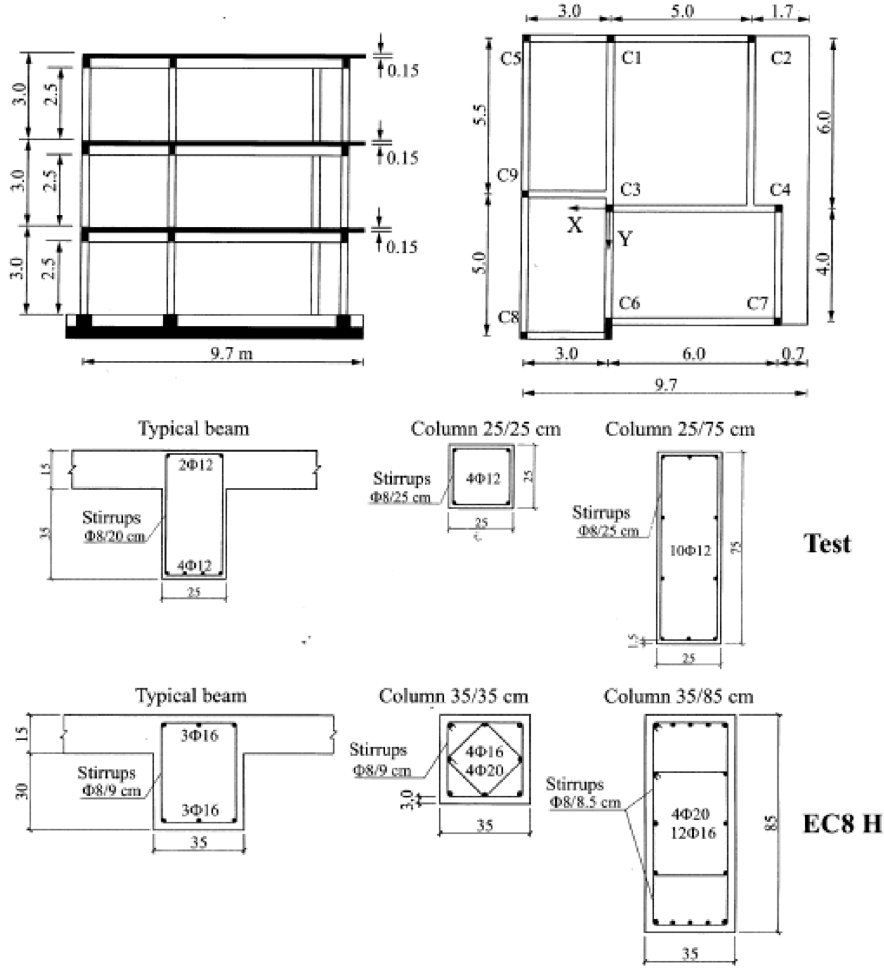


Figure 16.1. Elevation and plan view of the SPEAR building, and the typical cross-sections of the beams and columns of the variants Test and EC8 H.

accurate pre-test prediction of the structural response, which aided in selecting the appropriate intensity of the test ground motion. The post-test analyses simulated the measured response of the structure very well. Of course, some parameter fitting was necessary. Nevertheless, the results confirmed our belief that the main problem in seismic analysis is uncertain input data. With reliable input data, relatively simple mathematical models are able to provide acceptable results. In the paper, describing pre- and post-test analyses (Fajfar et al. 2006), we concluded:

The simple one-component model with concentrated plasticity has proved to be robust and efficient. The typical computing time for a time-history analysis amounted to about 2 minutes on a PC with an Intel Pentium 4 processor (3.0 GHz, 512MB RAM). The engineering judgement, which is needed in the mathematical modelling of complex existing RC structures, and is based on experience gained by studying experimental results and by numerical simulations of experiments, can be more easily exercised (mostly by adopting reasonable moment–rotation relationships in plastic hinges) than in the case of micro-modelling. It is the authors' view that models using one component members are a promising practical tool for use in design offices. Since the accuracy of results is mostly controlled by large uncertainties and the randomness of the input parameters, more sophisticated models do not necessarily provide more reliable results, as has also been demonstrated within the framework of the SPEAR project.

In fact, a highly sophisticated model used by one of the project partners required about three weeks of computing time for a single response history analysis and provided much worse results.

In the following, some results of analyses, needed for better understanding of some parts of this monograph, are presented.

The three fundamental periods of vibration of the building amount to 0.80 s, 0.69 s, and 0.58 s for the Test structure, and 0.56 s, 0.53 s, and 0.41 s for the EC8 H structure. For both structures, the first mode is predominantly in the X-direction, the second predominantly in the Y-direction, and the third mode is predominantly torsional.

The pushover curves for the X-direction are presented in Figure 16.2. The points corresponding to the NC limit state are marked, i.e., the deformation at which the NC limit state (expressed in terms of the ultimate chord rotation, which was determined quite conservatively) is attained at the first column. The pushover curves clearly demonstrate the larger stiffness, strength, and ductility of the structure designed according to EC8. The critical element is column C1 at the top of the second storey for the Test structure and the same column at the base for the EC8 H structure. In the case of the Test structure, a plastic mechanism is formed in the lower two storeys. Almost all the columns in the first and second storeys yield at both joints, whereas most of the beams remain in the elastic region. A favourable global plastic mechanism, in which all the beams yield, as well as the columns at their fixed base, occurs in the case of the EC8 H structure.

In Figure 16.3, the capacity diagrams and ground motion capacities in terms of elastic spectra (EC8 shape, soil type C) corresponding to the failure, i.e., the NC limit state are plotted. Considering the fundamental periods of the equivalent SDOF system T^*= 0.94 s and T^* = 0.61 s for the Test structure and the EC8 H structure, respectively, the spectral values $A_{NC}(T)$ amount to 0.40 g and to 1.89 g for the former and latter structure, respectively. Taking into account the EC8 spectral shape for soil type C, the corresponding peak ground accelerations PGA_{NC}

amount to 0.25 g and 0.77 g, respectively. As expected, the EC8 H structure is able to resist a much more severe ground motion than the Test structure. The inelastic spectra (corresponding to the ductility demand μ=3.2 and μ=6.5 for the Test structure and the EC8 H structure, respectively) using the simple R_μ–μ–T relation (section 5.4) are also plotted in Figure 16.3.

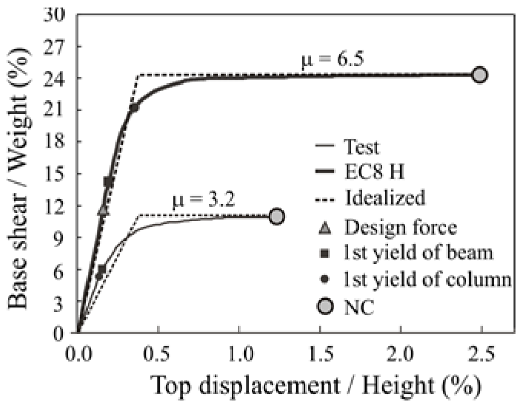


Figure 16.2. The normalised base shear – top displacement relationship for two variants of the SPEAR building - X direction (height of the structure = 9 m, weight W = 1900 kN for Test structure and W=2900 kN for EC8 H structure) (from Fajfar and Dolšek 2012).

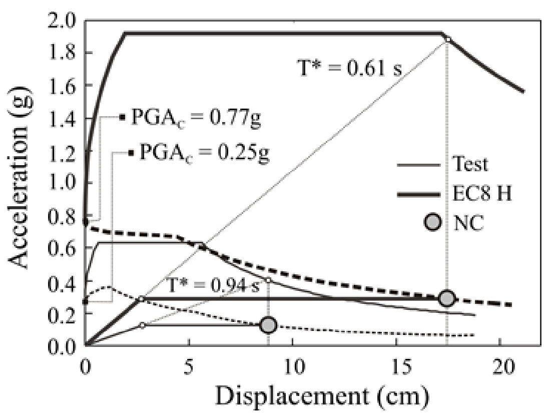


Figure 16.3. Elastic and inelastic spectra corresponding to the NC limit state and capacity diagram for the idealised SDOF system for two variants of the SPEAR building - X direction (from Fajfar and Dolšek 2012).

17 INTERNATIONAL COOPERATION

17.1 Bled Workshops

"With its bluish-green lake, picture-postcard church on an islet, a medieval castle clinging to a rocky cliff and some of the highest peaks of the Julian Alps and the Karavanke as backdrops, Bled is Slovenia's most popular resort" (description in Lonely Planet), located only about 50 km from Ljubljana. This place was a venue of four very successful international workshops, which became known as the "Bled workshops". They were a result of scientific cooperation between the US and Slovenia. The first three, in 1992, 1997, and 2004 were organised by Helmut Krawinkler from Stanford University and me, with great help from my colleagues at the University of Ljubljana. For the last workshop in 2011, dedicated to the organisers of the first three workshops, my colleague Matej Fischinger accepted the burden of organising, with some help from the American partner Boža Stojadinović, who was at that time at the University of California at Berkeley.

