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PERFORMANCE DOMAIN DESIGN PROCEDURE OF WALL BUILDINGS

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SUMMARY

In order to develop a new design, the designer faces a few numbers of preliminary designs and risk iterations to determine the most efficient structure. Similarly, in the case of reinforced concrete wall buildings, analysis is usually very complicated and time consuming only to find out that following minimum requirement was sufficient in the first place (due to the usually high wall-to-floor ratio). To simplify their design, a conceptual design approach is proposed which allows the designer to incorporate ductility or deformation-based response parameters into the initial phase of the design process.

The proposed "Performance domain" procedure considers multiple performance levels and makes calculation simpler, establishes the minimum material requests that correspond closer to reality. It is based on results obtained in parametric analysis as well as on extensive library of experimental data. The design procedure starts with the specification of desired performance objectives for the entire structural system, given the hazardous environment in which it is to be constructed, and then provides a direct rational path by which the structure may be designed to attain these goals.

The structural response limits are given by means of acceptability diagrams obtained through nonlinear response of various model wall buildings to a set of ground motions. The benefits of such methodology tend to be in improved understanding of the seismic performance of buildings. It should produce predictable and consistent seismic protection for new structures. In the paper, the method is described and discussed, and its basic derivations are given. Application of the method is illustrated by means of an example.

1. INTRODUCTION

Buildings with reinforced concrete structural walls are frequently used in Croatia. Wall-to-floor ratio (at least in one direction) is usually high and although these walls were typically lightly reinforced with simple reinforcement details, such structures exhibited well behavior during previous earthquakes. Nevertheless, their analysis is usually quite complicated and time consuming at which end we find out that following minimum requirements were enough in the first place. The basic goal of this research was to simplify design and evaluation of structural wall buildings by introducing "Performance domain" method as well as by the nonlinear static method N2 (as given in the Annex B of the Eurocode 8: Draft No 6, January 2003). Evaluation is performed on a chosen sample of wall buildings designed following minimum requirements according to the old HRN codes (for existing) and new seismic codes.

When structural walls are situated in advantageous position in a building, they can form an efficient lateral-force-resisting system, while simultaneously fulfilling other functional requirements. Buildings braced by structural walls are invariably stiffer than framed structures, reducing the possibility of excessive deformations

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under small earthquakes. The necessary strength to avoid structural damage under moderate earthquakes can be achieved by properly detailed longitudinal and transverse reinforcement, and provided that special detailing measures are adopted, dependable ductile response can be achieved under major earthquakes.

The structural engineering procedures outlined in most buildings codes utilize a force-based approach for the design of structures to resist earthquakes (Paulay, Priestley, 1992). It has been pointed out that this “equivalent-elastic” forced-based method of seismic design is often not the most effective approach. A primary reason for its inadequacy is the fact that the use of the capacity reduction factor assumes that buildings constructed with similar lateral force resisting systems possess the same ductility. This is clearly not the case, since ductility depends on several other factors such as material strengths, geometry, axial load and reinforcing ratio. Also, there are tendencies of the structural engineering profession toward performance-based design in order to accurately determine the performance of buildings and structural components by calculating deformation-based response parameters such as drift, rotation and strain under various levels of ground motion intensity. It is therefore clear that new seismic design methodologies are required and indeed, there are a number of simplified methods for the displacement based design (mostly employing an “equivalent” single-degree-of-freedom system) which allows the designer to incorporate ductility or deformation-based response parameters into the initial phase of the design process in order to obtain a building that responds more predictably to earthquakes of varying intensity.

2. PERFORMANCE BASED SEISMIC DESIGN

Performance-based seismic engineering is defined as consisting of the selection of design criteria, appropriate structural systems, layout, proportioning and detailing for a structure and its non-structural components and contents, and the assurance and control of construction quality and long-term maintenance, such that at specified levels of all the excitations (that can act on the building) and with defined levels of reliability, the building or facility will not be damaged beyond certain limit states.

2.1 Performance objectives

The first step is the selection of the performance design objectives. These objectives are selected and expressed in terms of expected levels of damage resulting from expected levels of earthquake ground motions. Performance objectives will range from code minimum requirements to fully operational in a maximum credible earthquake ground motion. A performance level represents a distinct band in the spectrum of damage to the structural and non-structural components and functions of the facility (Fig. 1). The seismic hazard at a given site is represented as a set of earthquake ground motions and associated hazards with specified probabilities of occurrence.

