Drafting of Macedonian NDPs for EN 1998 - Design of structures for earthquake resistance - Part 3: Assessment and retrofitting of buildings

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SUMMARY:
This paper presents a methodology for derivation of Nationally Defined Parameters (NDPs) in the frame of endorsement of the Eurocodes for design and building of structures as National Regulation. The proposed methodology has been applied to EN 1998-3 - Design of Structures for Earthquake Resistance - Part 3: Assessment and Retrofitting of Buildings. Although this code contains a relatively small number of pages related to verification of seismic vulnerability and retrofit of existing structures, it is a very important document, having in mind the number of structures it affects. The presented results in this paper are obtained from a research project in the frame of the Project for European assistance to the Macedonian Institute for Standardization aiming at drafting of NDPs. Upon comparative analysis of previous practice in evaluation of seismic vulnerability of structures as well as definition of retrofit measures and solutions for these procedures given in EN 1998-3, values of NDPs for this document are offered.

Keywords: Eurocode 8, NDPs, seismic vulnerability, retrofit, assessment

1. INTRODUCTION

One of the obligations Macedonia has to accomplish in the phase of preparation of its Construction Industry for membership in the European Union is endorsing of Eurocodes as National Regulation. The key issue in that regard is the acceptance of Nationally Defined Parameters (NDPs). The number of NDPs is really serious and serious should be the effort for their definition. One should have in mind the fact that some of them can seriously change the contemporary construction practise, while others, if not thoroughly analyzed, can affect the whole industry, especially the industry for production of construction materials. However, perhaps the most important are the nationally defined parameters, which have to identify the regional characteristics to ensure the intended safety of the structures.

One of the Eurocodes which undoubtedly affects all types of structures and all types of structural materials is Eurocode 8 (EN 1998-1:2004), which deals with design of seismic resistant structures. It is a comprehensive document for implementation of which a couple of hundreds of NDPs have to be verified. A significant part of these parameters has to reveal the seismic hazard of the region; the other part will bring new design procedures. Still, the major part will give the opportunity to reveal and introduce the previous design experience. To achieve this goal, serious and comprehensive research has to be performed.

Part 3 of Eurocode 8 (EN 1998-3:2005), which is related to verification of seismic vulnerability and retrofit of existing structures, is relatively small, but important closed piece, having in mind the number of structures it affects. By endorsing Eurocodes as National Regulation the statements from this document will affect any interventions on the structural system of all existing buildings. This part of Eurocode 8 has eight NDPs, see Table 1.1. In the frame of the Project for European assistance to the Macedonian Institute for Standardisation, the drafting of the NDPs for this document was completed (E. Dumova-Jovanoska et al, 2011). Part of the research activities performed to propose values for NDPs is presented in this paper.

Table 1.1. Nationally Determined Parameters in EN 1998-3:2005

<table>
<tr><th>Subclause</th><th>Nationally Determined Parameter</th><th>Eurocode recommendation</th></tr>
<tr><td>1.1(4)</td><td>Informative Annexes A, B and C.</td><td>[None]</td></tr>
<tr><td>2.1(2)P</td><td>Number of Limit States to be considered.</td><td>LS of Near Collapse (NC).
LS of Significant Damage (SD).
LS of Damage Limitation (DL).</td></tr>
<tr><td>2.1(3)P</td><td>Return period of seismic actions under which the Limit States should not be exceeded.</td><td>LS of Near Collapse (NC):
2.475 years, corresponding to a probability of exceedance of 2% in 50 years.
LS of Significant Damage (SD):
475 years, corresponding to a probability of exceedance of 10% in 50 years.
LS of Damage Limitation (DL):
225 years, corresponding to a probability of exceedance of 20% in 50 years.</td></tr>
<tr><td>2.2.1(7)P</td><td>Partial factors for materials.</td><td>Material partial factors γ_c and γ_s for the persistent and transient design situations and the accidental design situations for use in a country may be found in its National Annex to EN 1992-1-1:2004.
The recommended value for γ_m is 2/3 of the value specified in the National Annex to EN 1996-1-1:2004, but not less than 1.5. The recommended value for γ_s is 1.0.</td></tr>
<tr><td>3.3.1(4)</td><td>Confidence factors.</td><td>CF_{KL1} = 1.35.
CF_{KL2} = 1.20.
CF_{KL3} = 1.00.</td></tr>
<tr><td>3.4.4(1)P</td><td>Levels of inspection and testing.</td><td><table>
<tr><td></td><td>Inspection
(of details)</td><td>Testing
(of materials)</td></tr>
<tr><td></td><td colspan="2">For each type of primary element (beam, column, wall):</td></tr>
<tr><td>Level of inspection and testing</td><td>Percentage of elements that are checked for details</td><td>Material samples per floor</td></tr>
<tr><td>Limited</td><td>20</td><td>1</td></tr>
<tr><td>Extended</td><td>50</td><td>2</td></tr>
<tr><td>Comprehensive</td><td>80</td><td>3</td></tr>
</table></td></tr>
<tr><td>4.4.2(1)P</td><td>Maximum value of the ratio ρ_{max}/ρ_{min}</td><td>$\rho_{max}/\rho_{min} = 2.5$</td></tr>
<tr><td>4.4.4.5(2)</td><td>Complementary, non-contradictory information on non-linear static analysis procedures that can capture the effects of higher modes.</td><td>[None]</td></tr>
</table>