The beginning of the story was the "US-Yugoslavia Joint Fund for Scientific and Technical Cooperation", which was established in 1973 to encourage and support a wide range of scientific and technical cooperation between the former Yugoslavia and the USA. Earthquake engineering was one of the most popular topics in this cooperation. In Slovenia, initially only the Institute for Testing and Research in Materials and Structures (ZRMK) in Ljubljana participated in the joint fund projects related to earthquake engineering. The institute's manager, Viktor Turnšek, was an excellent engineer involved in research on masonry structures. In the early 1970s, the University of Ljubljana, which would be a natural partner in the Joint Fund projects, had not yet been involved in earthquake engineering research. When I started working in this field, I was very fortunate to get the opportunity to participate in a joint project on masonry structures between ZRMK and the University of California in Berkeley, with Jack Bouwkamp as the principal investigator. This involvement allowed me to spend three months in 1980 at Berkeley as a visiting scholar working under the supervision of Graham Powell, the author of DRAIN-2D, the most popular program for nonlinear structural analysis at that time.

My stay in Berkeley was perhaps the most important event of my career. At that time, UC Berkeley was a mecca for earthquake engineering with a large number of professional giants among its faculty members. I had a chance to interact with Vitelmo Bertero, Jack Bouwkamp, Anil Chopra, Ray Clough, Armen DerKiureghian, James Kelly, Steve Mahin, Hugh McNiven, Joseph Penzien, Egor Popov, Graham Powell, Robert Taylor, and Edward Wilson, among others: a real "who's who" in earthquake engineering. With many of them, I have been in contact in the following decades. The same applies to several visiting scholars from all around the world, who were at UC Berkeley at that time. In addition to my work on the development of an element for masonry walls for DRAIN-2D, which occupied most of my three-month stay in Berkeley, I used the time for collecting information related to earthquake engineering. Among other things, I attended the course Structural Design for Seismic Loads, given by Vitelmo Bertero. This course, taught in a passionate way that was typical for Bertero, opened my eyes to real problems in earthquake engineering.

A particular pleasure was to chat with Ray Clough and Joseph Penzien, the authors of the famous textbook "Dynamics of Structures" (Clough and Penzien 1975). At the time, it was the leading book in structural dynamics. Its deterministic part was the basis of my course "Dynamics of Structures", which I held at the University of Ljubljana from 1974 until my retirement. It also was a model for my textbook written in Slovenian in 1984, in which, in addition to standard topics of structural dynamics, the analysis of multi-storey structures (see Chapter 8) was included. My 550-page textbook is still the only book on structural dynamics in the Slovenian language. Although nowadays the great majority of Slovenian students and engineers speak English, it is crucial to write professional books in the native language in order to preserve it (the population of Slovenia is only about 2 million) and to develop it by introducing new technical terms. Returning to Berkeley, it is not a coincidence that the most popular book on structural dynamics in the last 25 years also stems from there. Chopra's "Dynamics of Structures" (Chopra 1995) has already had five editions.

In 1983, my efforts to involve the University in Ljubljana as a partner in the Joint Fund projects resulted in a project on seismic codes entitled "Evaluation of aseismic provisions in the USA and Yugoslavia" with the National Bureau of Standards (NSB) in Washington DC. This was the start of a very fruitful and beneficial official cooperation between our research group and US institutions. Several joint projects with UC Berkeley (Steve Mahin served as the US principal investigator), Stanford University (Helmut Krawinkler) and NSB, which later became the National Institute of Standards and Technology (NIST), followed. The projects have been focused on mathematical modelling and nonlinear analysis of RC structures, as well as on the code applications. This formal cooperation allowed my colleagues and myself to establish and maintain contacts with many US researchers and to be informed directly about the recent developments in earthquake engineering. The Joint Fund provided travel funds for our yearly visits to the US, and, very importantly at that time, convertible currency needed for travelling (in former Yugoslavia, it was not possible to officially change the local currency for foreign currency).

The closest relations were developed with Helmut Krawinkler. Every summer, Helmut visited his home country Austria and used this opportunity also to visit me in Slovenia. During one of his visits, the idea of organising an international workshop arose. After consultation with NSF and NIST officers, in early 1990 we submitted a proposal to the Joint Fund for a four-day workshop entitled “Nonlinear Seismic Analysis of RC buildings”. The workshop was scheduled for June 1991 in Bled. Its aim was a review of the state of knowledge on (a) mathematical modelling of RC structural walls and (b) energy concepts in seismic analysis and design. The plan was to invite 10 participants from US, 13 from different republics of Yugoslavia, both groups funded from the Joint Fund, and about 10 participants from third countries, at their own expense. The planned date of the workshop coincided with the conclusion of three of our joint research projects, so the workshop was also intended to serve as a place for presenting and discussing the results of the joint projects. The idea was to give more emphasis on discussion than on presentations, so the participants were asked to provide papers in advance.

In late 1990 and early 1991, Yugoslavia was very near collapse. In December 1990, a referendum on the independence of Slovenia was held, in which 88% of all Slovenian residents voted for the independence of Slovenia from Yugoslavia. Slovenia became independent through the passage of the legislation on 25 June 1991, on the very day that the Bled Workshop was scheduled to start. In the months before the scheduled workshop, many participants were concerned about the situation in Yugoslavia and questioned whether the workshop would take place. My colleagues and I naively underestimated the danger and assured that there were no problems and that the workshop would take place. A shock came with a phone call from the US embassy, which suggested cancelling the workshop. In any case, the US participants were not allowed to come. It seems that the US government was very well informed and has correctly foreseen the real probability of a war. In fact, it started in the morning of the day after the formal independence, one day after the workshop was supposed to start. In the “Ten-Day War”, the Slovenian forces successfully rejected Yugoslav military interference. The war ended with an agreement, and the Yugoslav National Army began its withdrawal from Slovenia. By the end of 1991, the last Yugoslav soldier left Slovenia.

At the time of the war, Helmut was in Austria. I was able to meet him there, and we decided to postpone the workshop for one year. So, the workshop was held on July 13-16, 1992. At that time, wars were being fought near Slovenia, in Croatia and Bosnia. Several invitees were afraid to come and cancelled their participation. I especially regretted the cancellation of Bertero, whose work I had greatly admired since my stay in Berkeley. I called him and tried to persuade him to come, saying that the danger in Slovenia was negligible compared to the danger somebody is exposed during post-earthquake field investigations. He answered “Peter, I know. But, you know, my wife will not allow me to go to Slovenia.”

Despite some unfortunate cancellations, a group of 21 prominent researchers from seven countries attended the workshop. It was one of the first international events in the newly independent state of Slovenia, so it attracted a great deal of attention, including from state authorities. Exciting technical and social programmes, the individual treatment of each participant, and the venue directly beside the beautiful lake contributed to the great success of the workshop. The participants were happy with the format of the workshop, proposed by Helmut Krawinkler, which allowed much time for in-depth discussions. The participants and the accompanying persons enjoyed an excellent social programme, designed mainly by Matej Fischinger who also made significant contributions to all other aspects of the organisation of the workshop. The main person behind the technical organisation was Janez Reflak, the head of our institute, who, together with the institute's secretary Darja and with our doctoral students, took care of the smallest details.

The same format and approach have been used, and the same team has been involved in the Bled workshops that followed. Since good word spread among colleagues around the world, our major problem related to subsequent Bled workshops was the selection of invited participants. There was a great interest in participation, but the format of the workshop did not allow too many participants. I know that many colleagues were unhappy about not being invited.

As a result of the first workshop, two monographs were published. The first, entitled "Nonlinear seismic analysis of reinforced concrete buildings", containing 23 papers, was published by Elsevier (Fajfar and Krawinkler 1992) before the workshop. The papers were divided into two sections: Energy concepts and damage models, and Behaviour of buildings with structural walls. The follow-up publication was published in December 1992 as The John A. Blume Earthquake Engineering Center (Stanford University) Report (Krawinkler and Fajfar 1992). It contains additional papers submitted at the time of the workshop, together with written discussions and conclusions. Our research group contributed papers on consistent inelastic spectra and equivalent ductility factors, taking into account cumulative damage, and on the multiple-vertical-line element model for nonlinear seismic analysis of structural walls.

The highlight of the Bled workshops was the second one, entitled "Seismic design methodologies for the next generation of codes", organised in June 1997, five years after the first one, at the time when performance-based design was emerging. The main sponsors were the "US-Slovenian joint board for scientific and technological cooperation" (continuing the tradition of the former US-Yugoslav joint fund), the US National Science Foundation, the US National Institute of Standards and Technology, and the Ministry of Science and Technology of the Republic of Slovenia. Thirty-seven participants and 10 observers from 13 countries attended the workshop. The list of participants included the majority of top researchers in earthquake engineering at that time and some leading practical engineers involved in code development. The workshop was intended to assess the state-of-the-art in earthquake engineering, to define the future directions for the development of seismic design methodologies, and to identify the research needs. Two topics were specifically addressed: (a) performance-based engineering concepts and (b) specific design and performance evaluation approaches for buildings and bridges. Papers on these topics were submitted well before the workshop, distributed in the form of preprints, and presented during the first two days of the workshop. The last two days were devoted to discussions organised in the form of plenary sessions and working group sessions. The proceedings (Fajfar and Krawinkler 1997) contain resolutions, conclusions, and recommendations made at the workshop, as well as a compendium of the final versions of the invited papers. As stated in the Resolutions,

> the Workshop provided a valuable forum to exchange research results and ideas on issues of importance to seismic risk reduction and the development of future seismic codes. . . . The participants . . . agreed that present codes need significant improvements and expansion, and that performance-based engineering concepts provide a suitable framework for this purpose.

