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SEISMIC PERFORMANCE OF THE EXISTING AND NEWLY DESIGNED WALL BUILDINGS

Ivica Guljas, Vladimir Sigmund, Tanja Kalman
University J.J.Strossmayer in Osijek, Faculty of Civil Engineering
Osijek, Croatia

SUMMARY

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.

1. INTRODUCTION

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 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 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.

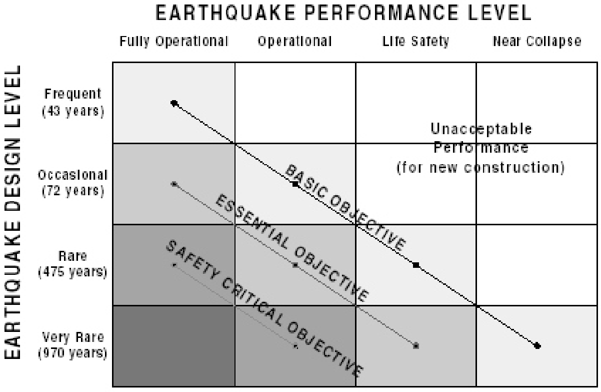


Fig.1: Recommended min.seismic performance design objectives for buildings (ATC-40,1996)

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).*

	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

Tab. 1: Deformation limits (ATC-40, 1996)

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.*

3. "PERFORMANCE DOMAIN" 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. 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. Nonlinear dynamic time history analysis of the models was calculated using nonlinear dynamic computer program LARZ (Lopez and Sozen, 1992) and recorded ground motions. The set of empirical equations, based on tests of elements (Stanic, Sigmund, Guljas, 2003), was derived for cracking moment, initial stiffness, yielding moment and stiffness reduction coefficient at yield. They completely define the idealized trilinear moment-rotation relationship for members (Fig. 2). Shear force and shear displacement relationship were calculated by taking only reinforcement for shear carrying capacity of the section (Sigmund, Guljas, Matosevic, 2000).

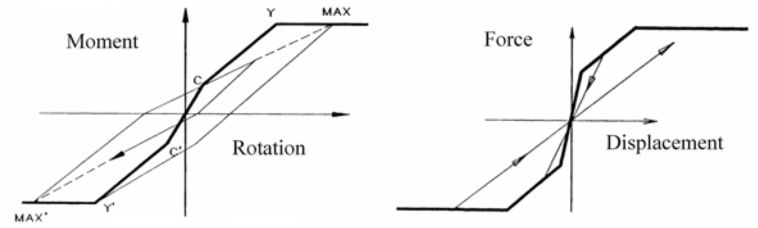


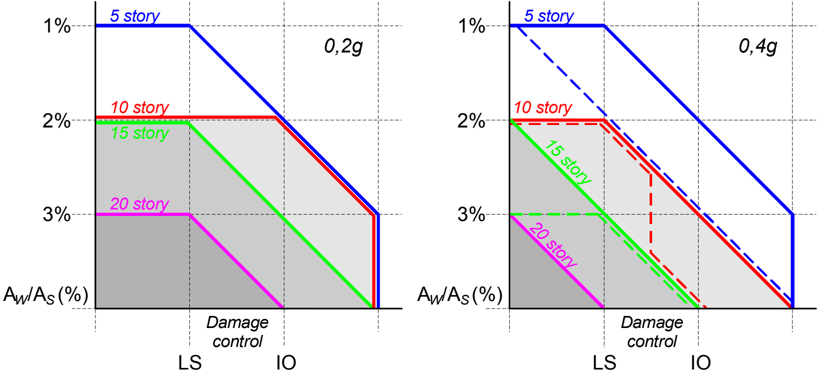
Fig. 2: Takeda hysteresis for moment-rotation and base shear-displacement

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.

(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.

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 (Fig. 3). 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.