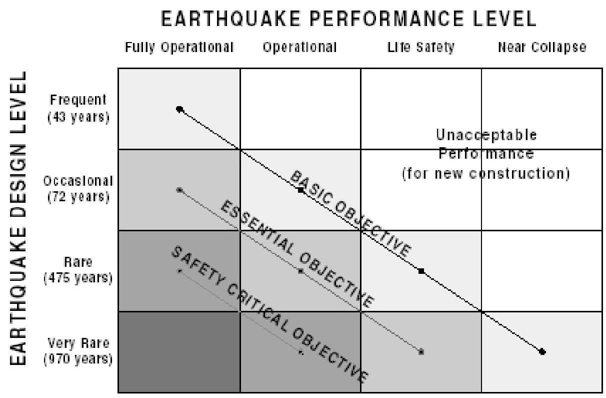


Figure 1: Recommended min. seismic performance design objectives for buildings [ATC-40, 1996]

The performance levels are keyed to limiting values of measurable structural response parameters, such as drift and ductility, structural damage indexes, story drift indexes and rate of deformations. When the performance levels are selected, the associated limiting values become the acceptability criteria to be verified in later stages of the design.

2.2 Response limits

To determine whether a building meets a specified performance objective, response quantities from an analysis are compared with limits for appropriate performance levels:

a) Global building acceptability limits. *These response limits include requirements for the vertical load capacity, lateral load resistance, and lateral drift (Tab. 1).*

b) Element and component acceptability limits. *Each element (frame, wall, diaphragm or foundation) must be checked to determine if its components respond within acceptable limits.*

Table 1: Deformation limits [ATC-40, 1996]

	Performance Level			
Interstory Drift Limit	***Immediate Occupancy***	***Damage Control***	***Life Safety***	***Structural Stability***
Maximum total drift	0,01	0,01 – 0,02	0,02	$0{,}33\frac{V_i}{P_i}$
Maximum inelastic drift	0,005	0,005 – 0,015	no limit	no limit

3. "PERFORMANCE DOMAIN" DESIGN PROCEDURE

The basic goal of the research that led to proposed procedure was to simplify design of structural wall buildings by distinguishing between the domains where simple or no calculations are required and walls are designed mainly according to minimum requirements prescribed by codes, and where more precise calculations are required. In order to establish this procedure, a parametric study has been done.

3.1 Numerical model

A set of model wall building structures were chosen and designed according to the EC8 code (Fig. 2). The varied parameters included:

- building height i.e. number of stories, $\boldsymbol{n}$ — 5, 10, 15
- wall length, $\boldsymbol{l_w}$ — 300cm, 450cm, 600cm
- wall width, $\boldsymbol{b_w}$ — 20cm, 25cm, 30cm
- wall to floor ratio, $\boldsymbol{\rho}$ — 1%, 2%, 3%
- seismic intensity, $\boldsymbol{a_{g,max}}$. — 0,2g, 0,4g.

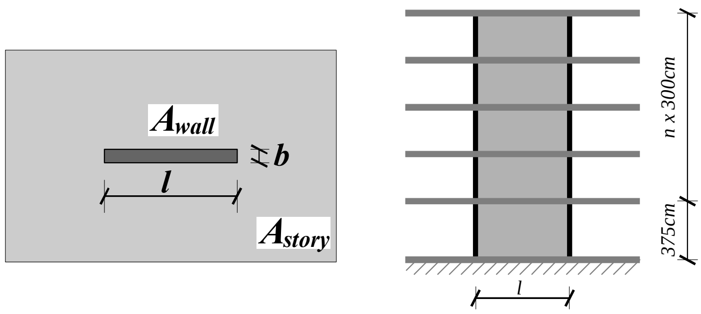


Figure 2: Model wall building layout

Each of the model walls was designed following minimum requirements given by the code as well as according to traditional force-based design procedures. Either way, the walls were carefully detailed to ensure their flexural ductility and protect them against shear failure by capacity design principles, in order to ensure that inelastic action will only occur in intended plastic hinges. Main features of the code minimum requirements are summarized in table 2, as follows [Eurocode 8, 2003].