One of our contribution, presented at the workshop, was related to the N2 method (Fajfar et al. 1997). The basic features of the method and the preliminary extension of its applicability to infilled frame structures and to bridges were discussed.

The workshop has had quite a substantial impact and initiated considerable progress worldwide to establish concepts and methods for performance-based earthquake engineering. It was a noticeable achievement to find a consensus between the different schools of thought represented at the workshop and to prepare conclusions acceptable for all the participants.

It is unusual that reviews of conference proceedings are published in leading journals. This happened in the case of the second Bled workshop. Robert Reitherman published a detailed critical review in Earthquake Engineering and Structural Dynamics (Vol. 27:1559-1562, 1998). In Earthquake Spectra (Vol.15 (2)) William J. Hall concluded his review with:

> In conclusion, this book contains the best collection of well-thought-out contributions on this subject in one place the reviewer has seen to date. For anyone who is forward-thinking about where research and development are taking us in the evolution of seismic design, this volume has much to offer. It is particularly useful not only to researchers in this field, but also to code and guideline developers, and to those in practice who strive to be at the forefront in performance-based applications – as difficult as that may be at this point in time.

It was seven years before the next Workshop on “Performance-based seismic design – concepts and implementation” was organised in Bled, at the end of June 2004, aimed at continuing an international dialogue on the worldwide implementation of new ideas. For this workshop, the main sponsors were the

Pacific Earthquake Engineering Research (PEER) Center, the world leader in the development of performance-based earthquake engineering, and the Ministry of Education, Science and Sport of Slovenia. Forty-five invited participants and 12 observers from 14 countries addressed the following topics: loss estimation, fragility and vulnerability and impact on risk management; implementation in engineering practice; performance-based design concepts; and integration of experimental and analytical simulations. An advisory committee consisting of Jack Moehle (chair), Greg Deierlein, Michael Fardis, and Toshimi Kabayasawa helped the two main organisers and editors of the workshop proceedings. The monograph contains resolutions, conclusions and recommendations made at the workshop, as well as a compendium of the final versions of 43 papers, whose initial versions were submitted before the workshop and posted on the workshop website. My colleagues and I prepared a paper entitled "Extensions of the N2 method – asymmetric buildings, infilled frames and Incremental N2". Some observations and discussions of the working groups are also included in the proceedings. To simplify the publication procedure, we decided to publish the proceedings as a PEER report (Fajfar and Krawinkler 2004). This was probably not a wise decision. The publishing process was very smooth. However, although both hard copies and a free web version were available, the proceedings have not reached such a broad worldwide audience as the proceedings of the second Bled workshop, published by a specialised international publishing company.

Another seven years passed before the fourth Bled workshop, entitled "Performance-Based Seismic Engineering – Vision for an Earthquake Resilient Society", was organised in June 2011. The younger generation took over the organisation of the workshop. Matej Fischinger and Boža Stojadinović, at that time still at the UC Berkeley, chaired the workshop with the help of the regional coordinators Masayoshi Nakashima (Japan/Asia), Peter Fajfar (Europe), and Jack Moehle and Andrei Reinhorn (Americas). For this workshop there were no major sponsors. Every invitee, with the exception of some special guests, was requested to cover his/her own expenses, whereas the organisation costs were covered with research funds of our institute.

The aim and scope of the workshop, as well as a summary of previous workshops, were described by Matej Fischinger in the Preface of the monograph published after the workshop (Fischinger 2014):

> Three famous workshops (those which were held in 1992, 1997, and 2004), which became known simply as the Bled workshops . . . produced widely cited reference books, which provided visions about the future development of earthquake engineering, as foreseen by leading researchers in the field. There are very few scientific events which can repeatedly bring together the best and leading researchers from all over the world, and thus provide a forum with a strong impact and authority for important developments in a particular scientific field. During Bled 1 (1992) the new

> emerging tools of nonlinear seismic analysis and design were discussed. These tools were, at the time, and still are, a prerequisite for modern performance-based earthquake engineering, a burgeoning idea that was incubated in the minds of the participants. During Bled 2 (1997) it became clear that performance-based design had become one of the leading new ideas in earthquake engineering. By the time Bled 3 was convened, in 2004, the procedures and methods of performance-based design and evaluation, which had been developed during extensive research, were being gradually adopted into everyday practice.
>
> Now, 20 years after the foundation of the tradition of the Bled workshops, we are witnesses to a world-wide breakthrough of this idea, with many different implementations and applications. The major research activities in the field of performance-based earthquake engineering have been supported and coordinated by large networks of research institutions and laboratories. However, even if this significant progress is taken into account, the earthquake engineering community is still facing many big challenges. Over just the last 5 years, several devastating earthquakes have reminded us that these destructive events still threaten the lives of millions of people, and very large amounts of property, as well as the social structure and economic well-being of individuals, communities, and countries all over the world. These events have clearly demonstrated that some of the traditional concepts of performance-based design are becoming out-of-date. First of all it has become clear that our research interest should go beyond the narrow technical aspects, and that the seismic resilience of the society as a whole should become an essential part of the planning and design process. The Bled 4 workshop was organized in order to discuss, develop and promote this idea in the light of the state-of-the-art achievements in the field, and this book presents the outcomes of this event. The workshop started exactly 20 years after the day when Slovenia had declared independence, 40 years after the Institute of Structural Engineering, Earthquake Engineering and Construction IT (IKPIR) had been established at the University of Ljubljana, and 500 years after the strongest earthquake to ever hit Slovenian lands, which occurred in 1511.

The workshop, attended by 41 participants, was as successful as the earlier ones. Some problems arose after the workshop, in the phase of the preparation of the monograph, published by Springer within the series "Geotechnical, Geological and Earthquake Engineering". To our great sorrow, soon after the workshop, the earthquake engineering community faced the loss of Helmut Krawinkler. As indicated in the preface to the monograph, "to honour Helmut's memory, Gregory Deierlein prepared the introductory chapter of the book based on Helmut's PowerPoint presentation, which had been presented at the beginning of the 2011 workshop. So the book includes priceless Helmut's last address to the engineering

Figure 17.1. Cover pages of Bled workshop proceedings.

community and his vision and advice for the future development of performance-based design and earthquake engineering." Moreover, Boža Stojadinović moved from the US to Europe, which prevented him from effectively cooperating in the editing of the monograph. Due to these problems, the monograph, edited by Matej Fischinger, was published with some delay.

In the monograph, the section entitled "Pushover-based analysis in performance-based seismic engineering – A view from Europe", was contributed by myself and Matjaž Dolšek. In this contribution, we advocated the use of pushover-based methods, claiming that such methods (although subject to several limitations) often represent a rational practice-oriented tool for the estimation of the seismic response of structures. We showed that the relations between quantities controlling the seismic response could be easily understood if a pushover-based analysis is presented graphically in the acceleration-displacement (AD) format. The N2 method and its extensions were very briefly summarised. Additionally, some recent pushover-based applications were listed. Finally, as an example of the application of pushover analysis, the seismic performance assessment of a multi-storey building with consideration of aleatory and epistemic uncertainties was presented.

For my colleagues and me, the four Bled workshops, which were organised over a period of two decades, were a unique, extremely useful and pleasant experience. The workshops enabled a close interaction with the most distinguished individuals working in earthquake engineering and, in some cases, also with their institutions. Lively discussions were an excellent source of new ideas. The workshops were also an opportunity to present the results of the work of our research group. At the same time, they provided a unique possibility for the education of younger researchers at our institute. I dare say that the workshops significantly contributed to the visibility of the Ljubljana research group in the global earthquake engineering community.

17.2 Other International Cooperation

International cooperation is a "conditio sine qua non" for researchers in earthquake engineering. Working in a small country distant from the main earthquake engineering centres, at our institute we have always been attempting to establish and maintain close contacts with colleagues all over the world, using both official and private channels. These efforts were mostly quite successful. We were able to establish extremely useful contacts with many researchers and with many of the most important research centres active in earthquake engineering and to participate in diverse international activities. All these contacts and activities greatly facilitated and enriched our research work.