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

Fig. 3: Recommended acceptability criteria

4. MODEL WALL BUILDING STRUCTURES

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. 4). Seismic responses of idealized 4-, 7- and 12- stories structural models with wall length $\boldsymbol{l_w}$=500cm, wall width $\boldsymbol{b_w}$=20cm, wall to floor ratio $\boldsymbol{\rho}$=2% were evaluated according to proposed "Performance domain" and nonlinear static N2 method for seismic intensities of $\boldsymbol{a_g}$=0,2g and $\boldsymbol{a_g}$=0,4g.

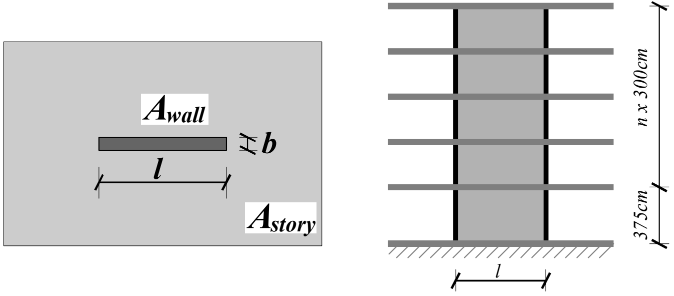


Fig. 4: Model wall building layout

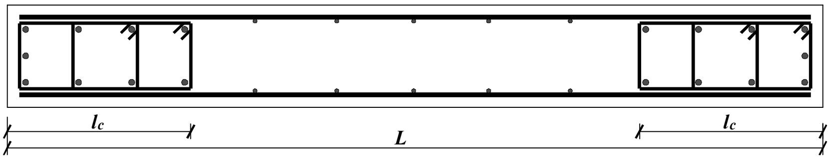


Fig. 5: Confined boundary element of free-edge wall end

	EC 8	**HRN**
Min. vertical reinforcement throughout the wall	0,004	0,0045 A_w
Thickness of the web	$b_{w0} \geq max(150mm, h_s/20)$	*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. 5)	$l_c \geq max(0,15 \cdot l_w, 1,5 \cdot b_w)$	$0,10 \cdot l_w$
Min. longitudinal reinforcement in boundary element	0,005	0,0015 A_w

Tab. 2: Main features of the code minimum requirements

5. ANALYSIS RESULTS

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.