2. CHOICE OF METHODOLOGY

2.1. Analysis of the Existing Situation

Since the disastrous 1963 Skopje earthquake, Macedonia has gathered an ample experience in seismic assessment, but also definition of measures for retrofitting of buildings. It is important to note that Macedonia, as part of Former Yugoslav Federation, was one of the first European countries which enforced the regulation for earthquake resistant design in 1964 (PIOVSP, 1981), as well as the first European country which enforced the regulation for seismic retrofit of buildings in 1985 (PSOROV, 1985). All these facts imply use of previous practice in this field as a background for definition of NDPs for Eurocode 8 – Part 3.

2.2. Methodology for Definition National Defined Parameters (NDP)

As it was mentioned, Macedonia has gathered a considerable experience in seismic assessment and retrofitting of buildings throughout the years. Having this in mind, it was decided to compare previous experience with the solutions given in EN 1998-3:2005. During the first stage, a comprehensive research of available data on existing buildings, as well as engineering practice of strengthening and upgrading of these buildings in Macedonia was performed. In Annex B of E. Dumova-Jovanoska et al. (2011), a list of 49 references is given. Most of them are in Macedonian language. In addition to the local practice, some experience from the countries of Former Yugoslavia was also considered, since they shared the same regulations and working habits in the preceding period.

The analysis of the collected data leads to the conclusion that, regarding structural materials, two classes of structures dominate in the R. Macedonia. Within these classes, two additional subclasses can be defined:

- **masonry structures**
    - with flexible floor diaphragms (wooden)
    - with rigid floor diaphragms (concrete)
- **reinforced concrete structures**
    - bending moment frames
    - bending moment frames with shear walls

Having in mind the limited time and resources available for the study, the following methodology for definition of NDPs was established:

- **First step:** Selection of representative structures for which necessary design projects were available. Nineteen (19) structures were selected for analysis. Their composition with regards to structural type and location is given in Table 2.1. Fig. 2.1 shows the location of the selected representative buildings, and Fig. 2.2 displays the seismic hazard map of Macedonia.
- **Second step:** Datasheets with selected relevant information for each representative building were prepared. A datasheet sample for a representative building is given in Fig. 2.3. As relevant for definition of the NDPs, the following information were selected:
    - Material type
    - Structural type
    - Available design project
    - Geometry
    - Detailing
    - Materials
    - Numerical model
    - Dynamic properties
    - Seismic action
    - Type of analysis, and
    - Verification, as well as some information regarding retrofitting solution.

- **Third step:** The relevant information related to the NDPs from the datasheets was summarized in Table 2.2.
- **Fourth step:** Comparative analysis of the data in a summary table and definition of choice for the Macedonian values for NDPs.