Except for some international conferences, my first international experience was my 10 months stay in the period 1972-73, with a DAAD (Deutscher Akademischer Austauschdienst) scholarship, at the Ruhr University in Bochum (RUB), where I drafted my doctoral thesis (see Section 8.2). I worked at the Institut für konstruktiven Ingenieurbau. There were several reasons for choosing this university. First, at that time, for a person from the former Yugoslavia, it was not easy to organise research work in a foreign country. When, as a young researcher, I joined an excursion of students to West Germany, I met in Bochum Karl-Heinrich Schrader, professor at RUB. In discussion, we found some common interests, and he expressed his willingness to host me as a visiting researcher. The second reason was the language. I was fluent in German, whereas, unfortunately, my English was very poor. In the school, I learned several foreign languages, but not English. An early morning English course, which I attended on a voluntary basis, gave no visible results. Later, when I attended my first World Conference on Earthquake Engineering in Rome in 1973, I realised that without English one cannot survive in earthquake engineering, and I intensively started learning English, at the age of 30. Of course, it is difficult to make up for what was lost in the youth, and my modest English has certainly been a handicap for my career, especially in the case of oral discussions. The third reason for choosing Bochum was its proximity to Ljubljana (relatively, compared to overseas destinations). My wife was not enthusiastic about joining me in Germany with two small children. So, I stayed in Bochum alone, but I travelled home a few times. It was a very long and tiring drive over the Alps (there were no tunnels at that time), but a motivated young person was able to manage it in a day if needed.

For several reasons, Bochum was a very good place to work without too many outside temptations. I was able to fully concentrate on the work of my doctoral thesis. The topic was elastic static, dynamic, and stability analysis of multi-storey building structures with the emphasis on seismic issues, i.e., an extension of the work performed in my master's thesis. My host Schrader and colleagues working in his chair were involved in research of different problems in structural dynamics, including those related to nuclear power plants, but not in earthquake engineering. Nevertheless, discussions with them were very useful and interesting. They allowed me to see some problems from another point of view.

After my return from Bochum, there were not many international activities until 1978, when I managed to organise a trip to the USA and Canada. I visited the University of California in Berkeley, Stanford University, the University of Southern California in Los Angeles, Caltech in Pasadena, MIT in Cambridge, and McMaster University in Hamilton. The travel expenses were covered by the funds earned from consulting within our institute. As a person, unknown in the world, I dared to write to some of the most famous professors of that time (among them were George Housner and Ray Clough), asking them to accept me for a short visit. My letters were accompanied by a recommendation letter of Sergej Bubnov, the son of a high ranking Russian admiral who had emigrated to Yugoslavia. Bubnov lived in Ljubljana and was active in earthquake engineering, mostly as an excellent organiser with good international connections. He had a clear vision of the importance of earthquake engineering for Europe and was one of three persons responsible for the establishment of the European Association of Earthquake

Engineering. I was very surprised to receive positive answers from all of the addressees. In fact, all of them proved to be excellent hosts and spent some time with me during my visits. During the trip, I was able to confirm my belief that close ties with American researchers are crucial for successful research in earthquake engineering. This trip was the start of very fruitful cooperation with the individuals and also some of the institutions that I visited. My relations with US researchers strengthened during my stay in Berkeley in 1980, described in Section 17.1.

The other key country in earthquake engineering is Japan. In 1982, I had a chance to participate in a one-month individual programme sponsored by the Japanese International Cooperation Agency (JICA). Within this programme, in which everything was organised by the hosts, I visited several universities, research institutes, and construction companies. The highlight of my trip was the attendance of the workshop in the Building Research Institute at Tsukuba, organised within the US-Japan cooperative research programme utilising large-scale testing facilities, which allowed me to become acquainted with several Japanese and US colleagues. For the first time, I met Helmut Krawinkler and Masayoshi Nakashima. As a result of good personal relations with Japanese researchers, we were able to establish also official cooperation with the University of Tokyo, which resulted, inter alia, in a joint research project with the University of Tokyo (Shunsuke Otani was the principal investigator on the Japanese side) and in two joint workshops in Ljubljana (in 2000 and 2001).

I was surprised when the Turkish organisers of the 7th World Conference on Earthquake Engineering invited me to participate in the "State-of-the-Art panel on Earthquake Resistant Design", together with famous practising engineers and professors John Blume, Ferry Borges, Henry Degenkolb, Shunzo Okamoto, and Seki Shibata. Our report was published in a monograph (Özmen et al. 1981) together with other panel reports. The report was not optimistic regarding practical applications of inelastic dynamic analysis in the near future.

Membership in the Yugoslav Earthquake Engineering delegation to China gave me a unique opportunity to visit China in 1981. Compared to today's China, at that time, it was like another world. We visited Tsinghua University in Beijing and Tongji University in Shanghai, as well as the Institute of Engineering Mechanics in Harbin. These three institutions were considered to be the most advanced in earthquake engineering in China in early 1980s, and remain at the top nowadays. Five years after the disastrous 1976 earthquake, the visit to Tangshan was still a unique experience. Contacts established during the visit to Tsinghua University made it possible to carry out a bilateral research project with this university several years later (1989-1991).

It may seem strange, but our cooperation with European partners began, with few exceptions, such as a bilateral project with the Technical University of Darmstadt in 1990/91, relatively late. Although the Italian engineers were the first to introduce seismic analysis after the 1908 Messina earthquake, the development of earthquake engineering in Europe was much slower than in more seismically active regions (i.e., California and Japan). Research in Europe intensified at the beginning of the 1990s, as a consequence of financial support from the European Commission within the Framework Programmes. A consortium of large-scale European facilities in earthquake engineering, coordinated by Roy Severn from the University of Bristol, was established; this consortium was the core of a research network that has been continuously involved in European research projects in the field of earthquake engineering. Our research team joined the network as soon as Slovenia became eligible for EU funding. We started in 1993-96 with a COST (European Cooperation in Science and Technology) project. In 1997-2000 we were, with two projects, involved in the INCO-COPERNICUS programme, intended for scientific cooperation of the EU countries with the Central and Eastern European countries and the new independent states. From the year 2000, we have been regularly included in the Framework Programmes created by the European Union/European Commission to support and foster research in the European Research Area. These activities contributed to the funding of our research work (in addition to national funds), facilitated cooperation with practically all major earthquake engineering centres in Europe, and supported young researchers from abroad to come for several months to Ljubljana. Most of them were from the University of Naples Federico II in Italy.

A strong earthquake in an urban area is a trove of information for everybody working in earthquake engineering. Post-earthquake field investigations are the best school for engineers. Although it is not easy to organise such field trips, with my colleagues we succeeded in visiting several of the areas hit by strong earthquakes. My first field investigation was in 1979 in Montenegro, at that time a part of the former Yugoslavia. My colleagues and I formed an official team supervising the work of local teams surveying the damage. After three weeks of fieldwork, we also numerically analysed several buildings and prepared an English (Fajfar 1981) and a Slovenian version of the report. Later, we had the opportunity to see, soon after the earthquakes, consequences of the Mexico City (1985), Kobe (1995), Izmit (1999), L'Aquila (2009), Chile (2010), and Emilia Romagna (2012) earthquakes, and to learn from them. The visit to the area of the 2011 Tohoku earthquake happened with a delay of one year, but it was still possible to observe some damage due to this disastrous event.

A precious experience, which certainly influenced my work, were visiting professorships at four high-ranking universities: McMaster University, Hamilton, Canada, 1994 (Hooker Distinguished Visiting Professorship, host Ahmed Ghobarah), Stanford University, USA, 1995 (Shimizu Visiting Professorship, host Helmut Krawinkler), Bristol University, UK, 2006 (host Roy Severn), University of Canterbury, Christchurch, 2009 (Visiting Erskine Fellowship, host Rajesh Dhakal). At Stanford, I was involved only in research, whereas at the other three places I held a course.

Twice I was involved in international teams writing a book related to earthquake engineering. In 1990, just before the former Yugoslavia disintegrated, a group of five authors from Croatia, Serbia, and Slovenia prepared a book of about 650 pages, entitled “Earthquake Engineering – Buildings”. It was written in the “Serbo-Croatian” language, as it was called at that time. More recently, four authors from different countries wrote the book “Seismic design of concrete buildings to Eurocode 8” (Fardis et al. 2015). In both books, my contributions were related to analysis, with the N2 method included. Frankly, in both cases, I was somewhat disappointed with the final outcome. I realised that it is extremely difficult to write a book if the contributions of several co-authors are closely related and interdependent, even if the co-authors are good friends. It is practically impossible to achieve a consistency of different parts, prepared by different authors. Also, in both cases it turned out that it would not be possible to complete the work if, in the end, one of the authors had not taken the lead.

Two other international activities that had an impact on my research activities were the work related to the European standard Eurocode 8 (EC8) and the editorship of the journal Earthquake Engineering and Structural Dynamics (EESD).