Table 2: Main features of the code minimum requirements [Eurocode 8, 2003]

	EC 8	HRN
Min. vertical reinforcement throughout the wall	0,004	0,0045 A_w
Thickness of the web	$\boldsymbol{b_{w0}} \geq max(150\text{mm}, h_s/20)$	*min* 150mm
Min. amount of web vertical reinforcement, $\rho_{v,min}$	0,002	0,0015 A_w
Min. amount of web horizontal reinforcement, $\rho_{h,min}$	0,002	0,002 $A_{w,v}$
Length of the confined boundary element, l_c (Fig. 3)	$l_c \geq max(0{,}15 \cdot l_w, 1{,}5 \cdot b_w)$	$0{,}10 \cdot l_w$
Min. longitudinal reinforcement in boundary element	0,005	0,0015 A_w

Within the context of force-based design, current codes suggest the use of an approximation of the first mode response of the wall, with a correction for higher mode effect (equivalent lateral force method). It has been done by distribution of moments along the height of the wall following an envelope of the calculated bending moment diagram, vertically displaced by a distance equal to the height of the critical region of the wall. As for the shear force envelope, it was obtained using the magnification factor depending on the ductility class of the structure. As an alternative, and presumably more accurate method to determine the seismic response, a multi-mode analysis was investigated in this study. Comparison with the equivalent lateral force method as well as with the time history analyses was made.

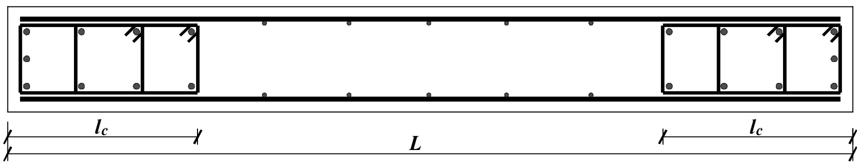


Figure 3: Confined boundary element of free-edge wall end [Eurocode 8, 2003]

3.2 Non-linear analysis

The platform for the nonlinear analysis has been the program LARZ [Lopez, Sozen, 1992]. Wall structure was defined in terms of its geometry, moment-curvature and shear force-deformation relationship for individual elements. Nonlinear response of wall elements included flexure and shear components that mutually contribute to deformations. Trilinear hysteresis in flexure was defined by Takeda rules. Hysteresis in shear was based on the similar system with fewer rules. Each wall consisted of several elements according to the number of stories. The elements were subsequently divided in sub-elements that can experience different stage of inelastic action in order to allow inelastic propagation through the story height. Instantaneous nonlinear characteristics of the structure were constant within a time interval. Damping (ξ) was taken as 2% of the critical. The wall structure was fixed at the base.

The primary moment-curvature and shear force-deformation relationship was derived from the section geometry (uncracked sections), existing initial axial load and stress-strain properties of concrete and steel, using the empirical approach. This procedure has been verified as one that gives reasonably good results [Sigmund at al, 2000]. In that way, we included bond-strength deterioration and slippage of reinforcement to the primary curves. The set of empirical equations, based on tests of elements, was derived for cracking moment (M_C), initial stiffness, yielding moment (M_Y) and stiffness reduction coefficient at yield (α_Y). They completely define the idealized trilinear moment-rotation relationship (M-θ) for members [Aoyama, 1981, Sugano, 1964]. Axial force was assumed to remain constant during the excitation. Shear force and shear displacement relationship were calculated by taking only reinforcement for shear carrying capacity of the section.

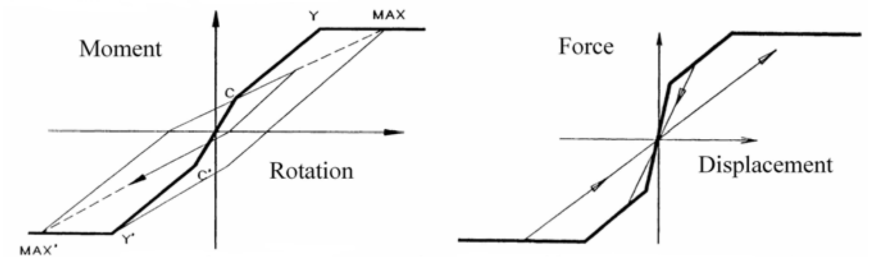


Figure 4: Takeda hysteresis for moment-rotation and base shear-displacement

3.3 Earthquake loading

Nonlinear dynamic time history analysis of the models was calculated using LARZ and recorded ground motions. The maximum accelerations were scaled so that ground motion spectral intensities were similar for the same earthquake zone. The set of three different ground motions was used: Bar N-S and Petrovac N-S recorded during the 1979 Monte-Negro earthquake and El Centro N-S 1940.