Table 2.1. Location of the selected representative buildings with material type

City	Number of buildings	
	Masonry	Reinforced concrete
Skopje	2	5
Bitola	5	1
Gevgelija	1	2
Tetovo	1	/
Kichevo	/	1
Shtip	1	/
Total	10	9

Figure 2.1. Geographical location of the representative buildings (given in rectangles)

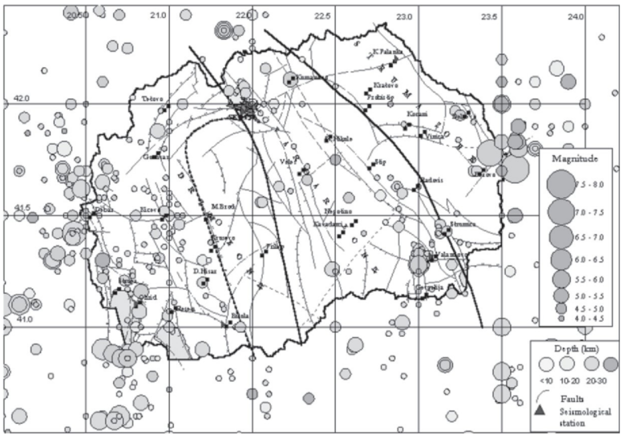


Figure 2.2. Epicenter map of earthquakes in Macedonia (1901-2000)

3. PROPOSAL OF NDPS VALUES FOR MACEDONIA

3.1. Number of Limit States to be considered and Return Period of Seismic Actions under which the Limit States should not be exceeded

The second NDP - *Number of Limit States to be considered* and the third one - *Return Period of Seismic Actions under which the Limit States should not be Exceeded* were analyzed with regards to the previous Macedonian practice in evaluation of seismic vulnerability of structures as two closely related parameters. In the columns *Limit State* and *Return Period* from Table 2.2 and the related notes, one can conclude that two main approaches were established in the previous practice of evaluation of seismic vulnerability.

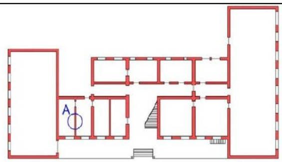
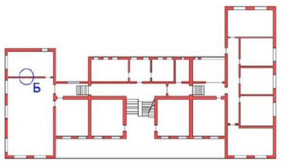
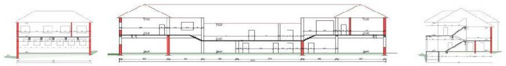
The first one represents application of the positive code regulation (PIOVSP, 1981) that includes linear analysis with equivalent seismic design force. The intensity of the design seismic force, according the definition from the PIOVSP, 1981, could be related to the limit state of significant damage (SD) and return period of seismic action of 500 years. This approach gives an opportunity to use very detailed mathematical models in the analysis, different available software products, with exact geometry of the structural elements and good distribution of the stiffness and masses, which could be considered as an advantage. On the other hand, a serious disadvantage of this approach is the lack of information about structural response in the inelastic range.

The second approach utilizes non-linear time-history structural analysis with different earthquake records, as well as, at least two levels of peak ground acceleration (a_g). The value of a_g usually varies in a wide range from 0.03g to 0.40g. The main goal in this analysis was to define the value of a_g for which inelastic (plastic) behaviour "Y" begins. This stage, with some flexibility, could be related to the Limit State of Damage Limitation (DL) and the value of a_g for which the failure is detected "U", could be related to somewhere in between the Limit State of Serious Damage (SD) and Limit State of Near Collapse (NC). From the values given in the final summary table, it was identified that, for the older structures built before 1964, the value of a_g for the "Y" state was usually in the range around 0.10g, while for the "U" state, it was usually around 0.25g. The big advantage of this approach is the possibility to get information regarding the non-linear and inelastic behaviour of buildings. Since it is a time-history analysis with numerical solution of dynamic equation, the influence of higher modes was properly included. On the other hand, the analysis was performed on simplified mathematical models of the buildings (Necevska-Cvetanovska, 1995). The simplified model was actually a shear type model, with concentrated masses at each story level. Each story was represented by a macro element, with stiffness derived as a sum of stiffnesses of all structural elements that belong to that story. Such model requires existence of rigid floor diaphragms and this is not always the case, especially not in the case of masonry structures. Modelling by macro elements also disables the opportunity for performing analysis at structural element level.