Slovenia was the first country, for professional and political reasons, to adopt the European standard for seismic design EC8. I have been a member of technical subcommittee CEN/TC 250/SC 8, responsible for EC8, from 1994 to 2019. Also, I was in charge of the implementation of EC8 in Slovenia. Participation at the meetings, attended by representatives of member countries, and studying the comments of national committees, gave me an opportunity to better understand the ways of thinking in different countries and their needs. I realised how difficult it is to draft Eurocodes, which are intended for application in all member countries and beyond. In the case of EC8, for example, the views of countries with low seismicity are often quite different from those in the countries with a greater seismic hazard. Due to the coordination necessary between different countries and between different Eurocodes, the process of implementing the European standards is exceptionally long. The result is usually a compromise between different views. The work related to standards and codes is demanding and often not very pleasant. However, it is essential. As Cornell and Krawinkler (2000) wrote, “The final challenge for . . . researchers is not in predicting performance or in estimation losses, but in contributing effectively to the reduction of losses and the improvement of safety”.

A valuable experience was my editorial work for the journal Earthquake Engineering and Structural Dynamics (EESD). I have always considered EESD to be the prime journal in earthquake engineering, so I submitted most of my papers to it. I assume that the invitation for joining the editorial board, which I received in 1996 from the editor (later executive editor) Anil Chopra came as a result of my several papers published in EESD, as well as of my peer reviewing activity. Later, in 2002, I became an associate editor (later an editor). Most of the time, the team of three editors consisted of Anil Chopra, Masayoshi Nakashima, and myself. It was a great pleasure and privilege to work with Anil and Masayoshi. Although we were considered to be representatives of three geographical regions (Americas, Asia and Europe), the authors were free to submit their papers to the editor of their choice. Each of the three editors was completely independent in the processing of the papers and decision making. Of course, the primary influence on the policy of the journal, which has always been oriented toward publishing high quality papers, has had the executive editor. He was responsible also for all business with the publisher (Wiley) and with IAEE. There is no doubt that the high reputation of EESD is mainly a result of the devoted work and skilful leadership of Anil Chopra.

I retired as the EESD editor at the end of 2015. The amount of work was considerable from the beginning to the end. When I started in 2002, regular mail service was still used. Later, everything changed to electronic communication which facilitated the processing of the manuscripts. However, the number of submissions has been steadily increasing. A considerable amount of manuscripts, some of them were of questionable quality, have been arriving. In the last year of my service, I processed 164 papers. The major problem was to find qualified reviewers. Sometimes, a reviewer first accepted, but then, after a considerable time, cancelled or did not submit the review in spite of many reminders. Although the processing of the manuscripts became highly automated, I was still keeping some direct contacts with reviewers by sending some personal messages. This attitude has been generally well received, possibly motivated some reviewers, and facilitated the review process to some extent. After 13 years of service, the editorial work was not as challenging and enjoyable as at the beginning, so I decided to retire. Fortunately, it was possible to find an excellent replacement. Michael Fardis agreed to become the new editor.

REFERENCES

SELECTED REFERENCES OF THE UNIVERSITY OF LJUBLJANA RESEARCH GROUP (IN CHRONOLOGICAL ORDER)

Fajfar P (1973) Beitrag zur dynamischen Berechnung von Hochhäusern. Die Bautechnik 50(5): 175-177.

Fajfar P (1975) Numerical analysis of multistory structures, Proceedings of the 5th European Conference on Earthquake Engineering, Istanbul; Paper 79.

Fajfar P (1976) EAVEK: Program for elastic analysis of multistorey structures. Report RC/IKPIR No.13. 1.edition (in Slovenian). University of Ljubljana. (English Edition 1987).

Fajfar P (1978) Numerisches Verfahren zur statischen, dynamischen und Stabilitätsberechnung von Hochhäusern. Die Bautechnik 55(2):37-42.

Fajfar P, Duhovnik J, Reflak J, Fischinger M, Breška Z (1981) The behaviour of buildings and other structures during the earthquakes of 1979 in Montenegro. Report IKPIR No. 19 A. University of Ljubljana.

Özmen G, Blume JA, Borges JF, Degenkolb HJ, Fajfar P, Okamoto S, Shibata H (1981) State-of-the-art panel report on earthquake resistant design. In: Ergünay O and Erdik M (eds.). State-of-the-art in earthquake engineering 1981. Turkish National Committee on Earthquake Engineering: Istanbul; 283-294.

Fischinger M, Fajfar P (1982) Inelastic spectra of some earthquakes recorded in Yugoslavia. Proceedings of the 7th European Conference on Earthquake Engineering, Athens; Vol.3:53-60.

Fajfar P, Fischinger M (1984) Parametric study of inelastic response to some earthquakes recorded in Southern Europe. Proceedings of the 8th World Conference on Earthquake Engineering, San Francisco. Prentice-Hall; Vol.4:75-82.

Fajfar P, Fischinger M (1987) Non-linear seismic analysis of RC buildings: Implications of a case study. European Earthquake Engineering 1(1):31-43.

Fajfar P, Fischinger M (1989) Study of the characteristics of earthquakes in Southern Europe, IAEA Meeting on Earthquake Ground Motion and Seismic Evaluation of Nuclear Power Plants, Moscow, 1986. International Atomic Energy Agency: Vienna. Publication IAEA-TC-472.2; Vol.1:572-582.

Fajfar P, Fischinger M. (1989) N2 – A method for non-linear seismic analysis of regular buildings. Proceedings of the 9th World Conference on Earthquake Engineering, Tokyo, Kyoto. Maruzen: Tokyo; Vol. V:111-116.

Fajfar P, Vidic T, Fischinger M (1989) Seismic demand in medium- and long-period structures. Earthquake Engineering & Structural Dynamics 18(8):1133-1144.

Fajfar P, Fischinger M (1990) A seismic procedure including energy concept. Proceedings of the 9th European Conference on Earthquake Engineering, Moscow; 312–321.

Fajfar P, Vidic T, Fischinger M (1990) A measure of earthquake motion capacity to damage medium-period structures. Soil Dynamics and Earthquake Engineering 9(5):236-242.

Fischinger M., Fajfar P (1990) On the response modification factors for reinforced concrete buildings. Proceedings of the 4th U.S. National Conference on Earthquake Engineering, Palm Springs. EERI: California; Vol.2:249-258.

Reflak J, Fajfar P (1991) Elastic analysis of infilled frames using substructures. Proceedings of the 6th Canadian Conference on Earthquake Engineering, Toronto. University of Toronto Press; 285-292.

Fajfar P (1992) Equivalent ductility factors, taking into account low-cycle fatigue. Earthquake Engineering & Structural Dynamics 21(10):837-848.

Fajfar P, Krawinkler H (eds.) (1992) Nonlinear seismic analysis and design of reinforced concrete buildings. Elsevier Applied Science: London.

Krawinkler H, Fajfar P (eds) (1992) Nonlinear seismic analysis and design of reinforced concrete buildings. Supplementary proceedings of a Workshop held in Bled, Slovenia, July 13-16, 1992. The John A. Blume Earthquake Engineering Center, Stanford University in conjunction with the Institute for Structural and Earthquake Engineering, University of Ljubljana. Report No 103.

Fajfar P, Vidic T (1994) Consistent inelastic design spectra: hysteretic and input energy. Earthquake Engineering & Structural Dynamics 23(5):523-537.

Fischinger M., Fajfar P (1994) Seismic force reduction factors. In: Rutenberg A (ed.). Earthquake Engineering, Proceedings of the 7th Regional European Seminar on Earthquake Engineering, Haifa. Balkema; 279-296.

Peruš I, Fajfar P, Grabec I (1994) Prediction of the seismic capacity of rc structural walls by non-parametric multidimensional regression. Earthquake Engineering & Structural Dynamics 23(10): 1139-1155.

Novak D, Fajfar P (1994) Nonlinear floor response spectra for rational aseismic design of equipment (in Slovenian). Proceedings of the 16th Slovenian Conference of Structural Engineers, Ljubljana; 95-102.

Vidic T, Fajfar P, Fischinger M (1994) Consistent inelastic design spectra: strength and displacement. Earthquake Engineering & Structural Dynamics 23(5):507-521.

Fajfar P (1995a) Elastic and inelastic response spectra (State-of-the-art paper). Proceedings of the 10th European Conference on Earthquake Engineering, Vienna. A.A.Balkema: Rotterdam, Brookfield: 1169-1178.

Fajfar P (1995b) Design spectra for the new generation of codes: Eurocode 8 achieves the half-way mark. Proceedings of the 10th European Conference on Earthquake Engineering, Vienna. A.A.Balkema: Rotterdam, Brookfield: 2969-2974.

Fajfar P, Novak D (1995) Floor response spectra for inelastic structures. Transactions of the 13th International Conference on Structural Mechanics in Reactor Technology (SMiRT 13), Porto Alegre, Brazil. Universidade Federal do Rio Grande do Sul: Porto Alegre; paper no. K044/1:259-264.

Fajfar P. (1996) Design spectra for the new generation of codes. Proceedings of the 11th World Conference on Earthquake Engineering, Acapulco; paper No.2127.

Fajfar P (ed.) (1996) Towards a new seismic design methodology for buildings: research at the University of Ljubljana. Faculty of Civil Engineering and Geodesy, Institute of Structural and Earthquake Engineering: Ljubljana.