Table 3: Basic information about the ground motions used

$a_{g,max}$ = 0,2g Record	Duration (sec)	Acc (g)	SI_20% (cm)	Tg (sec)	$a_{g,max}$ = 0,4g Record	Acc (g)	SI_20% (cm)
0.67*Bar NS	47,84	0,24	121,0	0,98	1.00*Bar NS	0,364	180,0
1.00*El Centro	42,40	0,33	93,6	0,55	1.50*El Centro	0,500	178,0
0.78*Petrovac NS	48,26	0,34	118,0	0,47	1.00*Petrovac NS	0,436	151,0

Where: Acc=peak ground acceleration; SI=Housner's spectral intensity for 20% damping; Tg=ground motion characteristics period.

3.4 Numerical model calibration

Experimental data base used in the study was compiled from the available literature and includes data from laboratory tests carried out on isolated cantilever reinforced concrete walls with three cross-section type: rectangular, barbel and flanged (Fig. 5). Work on that database considers devising a protocol of presenting the research data in the performance form. Relationship between qualitative performance description and engineering parameters that can be considered in design is established. The inputs of the created neural networks are geometrical and material properties, reinforcement ratios and loading. Output variables, which have an important role in performance evaluation, are deformation capacity in terms of drift (δ), shear strength (V) and mode of failure. A set of neural networks were devised and tested until the output results satisfied the set up quality criteria, and the one that gave best overall results was later on used [Stanic at al, 2003].
The results of the predictions achieved using neural networks were compared with independent experimental results (data which have not been used during training and testing of the considered neural networks) of several laboratory tests carried out on reinforced concrete walls. The comparison showed good accuracy of the obtained predictions implying a reliable applicability of neural networks trained on the compiled experimental database to predict the seismic behavior of reinforced concrete structural walls.
The experimental database used in this study includes data from laboratory tests carried out on 285 reinforced concrete walls. All the test specimens were isolated walls fixed at the base. Test walls with rectangular (R), barbell (B) and flanged cross-sections (F) were subjected to either monotonic or various cyclic horizontal loading regimes. The measured response variables are maximum shear force (V_{max}), drift index (ratio of maximum top displacement to the height of the wall) and failure type (S-shear and F-flexural failure). It should be pointed out that for a number of tests the available data were incomplete – so, the original database had to be reduced and rearranged in form suitable for the neural network.

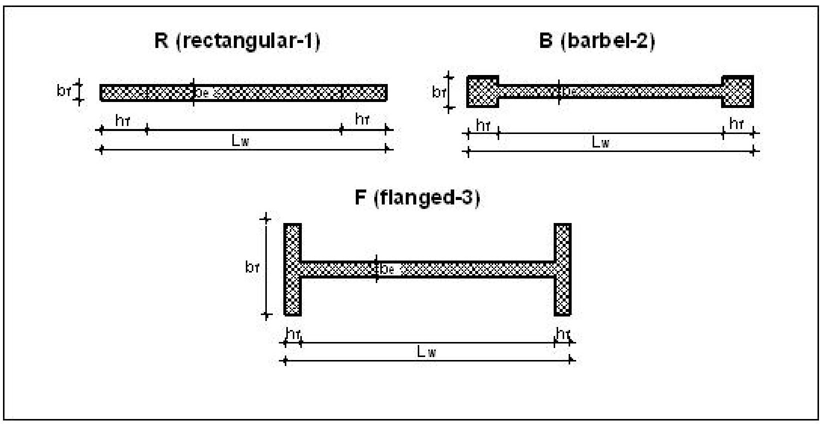


Figure 5: Cross-sectional wall types [Stanic, Sigmund and Guljas, 2003]

Based on theoretical background and available database, the following variables were chosen as input variables influencing structural wall behavior subjected to horizontal loading:

L type of loading: A(1) – alternating, R(2) – repeated, M(3) – monotonic, C (4) – cyclic;
S cross section type: R (1) – rectangular, B (2) – barbell, F (3) – flanged;
rhos (ρ_s) - ratio of effect. volume of confinement reinforcement in boundary element to the volume of the core,
f_{ys} - yield stress of confinement reinforcement in boundary element,
rhobe (ρ_b) - ratio of longitudinal reinforcement in boundary element,
f_{ybe} - yield stress of longitudinal reinforcement in boundary element,
rhov (ρ_v) - ratio of distributed vertical web reinforcement in wall,
f_{yv} - yield stress of distributed vertical web reinforcement,
rhoh (ρ_h) - ratio of distributed horizontal web reinforcement in wall,
f_{yh} - yield stress of distributed horizontal web reinforcement,
b_e - thickness of the wall web,
b_f - width of boundary element,
h_f - length of boundary element,
L_w - length of the wall,
f_c - concrete cylinder compressive strength,
I - moment of inertia
P/A – axial stress in the wall.