Project title: Technical assistance for capacity building of the Institute of Standardisation (ISRM) EuropeAid/129223/C/SER/MK AFNOR Ref.: E4074 – M1022002

Project:	**Geoloshki zavod**
	Bihacka str. No. 6, 1000 Skopje, Macedonia
Material type:	Masonry
Structural type:	Unreinforced brick masonry
Date of construction:	1930
Reference:	Study for assessment the bearing capacity and stability of the existing building at "Bihacka" str., No. 6 (ex-building of the Geology institute) in Skopje, May 2010, Faculty of Civil Engineering, G. Markovski, E. Dumova-Jovanoska, S. Churilov
Information selected by:	Elena Dumova-Jovanoska, Sergey Churilov

Photo:	

Geometry: Layout	
Ground floor	
First floor	
Sections	

Project title: Technical assistance for capacity building of the Institute of Standardisation (ISRM) EuropeAid/129223/C/SER/MK AFNOR Ref.: E4074 – M1022002

ASSESMENT

Parameters and procedures adopted in project for assessment:

Project:	**Geoloshki zavod**
Available design project	No
Geometry	In-situ geometry measurements and drafting of drawing plans for the structural system
Detailing	1 beam cross-section, 1 slab cross-section
Materials	6 brick samples from wall on ground floor, 6 brick samples from wall on first floor. Compression tests on bricks according MKS B.D8.011/1987, f_b=5.7 N/mm^2. Mortar quality f_m = 2.0 N/mm^2 by engineering judgment (available experimental results $f_{m,cement}$~27 N/mm^2, $f_{m,lime}$~0.6 N/mm^2). γ=14 kN/m^3 from brick samples.
Numerical model	3D FEM models. Two software packages ***Robot Millennium, Fedra*** – elastic analysis. Material mechanical properties acc. EC6. f_k=2.29 N/mm^2, E=2290 N/mm^2, G=920 N/mm^2. Walls and slabs with shell FE, columns and beams elements with linear FE. All walls with thickness ≥ 19 cm are taken into account.
Dynamic properties T (sec)	***Robot Millennium*** – Distributed masses in node points, modal analysis with 20 modes, T^1_x=0.08 sec (66% mass), T^1_y=0.09 sec (25% mass), T^1_{rot}=0.08 sec (42% mass) ***Fedra*** – Approximate fundamental period, (EC8, eq.4.6), T^1=0.22 sec
Seismic action	***Robot Millennium*** – Equivalent static force method acc. PIOVSP'81. Spectral analysis with horizontal design spectrum acc. EC8. a=0.32g, soil B, q=1.5. ***Fedra*** – Lateral force method acc. EC8.
Type of analysis	Linear static, multi-modal, spectral analysis
Verification	
Force	1. Comparison of tensile stresses from design seismic loads to allowable tensile stresses $\sigma_n = \sqrt{\frac{\sigma_0^2}{4} + (1.5 \cdot \tau_0)^2} - \frac{\sigma_0}{2} \geq \sigma_{n,doz} = 0.06\ N/mm^2$. 2. Ultimate limit state design approach acc. EC6 §6, vertical loading $N_{Ed} \leq N_{Rd}$, shear loading $V_{Ed} \leq V_{Rd}$ for load combinations 1.35g+1.5q and 1.0g+0.3q
Displacement	

Figure 2.3. Prepared sample datasheet for a representative building

Table 2.2. Extract from the summary table with relevant information on each representative building

Project	Limit state	Return period	Partial factor for materials	Confidence factors	Inspection and testing	Maximum ρ_{max}/ρ_{min}	Higher modes
Static and seismic analysis of the existing state of the structure Geoloshki zavod-Skopje	PIOVSP Code design	From seismological map of RM for 500 years, MCS – VIII, (acc. to PIOVS'81)	1.0	1.0	Full survey. Limited in-situ testing. KL1: Limited knowledge	RMI: ρ_{min}=0.43, ρ_{max}=1.85 ρ/ρ =4.30 Fedra: ρ_{min}=0.65, ρ_{max}=6.13 ρ/ρ =9.43	N/A
Strengthening and analysis of the Hotel "Epinal" Bitola	PIOVSP Code design	From seismological map of RM for 500 years, MCS – VIII, (acc. to PIOVS'81)	1.0	1.0	Original documentation. KL2: Normal knowledge	N/A	N/A
Static, seismic and dynamic analysis of the newly designed state of structure PHI "State hospital", Block A, Kichevo – (book 3)	Level_1 (4) Level_2 (4)	0.23g – DE (5) (30-40% in 100y) 0.28g – ME (5) (10-20% in 100y)	Acc. to PBAB'87	1.0	Details: Limited Testing : Comprehensive	1.45 (6) State "Y" 3.36 (6) State "U"	N_TH_A (3)