Fajfar P, Gašperšič P (1996) The N2 method for the seismic damage analysis of RC buildings. Earthquake Engineering & Structural Dynamics 25(1):31-46.

Kilar V, Fajfar P. (1997) Simple push-over analysis of asymmetric buildings. Earthquake Engineering & Structural Dynamics 26(2): 233-249.

Fajfar P, Krawinkler H (eds.) (1997) Seismic design methodologies for the next generation of codes. Proceedings of the International Workshop on Seismic Design Methodologies for the Next Generation of Codes, Bled, Slovenia, 24-27 June 1997. A. A. Balkema: Rotterdam, Brookfield.

Fajfar P, Gašperšič P, Drobnič D (1997) A simplified nonlinear method for seismic damage analysis of structures. In: Fajfar P, Krawinkler H (eds.) (1997); 183-193.

Fajfar P, Pauley T (1997) Notes on definitions of overstrength factors. In: Fajfar P, Krawinkler H (eds.)

(1997); 407-409.

Fajfar P (1998) Capacity spectrum method based on inelastic demand spectra. IKPIR Report No. EE – 3/98. University of Ljubljana.

Chai YH, Fajfar P, Romstad KM (1998) Formulation of duration-dependent inelastic seismic design spectrum. Journal of Structural Engineering ASCE 124(8):913-921.

Fajfar P, Gašperšič P (1998) A simplified non-linear method for seismic evaluation of RC bridges, Proceedings of the 6th U.S. National Conference on Earthquake Engineering, Seattle, WA.

Fajfar P (1999) Capacity spectrum method based on inelastic demand spectra, Earthquake Engineering & Structural Dynamics 28(9):979-993.

Chai YH, Fajfar P (2000) A procedure for estimating input energy spectra for seismic design. Journal of Earthquake Engineering 4(4):539-561.

Fajfar P (2000) A nonlinear analysis method for performance-based seismic design. Earthquake Spectra 16(3):573-592.

Dolšek M, Fajfar P (2001) Soft storey effects in uniformly infilled reinforced concrete frames. Journal of Earthquake Engineering 5(1):1-12

Fajfar P (2002) Structural analysis in earthquake engineering - a breakthrough of simplified non-linear methods. Proceedings of the 12th European Conference on Earthquake Engineering, London. Keynote lecture. Elsevier: Amsterdam [etc.]; paper 843.

Cuesta I, Aschheim MA., Fajfar P (2003) Simplified R-factor relationships for strong ground motions. Earthquake Spectra 19(1):25-45.

Isaković T, Fischinger M, Kante P (2003) Bridges: when is single mode seismic analysis adequate?. Proceedings of the Institution of Civil Engineers - Structures and Buildings 156(2): 165-173.

Fajfar P, Krawinkler H (eds.) (2004) Performance-based seismic design: concepts and implementation. Proceedings of the International Workshop, Bled, Slovenia, 28 June - 1 July 2004, PEER report 2004/05. Pacific Earthquake Engineering Research Center: Berkeley, CA.

Dolšek M, Fajfar P (2004a) Inelastic spectra for infilled reinforced concrete frames. Earthquake Engineering & Structural Dynamics 33(15):1395-1416.

Dolšek M, Fajfar P (2004b) IN2 - a simple alternative for IDA. Proceedings of the 13th World Conference on Earthquake Engineering, Vancouver. Canadian Association for Earthquake Engineering, International Association Earthquake Engineering; Paper No. 3353.

Dolšek M, Fajfar P (2005) Simplified non-linear seismic analysis of infilled reinforced concrete frames. Earthquake Engineering & Structural Dynamics 34(1):49-66.

Fajfar P, Marušić D, Peruš I (2005) Torsional effects in the pushover-based seismic analysis of buildings. Journal of Earthquake Engineering 9(6): 831-854.

Marušić D, Fajfar P (2005) On the inelastic seismic response of asymmetric buildings under bi-axial excitation. Earthquake Engineering & Structural Dynamics 34(8):943-963.

Peruš I, Fajfar P (2005) On the inelastic torsional response of single-storey structures under bi-axial excitation. Earthquake Engineering & Structural Dynamics 34(8):931-941.

Fajfar P, Dolšek M, Marušić D, Stratan A (2006) Pre-and post-test mathematical modelling of a plan-asymmetric reinforced concrete frame building. Earthquake Engineering & Structural Dynamics 35(11):1359-1379.

Isaković T, Fischinger M (2006) Higher modes in simplified inelastic seismic analysis of single column bent viaducts. Earthquake Engineering & Structural Dynamics 35(1): 95-114.

Peruš I, Poljanšek K, Fajfar P (2006) Flexural deformation capacity of rectangular RC columns determined by the CAE method. Earthquake Engineering & Structural Dynamics 35(12):1453-1470.

Dolšek M, Fajfar P (2007) Simplified probabilistic seismic performance assessment of plan asymmetric buildings. Earthquake Engineering & Structural Dynamics 36(13):2021-2041.

Peruš I, Fajfar P (2007) Prediction of the force - drift envelope for RC columns in flexure by CAE method. Earthquake Engineering & Structural Dynamics 36(15):2345-2363.

Dolšek M, Fajfar P (2008a) The effects of masonry infills on the seismic response of a four-storey reinforced concrete frame - a deterministic assessment. Engineering Structures 30(7):1991-2001.

Dolšek M, Fajfar P (2008b) The effects of masonry infills on the seismic response of a four-storey reinforced concrete frame - a probabilistic assessment. Engineering Structures 30(11):3186-3192.

Fajfar P, Marušić D, Peruš I, Kreslin M (2008) The N2 method for asymmetric buildings. In: Bento R, Pinho R (eds). 3Dpushover 2008 - Nonlinear static methods for design/assessment of 3D structures. IST press: Lisbon; 85-98.

Isaković T, Popeyo Lazaro MN, Fischinger M (2008) Applicability of pushover methods for the seismic analysis of single-column bent viaducts. Earthquake Engineering & Structural Dynamics 37(8):1185-1202.

Poljanšek K, Peruš I, Fajfar P (2009) Hysteretic energy dissipation capacity and the cyclic to monotonic drift ratio for rectangular RC columns in flexure. Earthquake Engineering & Structural Dynamics 38(7):907-928.

Rozman M, Fajfar P (2009) Seismic response of a RC frame building designed according to old and modern practices. Bulletin of Earthquake Engineering 7(3):779–799.

Kreslin M, Fajfar P (2010). Seismic evaluation of an existing complex RC building. Bulletin of earthquake engineering 8(2):363-385.

Isaković T, Fischinger M (2011) Applicability of Pushover Methods to the Seismic Analyses of an RC Bridge, Experimentally Tested on Tree Shake Tables. Journal of Earthquake Engineering 15(2):303-320.

Kreslin M, Fajfar P (2011). The extended N2 method taking into account higher mode effects in elevation. Earthquake Engineering & Structural Dynamics 40(14):1571-1589.

Fajfar P, Dolšek M (2012) A practice-oriented estimation of the failure probability of building structures. Earthquake Engineering & Structural Dynamics 41(3): 531-547.

Kreslin M, Fajfar P (2012) The extended N2 method considering higher mode effects in both plan and elevation. Bulletin of Earthquake Engineering 10(2):695-715.

Fajfar P, Dolšek M. (2014) Pushover-based analysis in performance-based seismic engineering – A view from Europe. Chapter 19 in Fischinger (ed.) (2014): 265-277.

Fajfar P, Polič M, Klinc R (2014) Perception of seismic risk by experts and lay people (in Slovenian). Gradbeni vestnik 63(5):111-118.

Fischinger M (ed.) (2014) Performance-based seismic engineering: vision for an earthquake resilient society. Geotechnical, Geological, and Earthquake Engineering 32. Springer: Dordrecht [etc.].

Kosič M, Fajfar P, Dolšek M (2014) Approximate seismic risk assessment of building structures with explicit consideration of uncertainties. Earthquake Engineering & Structural Dynamics 43(10):1483-1502.

Fardis MN, Carvalho EC, Fajfar P, Pecker A (2015) Seismic design of concrete buildings to Eurocode 8. CRC Press: Boca Raton, FL.

Vukobratović V, Fajfar P (2015) A method for the direct determination of approximate floor response spectra for SDOF inelastic structures. Bulletin of Earthquake Engineering 13(5):1405–1424.

Klinc R, Peruš I, Dolenc M, Fajfar P (2016) Online version of the program EAVEK (in Slovenian), Gradbeni vestnik 65(1):10-17.

Kosič M, Dolšek M, Fajfar P (2016) Dispersions for pushover-based risk assessment of reinforced concrete frames and cantilever walls. Earthquake Engineering & Structural Dynamics 45(13):2163-2183.

Sinkovič K, Peruš I, Fajfar P (2016) Assessment of the seismic performance of low-rise RC structures by procedures with different levels of complexity. Bulletin of earthquake engineering 14(1):213-239.

Vukobratović V, Fajfar P (2016) A method for the direct estimation of floor acceleration spectra for elastic and inelastic MDOF structures. Earthquake Engineering & Structural Dynamics 45(15):2495–2511.