Particular input variables having some kind of functional interdependence have been left out in order to increase the effectiveness of neural networks to be trained: A_{be} - cross-section area of boundary element, A_{web} - cross-section area of wall web, A_{cw} - cross-section area of wall, and steel areas A_{sbe} and A_{swv} , h_w – height of the walls. The following variables were chosen as output variables describing structural walls behavior subjected to horizontal loading:

V_{max} (maximum shear force),
u_{max} / h_w (drift index),
failure type (F-flexure 1; S-shear 2).

Once the network is trained, it could be used for prediction of wall seismic performance. Network quality is checked against the independent data network has never seen before.

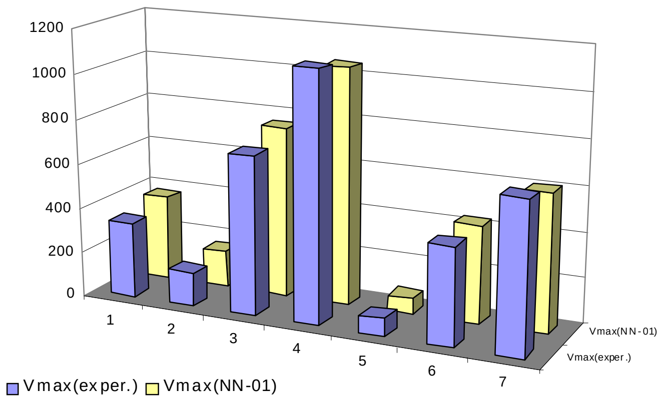


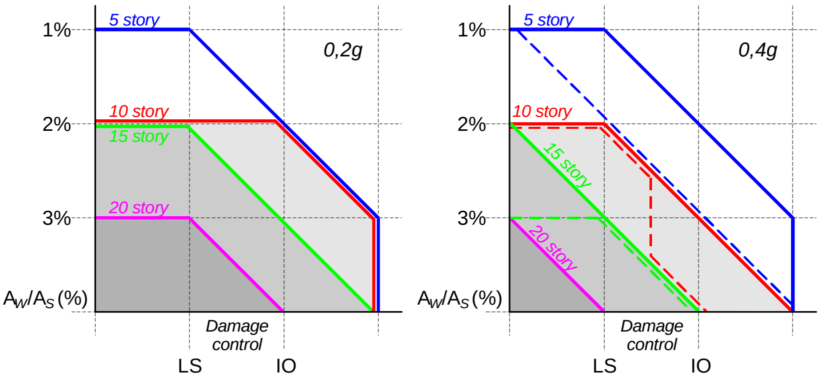
Figure 6: Comparison of experimental results and neural network's prediction for shear force V

3.5 Design procedure

Based on results obtained in parametric analysis, a conceptual comprehensive design approach is proposed. The design procedure starts with the specification of desired performance objectives for the entire structural system, given the hazardous environment in which it is to be constructed, and then provides a direct rational path by which the structure may be designed to attain these goals. After a problem statement is set, the numerical design phase that follows consists of two main groups of steps:

(1) Preliminary design procedure, leading to preliminary sizing and detailing according to the recommended acceptability criteria (Fig. 8).

(2) Final design procedure, where chosen sizing and detailing is checked against the recommended acceptability criteria. If the desired performance is achieved, numerical design procedure can be made according to gravity loading as well as to earthquake induced horizontal loading, that can be two folded: either following minimum code requirements or by means of additional calculations. Either way, the last step should involve structural detailing according to capacity design procedure (Fig. 9) [Paulay and Priestley, 1992].



A_w/A_s – wall-to-floor ratio;   LS – Life Safe;   IO – Immediate Occupancy
- - - design according to minimum code requirements

Figure 7: Recommended acceptability criteria

The structural response limits are given by means of acceptability diagrams obtained through various model wall buildings nonlinear response to a set of ground motions. This methodology tends to be transparent, i.e. based on well-established fundamental principles of structural dynamics, mechanical behavior of real buildings and in compliance with the worldwide-accepted philosophy for seismic design.