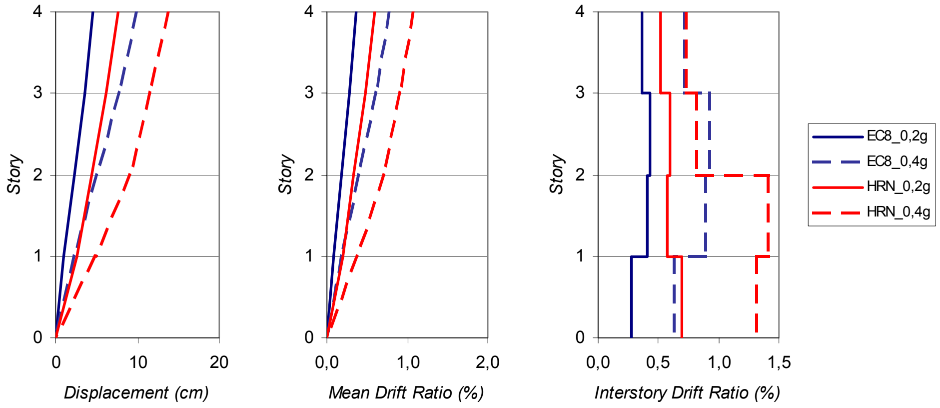


Fig. 6: Displacements and story drifts of the 4_story wall buildings

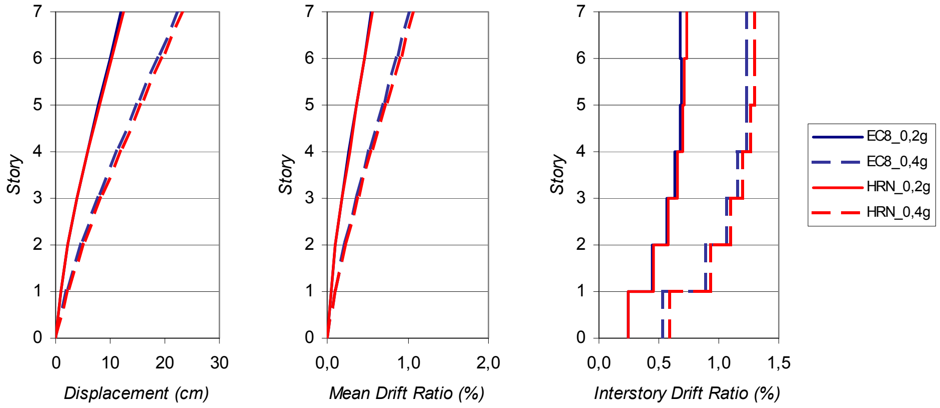


Fig. 7: Displacements and story drifts of the 7_story wall buildings

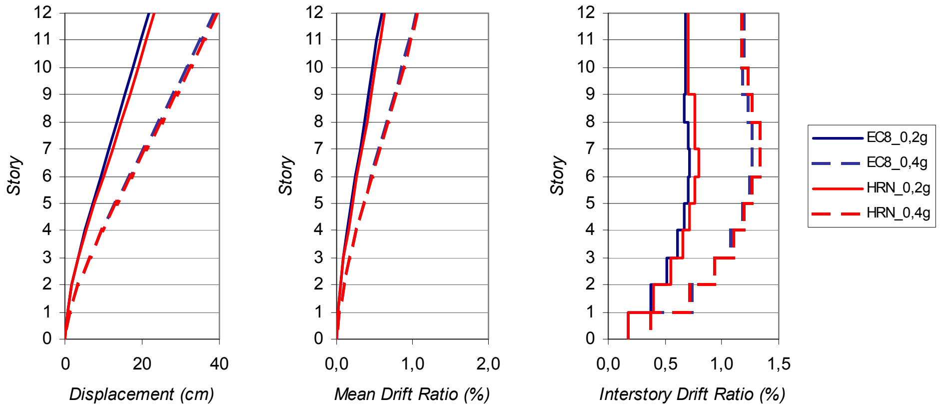


Fig. 8: Displacements and story drifts of the 12_story wall buildings

6. 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. Finally, accurately determined 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 is, after all, a meaningful contribution to Symposium main motto “Keep Concrete Attractive”.

7. REFERENCES

ATC-40 Report (1996), “Seismic Evaluation and Retrofit of Concrete Buildings”, *Applied Technology Council Report No. SSc 96-01*, Redwood City, California, 1996.

Eurocode 8 (2003), “Design of Structures for Earthquake Resistance. Part 1: General Rules, Seismic Actions and Rules for Buildings. Draft No.6”, *European Committee for Standardization CEN*, Brussels, 2003.

Lopez,R.; Sozen,M.A. (1992) “A Guide to Data Preparation for LARZWD Computer Programs for Nonlinear Analysis of Planar Reinforced Concrete Structures, Incorporating Frames and Walls”, *University of Illinois*, Urbana, USA, 1992.

Paulay,T.; Priestley,M.J.N. (1992), “Seismic Design of Reinforced Concrete and Masonry Buildings”, John Wiley & Sons, New York, USA, 1992.

Sigmund,V.; Guljas,I; Matosevic,D. (2000), “CAMUS 3 - International Benchmark, Report on Numerical Modeling”, *Combescure,D, CEA, Commissariat a L’Energie Atomique*, Saclay, Paris, May 2000, pp. 215-230,.

Stanic, A.; Sigmund,V.; Guljas,I. (2003), “Behavior of the Walls under In-plane Horizontal Loadings”, *Fyb-Symposium: Concrete Structures in Seismic Regions*, Athens, Greece.