Dolšek M, Lazar Sinković N, Žižmond J (2017) IM-based and EDP-based decision models for the verification of the seismic collapse safety of buildings. Earthquake Engineering & Structural Dynamics 36(15):2665-2682.

Kosič M, Dolšek M, Fajfar P (2017) Pushover-based risk assessment method: a practical tool for risk assessment of building structures. Proceedings of the 16th World Conference on Earthquake Engineering, Santiago.

Vukobratović V, Fajfar P (2017) Code-oriented floor acceleration spectra for building structures. Bulletin of Earthquake Engineering 15(7):3013-3026.

Žižmond J, Dolšek M (2017) The formulation of risk-targeted behaviour factor and its application to reinforced concrete buildings. Proceedings of the 16th World Conference on Earthquake Engineering, Santiago; Paper 1659.

Fajfar P (2018) Analysis in seismic provisions for buildings - past, present and future. The fifth Prof. Nicholas Ambraseys lecture. Bulletin of Earthquake Engineering 16(7):2567-2608.

Žižmond J, Dolšek M (2019) The formulation of risk-targeted seismic action for the force-based seismic design of structures. Earthquake Engineering & Structural Dynamics 48(12):1406-1428.

OTHER REFERENCES (IN ALPHABETICAL ORDER)

Abrahamson N (2018) What changes to expect in seismic hazard analyses in the next 5 years? 11th U.S. National Conference on Earthquake Engineering, Los Angeles.

Akiyama H (1985) Earthquake-Resistant Limit-State Design for Buildings. University of Tokyo Press: Tokyo.

Army (1986) Seismic design guidelines for essential buildings. Departments of the Army (TM 5-809-10-1), Navy (NAVFAC P355.1), and the Air Force (AFM 88-3, Chapter 13, Section A). Washington, D.C.

ASCE (2014) Seismic evaluation and retrofit of existing buildings, ASCE/SEI 41-13. American Society of Civil Engineers: Reston, VA.

ASCE (2017) Minimum design loads and associated criteria for buildings and other structures, ASCE/SEI 7-16. American Society of Civil Engineers: Reston, VA.

ATC (1978) Tentative provisions for the development of seismic regulations for buildings, ATC 3-06. National Bureau of Standards: Washington D.C.

ATC (1995) Structural response modification factors, ATC-19. Applied Technology Council: Redwood City, CA.

ATC (1996) Seismic evaluation and retrofit of concrete buildings (Report SSC 96-01 of California Seismic Safety Commission), Report ATC-40. Applied Technology Council: Redwood City, CA.

Aydinoğlu MN, Önem G (2010) Evaluation of analysis procedures for seismic assessment and retrofit design. In: Garevski M, Ansal A (eds.) Earthquake engineering in Europe. Geotechnical, Geological and Earthquake Engineering 17. Springer, Dordrecht; 171-198.

Baros DK, Anagnostopoulos SA (2008) An assessment of static non-linear pushover analyses in 2-D and 3-D applications. In: Bento R, Pinho R (eds). 3Dpushover 2008 - Nonlinear static methods for design/assessment of 3D structures. IST press: Lisbon.

Bertero VV (1986) Evaluation of response reduction factors recommended by ATC and SEAOC. Proceedings of the 3rd U.S. National Conference on Earthquake Engineering, Charlston. EERI; vol.III:1663-1674.

Bertero RD, Bertero VV (1992) Tall reinforced concrete buildings: conceptual earthquake-resistant design methodology. Report No. UCB/EERC-92/16. Univ. of California: Berkeley, CA.

Bertero VV (1995) Tri-service manual methods. In Vision 2000, Part 2, Appendix J. Structural Engineers Association of California. Sacramento, CA.

CEN (2004) European standard EN 1998–1:2004, Eurocode 8: Design of structures for earthquake resistance, Part 1: General rules, seismic action and rules for buildings. European Committee for Standardization: Brussels.

CEN (2005) European standard EN 1998–3:2005, Eurocode 8: Design of structures for earthquake resistance, Part 3: Assessment and retrofitting of buildings. European Committee for Standardization: Brussels.

CEN (2019a) European standard EN 1998–1-1:2019 (draft), Eurocode 8: Design of structures for earthquake resistance, Part 1-1: General rules and seismic action. European Committee for Standardization: Brussels.

CEN (2019b) Annex E (Informative) Simplified reliability-based verification format. Annex to CEN (2019a). European Committee for Standardization: Brussels. (Originally prepared by Dolšek M, Žižmond J, Kosič M, Lazar Sinković N).

CEN (2019c) European standard EN 1998–1-2:2019.3 (draft), Eurocode 8: Design of structures for earthquake resistance, Part 1-2: Rules for new buildings. European Committee for Standardization: Brussels.

Chopra AK (1995) Dynamics of Structures: Theory and applications to earthquake engineering, First edition. Prentice-Hall. (5th edition, 2017. Pearson: Hoboken, NJ).

Chopra AK, Goel RK (1999a) Capacity-demand-diagram methods for estimating seismic deformation of inelastic structures: SDF systems. Report PEER-1999/02. Pacific Earthquake Engineering Research Center, University of California: Berkeley, CA.

Chopra AK, Goel RK (1999b) Capacity-demand-diagram methods based on inelastic design spectrum. Earthquake Spectra 15(4):637-656.

Chopra AK, Goel RK (2000) Evaluation of NSP to estimate seismic deformation: SDF systems, Journal of Structural Engineering ASCE 126(4):482-490.

Clough RW, Penzien J (1975) Dynamics of structures. McGraw-Hill: New York [etc].

Clough R (1980) The finite element method after twenty-five years: A personal view. Computers and Structures 12(4): 361–370.

Cornell CA (1996) Calculating building seismic performance reliability: a basis for multi-level design norms. Proceedings of the 11th World Conference on Earthquake Engineering, Mexico City; Paper No.2122.

Cornell CA, Jalayar F, Hamburger RO, Foutch DA (2002) Probabilistic basis for 2000 SAC Federal Emergency Management Agency steel moment frame guidelines. Journal of Structural Engineering ASCE 128 (4):526-533.

Cornell CA, Krawinkler H (2000) Progress and challenges in seismic performance assessment. PEER Center news 3(2):1-3. Pacific Earthquake Engineering Center: Berkeley, CA.

FEMA (1997) NEHRP guidelines for the seismic rehabilitation of buildings, FEMA 273. Federal Emergency Management Agency: Washington, D.C.

FEMA (2000) Prestandard and commentary for the seismic rehabilitation of buildings, FEMA 356. Federal Emergency Management Agency: Washington, D.C.

FEMA (2009) Quantification of Building Seismic Performance Factors, FEMA P-695. Federal Emergency Management Agency: Washington, D.C.

FEMA (2012) Reducing the Risks of Nonstructural Earthquake Damage – A Practical Guide. FEMA E-74. Federal Emergency Management Agency: Washington, D.C.

Fenves SJ, Logcher RD, Mauch S P (1964) STRESS-A User's Manual. MIT Press: Cambridge, MA.

fib (2012) Probabilistic performance-based seismic design. Technical report prepared by Task Group 7.7. International Federation for Structural Concrete: Lausanne, Switzerland.

Freeman SA, Nicoletti JP, Tyrell JV (1975) Evaluations of existing buildings for seismic risk - A case study of Puget Sound naval shipyard, Bremerton, Washington. Proceedings of the U.S. National Conference on Earthquake Engineering, Berkeley, CA; 113-122.

Grabec I, Sachse W (1997) Synergetics of Measurement, Prediction and Control. Springer: Berlin.

Gülkan P, Reitherman R (2012) The IAEE at fifty: A brief history of the International Association for Earthquake Engineering. IAEE.

Hanson RD, Reitherman R (Interviewers) (2015) William J.Hall, Connections. The EERI Oral History Series. Earthquake Engineering Research Institute: Oakland, CA.

Housner GW (1956) Limit design of structures to resist earthquakes, Proceedings of the 1st World Conference on Earthquake Engineering. EERI: Berkeley, CA.

Krawinkler H (1995) New trends in seismic design methodology. Proceedings of the 10th European Conference on Earthquake Engineering, Vienna. Balkema: Rotterdam; Vol. 2:821-830.

Krawinkler H (2006) Importance of good nonlinear analysis. The Structural Design of Tall and Special Buildings 15(5):515-531.

Krawinkler H, Seneviratna GDPK (1998) Pros and cons of a pushover analysis for seismic performance evaluation. Engineering Structures 20(4-6):452-464.

Mahaney JA, Paret TF, Kehoe BE, Freeman SA (1993) The capacity spectrum method for evaluating structural response during the Loma Prieta Earthquake. Proceedings of 1993 National Earthquake Conference: 501-510. Earthquake Engineering Research Institute: Oakland, CA.

Mazzolani FM, Piluso V (1996) Theory and Design of Seismic Resistant Steel Frames. E&FN Spon: London.