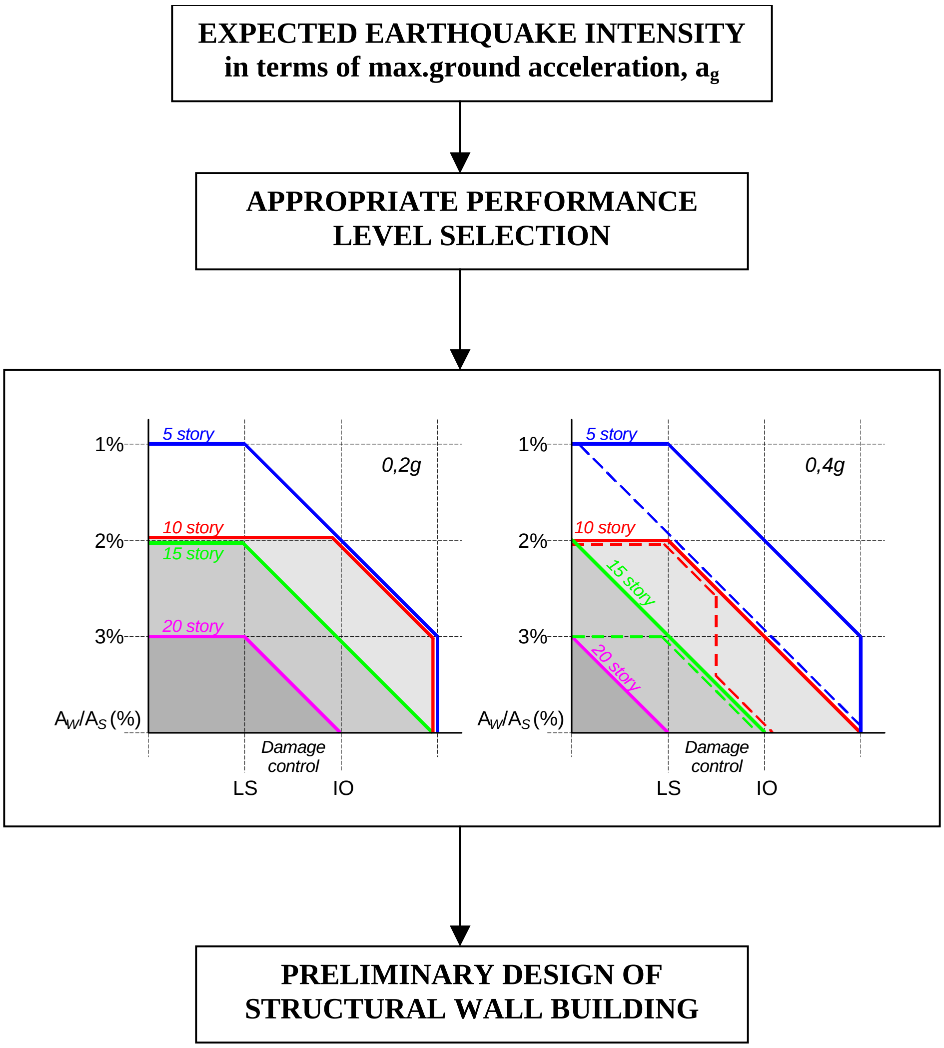


Figure 8: Structural wall buildings preliminary design procedure

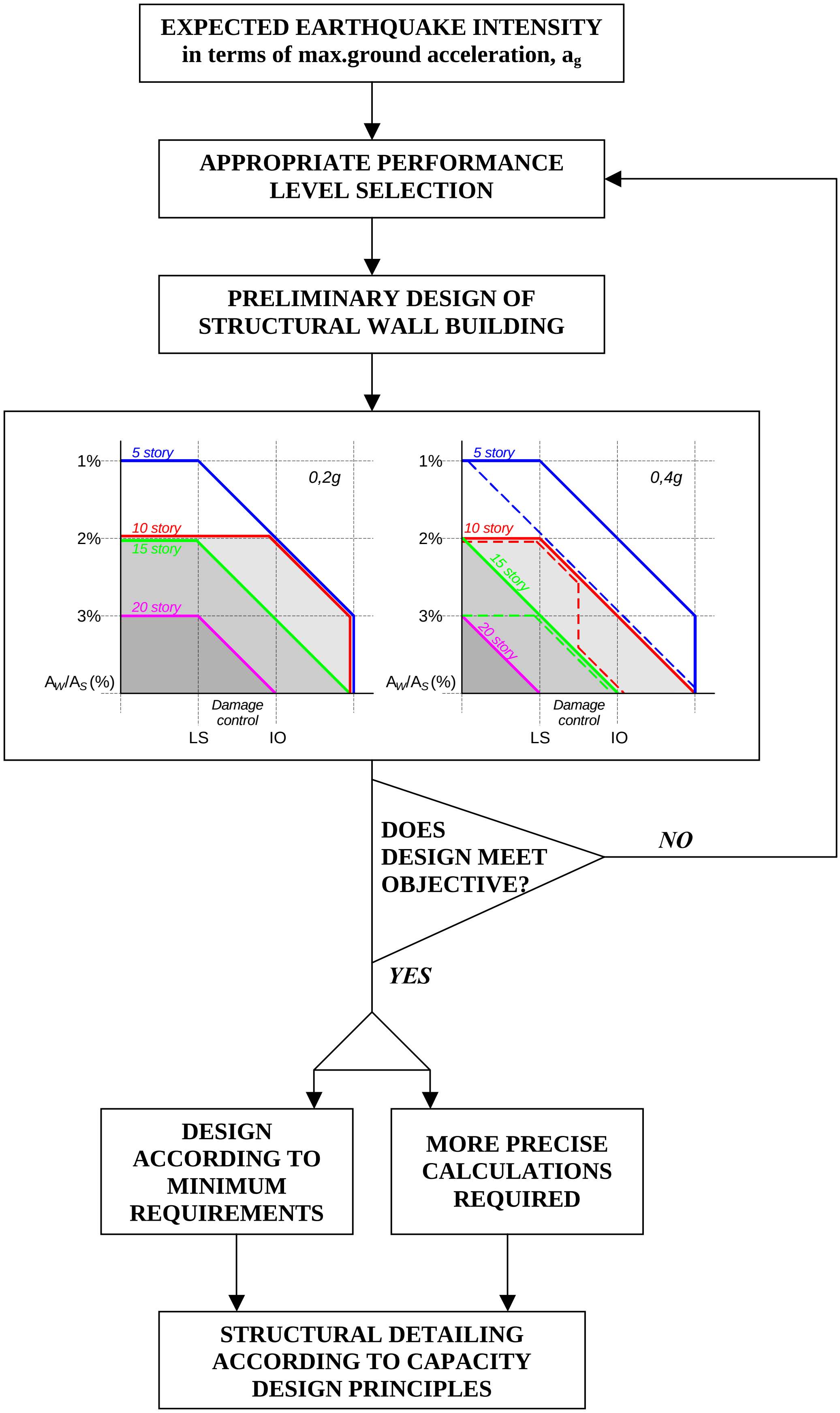


Figure 9: Structural wall buildings final design procedure

4. EXAMPLE

A set of model wall building structures were chosen and designed following minimum requirements according to the previous national HRN codes (for existing buildings) and new seismic Eurocode 8 code (Fig. 2). Seismic responses of idealized 4-, 7- and 12- stories structural models with wall length l_w=500cm, wall width b_w=20cm, wall to floor ratio ρ=2% were evaluated according to proposed "Performance domain" and nonlinear static N2 method for seismic intensities of a_g=0,2g and a_g=0,4g.
Comparing the maximum roof level displacement, relative story displacement, places of the hinge openings and their plastic rotations, evaluation of the structural performance has been done. Structural behavior criteria were evaluated according to the Functional and Life Safe performance objectives. Here are the main observations regarding the results of walls nonlinear analysis:

- For the moderate seismic intensity, all the model walls responded well mainly with the elastic response, but as for the regions with high seismic intensity, all the model walls responded by yielding of the flexural reinforcement in the plastic regions at the base of the wall consistently with capacity design.
- There were a few cases with the shear demand at the base at the higher design intensity exceeding the amplified design shear profile, mainly by influence of the second mode. This could be avoided by increasing the amount of horizontal reinforcement through the critical height of the wall.
- The achieved performance levels expressed in terms of mean drift ratios, interstory drift ratios and plastic hinge rotations, confirmed our introductory statement about very favorable seismic response of wall buildings. Namely, in all cases analyzed for moderate seismic intensity achieved the requirements of Immediate Occupancy structural level. For higher levels of seismic intensity, wall response parameters corresponded to Damage control structural range.