From all data presented in the summary table, but also given in the datasheets, it is obvious that the structural type and the structural material, which are closely related to the period of design and construction, seriously influence the defined values of a_g for both analyzed states "Y" and "U". The values of a_g defined for masonry structures were found much lower than the values of a_g defined for reinforced-concrete buildings, built during the last couple of decades. Having in mind this consideration, it is inevitable to conclude that additional, more comprehensive, analysis of the behaviour of existing structures, with regards to different structural and material types, is strongly recommended. In this sense, the following recommendations should be considered:

- The first regulation on earthquake-resistant design of buildings in Former Yugoslavia was introduced in 1964, after the disastrous 1963 Skopje earthquake, and a new edition of the regulations was published in 1981, which is still valid at this moment. Accordingly, the design and building of the buildings could be divided in two main periods: before existence of appropriate regulations, 1964, and after introduction of such regulations. The second period could be subdivided in two periods: from 1964 until 1981, and the period after 1981.

- In the frame of the periods proposed previously, the analysis of the structural behaviour of different structural and material types should be performed in more details. This consideration is also very much connected to the period of design and construction. Before 1963, buildings with unreinforced masonry dominate in the construction practise, while after the 1963 Skopje earthquake, almost exclusively reinforced concrete structures were built.

Having in mind the limited information given in the datasheets, the summary table, and the above considerations, the following proposal for the values of the NDP - *number of limit states to be considered* and the NDP - *return period of seismic actions under which the limit states should not be exceeded,* has been made:

- Limit State of Damage Limitation (DL) and Limit State of Serious Damage (SD) should be checked for all buildings.
- Limit State of Near Collapse (NC) should be checked only for historical buildings and monuments. A historical building was defined as a building of an urban area which has a "cultural

value" as a whole (historical urban area), but a single building is not a monument. This means that preservation of historical building concerns the general character of the construction techniques typical for the whole area. A monument is a structure of an important "cultural value" so high that it is necessary to guarantee its preservation, generally with its architectural, typological and material characteristics. The relevant institution should give an opinion regarding the status of a building as historical or a monument.

The proposal for the values of NDP - *return period of seismic actions under which the limit states should not be exceeded* is as follows:

- For LS of Near Collapse (NC) and Significant Damage (SD) - to use the recommended values.
- For LS of Damage Limitation (DL), the buildings are divided in two groups:
    - Group 1 (built before 1964) – the return period is 95 years corresponding to a probability of exceedance of 50% in 50 years
    - Group 2 (built after 1964) – the return period is 225 years, corresponding to a probability of exceedance of 20% in 50 years

This proposal was made considering that higher values of the return period for seismic actions for the State of Significant Damage would not bring higher level of life safety, but only higher level of material cost for reconstruction after an earthquake event. This means that, for older buildings, getting the same level of security, with regards to the State of Significant Damage, as that for the newer ones will produce much higher costs of necessary measures which are not always economically affordable to the owners.

3.2. Partial Factors for Materials

When it comes to definition of material properties in the frames of seismic assessment of existing buildings, from the information presented in the datasheets and summarized in Table 2.2, it is clear that, in cases where design documentation existed, the design values of the material properties were used. Different approaches could be stated in cases when information regarding material properties was not available.

In the case of masonry structures, in absence of relevant information, the values of the material properties were usually defined on the basis of previous experience, after visual inspection of the building. Rarely, when testing of specimens extracted from a building was performed, the average of the experimentally achieved values was accepted without any additional partial factors. From previous experience, it was clear that the material properties for masonry, as extremely composite material, defined by testing of specimens from different positions of the same structural element varied in so wide range that it could not be covered by partial factors for materials. On the other hand, the values for the mechanical characteristics of masonry, usually used in Macedonia, were relatively low, so they had less influence on the defined level of seismic resistance of the building than the choice of mathematical model and type of analysis.