NZSEE (2017) The seismic assessment of existing buildings. Technical guidelines for engineering assessments, Part A: Assessment objectives and principles. New Zealand Society for Earthquake Engineering [etc.].

Park YJ, Ang AHS, Wen YK (1984) Seismic damage analysis and damage- limiting design of RC buildings, Structural Research Series No. 516, University of Illinois: Urbana, IL.

Park YJ, Ang AHS, Wen YK (1987) Damage-limiting aseismic design of buildings. Earthquake Spectra 3(1):1-26.

Regione Autonoma Friuli-Venezia Giulia (1977) Raccomandazioni per la riparazione strutturale degli edifici in muratura. DT 2, Udine, Italy.

Reinhorn AM (1997) Inelastic analysis techniques in seismic evaluations. In: P. Fajfar P, Krawinkler H. (eds) (1997); 277-287.

Reitherman R (interviewer) (2009) Vitelmo V. Bertero. Connections. The EERI oral history series. Earthquake Engineering Research Institute: Oakland, CA.

Saiidi M, Sozen MA (1981) Simple nonlinear seismic analysis of R/C structures. Journal of the Structural Division ASCE 107(5):937-952.

SEAOC (1995) Vision 2000, Performance based seismic engineering of buildings. Structural Engineers Association of California: Sacramento, CA.

Sullivan TJ, Calvi PM, Nascimbene R (2013) Towards improved floor spectra estimates for seismic design. Earthquakes and Structures 4(1):109–132.

Terzic V, Schoettler MJ, Restrepo JI, Mahin SA (2015) Concrete column blind prediction contest 2010: Outcomes and observations. PEER Report 2015/01. Pacific Earthquake Engineering Research

Center: Berkeley, CA.

Tomaževič M (1978) Improvement of computer program POR (in Slovenian). Report ZRMK-IK. Institute for Testing and Research in Materials and Structures: Ljubljana, Slovenia.

Uang C-M (1991) Establishing R (or R_W) and C_d factors for building seismic provisions. Journal of Structural Engineering ASCE 117(1):19-28.

Uang C-M, Bertero VV (1988) Use of energy as a design criterion in earthquake resistant design. Report UCB/EERC-88/18. Earthquake Engineering Research Center, University of California: Berkeley, CA.

Vamvatsikos D, Cornell CA (2002) Incremental dynamic analysis. Earthquake Engineering & Structural Dynamics 31:491–514.

Veletsos A.S, Newmark N.M (1960) Effect of inelastic behavior on the response of simple systems to earthquake motions. Proceedings of the 2nd World Conference on Earthquake Engineering, Tokyo.

Winokur A, Glück J (1968), Lateral loads in asymmetric multistory structures, Journal of Structural Division, ASCE, 94(3):645-656.

Yasui Y, Yoshihara J, Takeda T, Miyamoto A (1993) Direct generation method for floor response spectra. In: Kussmaul K (ed.). Transactions of the 12th International Conference on Structural Mechanics in Reactor Technology (SMiRT 12). Elsevier Science Publishers B.V.: Stuttgart, Germany: Paper K13/4; 367–372.

ACKNOWLEDGEMENTS

Nobody has been more important to me than my family, my loving and supportive wife Metka, and my two wonderful children: my son Iztok and my daughter Jasna. They often suffered due to my physical and/or mental absence. Metka had a degree in veterinary medicine, but she was never employed. She took the best care of our children and home. Mostly due to her, we have had a peaceful and well-ordered family life, and I was able to spend much (maybe too much if I am now looking back) of my energy and time for work. Unfortunately, Metka passed away after 49 years of our marriage. When, shortly before she left me, I told her that I was going to receive the highest national award for scientific work, she modestly mentioned that she also deserved some credit for it. She was right. Without her, everything would be quite different. Iztok and Jasna live with their families very near to me, and we have very close relationships. My grandson Erik and granddaughter Zarja bring much joy to our lives. I would also like to express my very profound gratitude to my parents and grandparents for their love, support and care, and for providing me with a good education.

In engineering, it is not possible to work alone, as it may be in mathematics and philosophy. The results described in this monograph could not have been obtained without funding from national and some foreign agencies, and the dedicated work of all my colleagues and students. Some of them are specifically mentioned throughout the text of this book. Here, however, I would like to point out four individuals, good friends, with whom I had a pleasure of working almost all my professional life, who, in different ways, influenced my career and contributed to the results achieved.

In 1971, a special unit, called the "Computing Centre", was established at the Faculty of Architecture, Civil and Geodetic Engineering (later Faculty of Civil and Geodetic Engineering, referred to in this book as the "faculty"). It was in this unit that I got my first regular job at the university. The team was composed of young people who were breaking new ground in the field of computer applications in structural engineering. In 1980, the Computing Centre became the Institute for Structural and Earthquake Engineering and Construction IT (IKPIR) (referred to in this book as the "institute"). I worked there until I retired.

The driving force behind the Computing Centre and the institute was Janez Reflak, who was the head of the centre and, most of the time, until he retired, also of the institute. Janez, my best friend at the faculty, has strongly influenced my career. We worked together from my first day at the faculty as a young researcher. Even now, when both of us are retired, we regularly meet at the faculty for a

morning coffee. As a person with exceptional interpersonal skills, Janez was an outstanding leader. The atmosphere in the institute was like that of a family, and it was a great pleasure to work in such an environment. We were at the right moment at the right place to do some pioneering work in the implementation of emerging computational methods in structural analysis in Slovenia. In addition to teaching and research activities, we were heavily involved in consulting. Cooperation with practising engineers was highly beneficial for both sides. It helped us to recognise the real problems in practice and to adapt our teaching and research accordingly. Moreover, even though we lived in a socialist system, Janez managed to establish a highly effective organisational system, which allowed the institute to retain the majority of the income from consulting, which has been used for supporting our international cooperation and also to provide some extra payment to the individuals involved in the consulting activities.

Matej Fischinger was the first person I worked with in the field of earthquake engineering. As a young assistant professor, I invited him to help me in research activities when he was still an undergraduate student, and we have enjoyed excellent collaboration ever since then. Matej was one of the first students to write his bachelor thesis with me in 1976; he was my first master's student, and one of my first doctoral students. The topic of both master's and doctoral theses was the nonlinear analysis of reinforced concrete structures. He made his career at the faculty and soon became a full professor. At the beginning, we did most of the work together and spent long hours using inexpensive computer time during nights and weekends. Together, we wrote the initial papers on the N2 method. In 1988, we received, as a team of two, a national award for research achievements in nonlinear analysis of RC structures. Later, Matej's research interest shifted more to design of RC structures, for which he mostly worked with Tatjana Isaković. Matej is known for his sound engineering thinking. He relies mostly on conceptual design and detailing, and he often doubts in results of sophisticated analyses, which may sometimes be contrary to common sense. He has always had original ideas. Although we might not agree on everything, his ideas have very often made me think about a problem from another point of view. His writing and lecturing abilities are excellent. In addition to all his professional qualities, he is well known as a devoted family man, an excellent cook, a world traveller and a photographer. Famous are his handmade calendars containing photos taken during his travels.

Helmut Krawinkler has had a profound impact on my career. We met for the first time in 1982 during a US-Japan workshop in Tsukuba in Japan. This was the start of a beautiful friendship. Helmut's home country was Austria, a neighbour of Slovenia. His mother came from Carinthia, a region at the border with Slovenia, where a significant part of the population is of Slovenian origin. Every summer, Helmut visited his home country and also used this opportunity to visit me in Slovenia. We have had the same cultural background, very similar views on engineering problems and very similar research interests. Of course, as a professor at Stanford, Helmut had the privilege of being part of one of the most prestigious universities hiring the best students from all around the world. By the way, he often wondered how we were able, in small Slovenia, to find such excellent domestic students, comparable to those in Stanford. Helmut opened the door wide to Stanford to my colleagues and me, and to all the results produced in the Pacific Earthquake Engineering (PEER) centre. The Bled workshops, which we organised together, were a unique experience. Unfortunately, Helmut passed away prematurely in 2012, full of energy and ideas. After that, I still visited Stanford a few times. However, without Helmut, it was a different place for me.

Last but not least, I would like to express my sincere appreciation to Masayoshi Nakashima, the "culprit" for this book. We met for the first time at the same event (1982 in Tsukuba) as with Helmut Krawinkler. Although the narrow topic of our research was different, I was indeed fortunate to have an opportunity to cooperate with Masayoshi in many different roles. He attended all the Bled workshops except the first one. All participants enjoyed his excellent presentations, full of wit. In spite of his enormous commitments in the months following the 2011 Tohoku earthquake, he organised and coordinated the Japan/Asia group of participants at the last Bled workshop. To interact with him in the team of editors of Earthquake Engineering and Structural Dynamics was a great pleasure. Masayoshi was an excellent host during my several visits to Japan, including the site visit after the 1995 Kobe earthquake. I have admired his wisdom, outstanding professional and communication skills, high intelligence, his readiness to accept different responsible positions, and ability to perform a remarkable job everywhere.

International Association for Earthquake Engineering