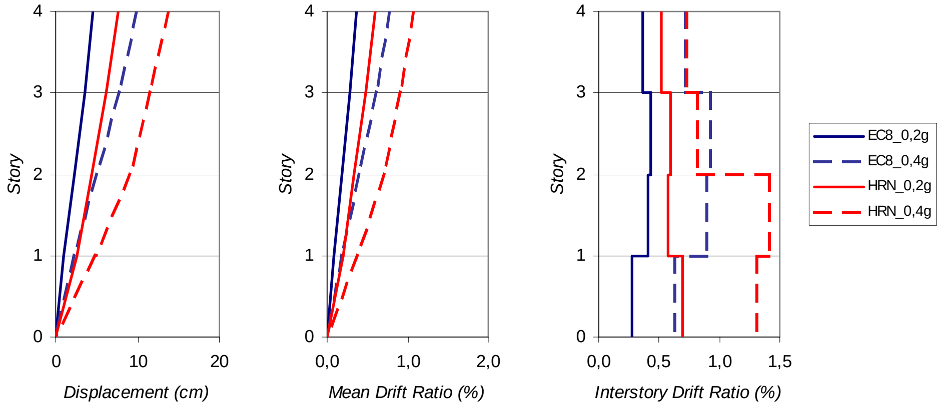


Figure 10: Displacements and story drifts of the 4-story wall buildings

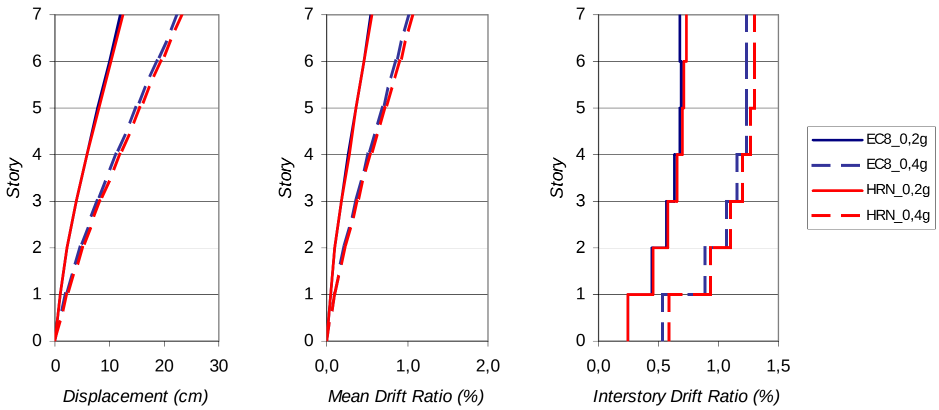


Figure 11: Displacements and story drifts of the 7-story wall buildings

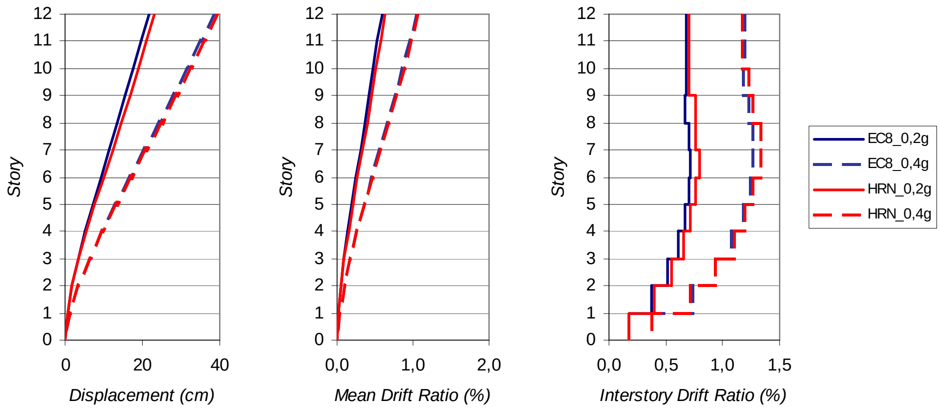


Figure 12: Displacements and story drifts of the 12-story wall buildings

5. CONCLUSIONS

The envelopes of the maximum displacements and story drifts are similar for the HRN and EC8 structures in both seismic zones, which indicates on very favorable seismic response and performance for both newly designed structural wall buildings (according to EC8 codes) as well as for existing buildings (designed according to previous national HRN codes). Analysed walls representing actual structures possess large overstrength even in the case of the minimum reinforcement. Consequently, the response of low structures was practically elastic (with low ductility demand) in most cases. The same was valid for all buildings in areas with lower seismicity. Understanding of the true behavior of structural elements is essential for any performance based design procedure. However, the benefits of such methodologies (as in the case of the N2 and "Performance domain" methods) will be in fundamentally improved understanding of the seismic performance of buildings. Also, it should enhance the options for building owners in the management of seismic risk in an effective and efficient way.

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