In the case of the reinforced concrete buildings, the situation was different. In absence of design values for material properties, testing of considerable number of specimens extracted from different position in the building was performed, or non-destructive testing methods were applied. In such cases, the average of experimentally achieved values was used to define the concrete strength and class. It is important to note that, according to the positive regulation for reinforced concrete structures (PBAB 1987), the design compressive strength is defined with 30% reduction of the average value defined by testing of a certain number of concrete specimens. That means that partial material factor is used, and the value varies from 1.5 to 1.75, depending on the number of test specimens.
The above consideration leads to a proposal for the values of NDP - *partial factors for materials* as follows:

- For masonry 1.0
- For concrete 1.5.

3.3. Confidence Factors and Levels of Inspection and Testing

The NDP - *confidence factors* and NDP-*levels of inspection and testing* are two closely related parameters, also. From the information given in the datasheets and summarized in Table 2.2, it is obvious that the level of inspection and number of in-situ test varies in a vide range from case to case. These parameters depend on the importance of the structure, the available design documentation, and the costs for necessary investigation which are an important factor, as well. Anyway, regardless the level of inspection and the importance of the building, the confidence factor was never used, as a resource to cover the uncertainties in the investigations.

Looking through the information for representative structures, the proposed levels of inspection and testing and the related values of the confidence factors, it was apparent that usage of these values would bring higher confidence to the defined seismic resistance of the existing buildings.

Upon previous considerations, EN 1998-3 recommendations were proposed to be used as Macedonian NDP values for *Confidence factors* and *Levels of Inspection and Testing*.

3.4. Maximum Value of the ratio ρ_{max}/ρ_{min}

Previously, it was found out that two main approaches for seismic assessment of buildings were used in the previous Macedonian practice. The first one applies the positive regulations, which means linear static analysis, and the second one applies the non-linear time-history analysis. The conditions that have governed the choice were not the buildings themselves, with their characteristics like regularity or irregularity, but mainly the experience of the professionals performing the analysis. In the attempt to find the information, which is relevant to the NDP - maximum value of the ratio ρ_{max}/ρ_{min}, it was concluded that the data regarding safety verification were most closely connected to the issue.

From the summarized data and the relevant notes, it could be concluded that, when time-history analysis was used, since the mathematical model consists of macro elements at story level, the irregularities could be detected only at the story level. In cases of linear static analysis, the verification was made at element level, but even when the ratio ρ_{max}/ρ_{min} was higher, additional non-linear analysis was not performed.

Having in mind that the suggested control of the ratio ρ_{max}/ρ_{min} was given to avoid usage of lateral force analysis for buildings with serious irregularities, but also having in mind the previous practice which shows that even without this restrain, good retrofitting solutions were achieved, it was proposed to use the highest suggested value ($\rho_{max}/\rho_{min}= 3$) as Macedonian value of NDP - maximum value of the ratio ρ_{max}/ρ_{min}.

3.5. Complementary, Non-contradictory Information on Non-linear Static Analysis Procedures that can Capture the Effects of Higher Modes

From all elaborations given above, it can be concluded that non-linear static analysis was not used by professionals as a type of analysis for seismic assessment of structures. On the other hand, for the last couple of years, there has been a growing interest in this method of analysis so that several master theses were dedicated to investigation of this topic.
Regarding treatment of the influence of the higher modes on the response of the structures in the Macedonian practice, it depends on the choice of the approach. In the case of linear static analysis according to the positive regulations, the influence of the higher modes is given by adding 15% of the total seismic force to the highest story for buildings higher than 5 stories. In the case of a non-linear time-history analysis, the influence of the higher modes is part of the numerical solution of the dynamic equation.
As NDP - *complementary, non-contradictory information on non-linear static analysis procedures that can capture the effects of the higher modes,* it was proposed to use the analysis methods and provisions given in Chopra and Goel (2002) and Antoniou and Pinho (2004).

4. CONCLUSIONS

Given the thorough and methodical analysis from the Macedonian experience in seismic assessment of buildings, but also having in mind the main goals as well as concepts defined in EN 1998-3:2005, a proposal for the values of the National Defined Parameters (NDPs) for EN 1998 - Design of Structures for Earthquake Resistance - Part 3: Assessment and Retrofitting of Buildings, is given.

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