13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 468

SEISMIC VULNERABILITY EVALUATION OF LIGHTLY REINFORCED WALLS

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SUMMARY

Do we need strict ductile requirements for structural walls in regions with low/moderate seismicity? What would be appropriate behavior factor? What would be seismic performance of the wall with minimum reinforcement? The suggested answers (predominantly provided for thin cantilever walls with rectangular cross section and buildings with high wall-to-floor ratio) are based on several shaking table tests, numerical simulations using multiple-vertical-line-element for 2D and 3D problems, parametric study and an example of probabilistic seismic performance assessment (PSPA). It has been demonstrated that minimum reinforcement frequently produces enough (over)strength of the wall that the response to earthquakes of low to moderate intensity (defined by maximum ground acceleration between 0.1 and 0.3 g) would be practically elastic. This would suggest less stringent details and high behavior factors. However, after exhausting overstrength, cantilever walls have little redundancy. The inelastic demand increases very much for small increase in demand. Accordingly, PSPA indicated rather high probability of exceeding the failure criteria even for the 5-story wall, for which pushover analysis had predicted nearly elastic response at the design level of the earthquake.

INTRODUCTION

Reference research works and publications in the field of seismic behavior of reinforced concrete structural walls (i.e. Paulay [1]) analyze rather heavily reinforced walls with ductile structural details. This reflects in most modern seismic codes, in particular those in New Zealand, which have got strong influence on the development of Structural Eurocodes. However, in Europe as well as in many other parts of the world (i.e. Chile or China) thin lightly reinforced walls prevail. In spite of limited ductility of this type of walls, many such buildings behaved well during strong earthquakes in the past [2], [3]. Consequently it seems that rigorous requirements for high ductility in the codes may impose unjustified burden to national economies in countries with low to moderate seismicity.

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It was observed [4] that overstrength is an important factor contributing to good seismic behavior of buildings with lightly reinforced walls. In central and western Europe this overstrength is frequently attributed to specific architectural concept, where load bearing structural walls are used as partition walls. Consequently, buildings with high wall-to-floor area ratio in (at least one principal direction of the structure) are typical in Europe (Figure 1). However, the question arises when such overstrength exists and when it can be relied on? Although observations of the behavior of lightly reinforced structural walls have been supported by some experimental and analytical research (i.e. [5], [6]; see also next section), the evidence is still very limited. In particular, little is known about the near collapse mechanism, which limited ductile walls approach very soon after the (over)strength has been exhausted. Therefore the question still exists when and to what extent such – relatively lightly reinforced walls can be used in seismic areas and what code requirements should be used in their design?

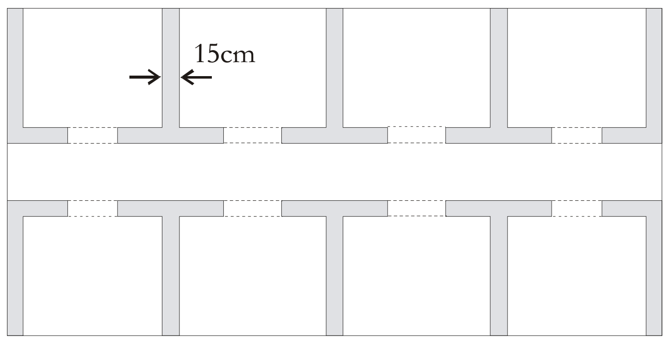


Figure 1. Typical floor plan of a wall structure in central Europe

Related, experimentally supported analytical research done at the University of Ljubljana is presented in this paper:

- A modified multiple-vertical-line-element macro model [7] was developed to study seismic response of structural walls, loaded in one horizontal direction.
- After several applications of the model using existing test results [7], the model was successfully used in blind predictions of the seismic response of cantilever RC walls subjected to a series of consequent earthquakes on a shaking table in the frame of the CAMUS 1 [8] and CAMUS 3 [9] programs in France.
- Using the calibrated model a parametric study of seismic response parameters was done for cantilever walls using simplified non-linear N2 evaluation procedure.
- Vulnerability analysis of a typical rectangular structural wall was made.
- Bi-directional version of the MVLEM enabling 3-D analysis of structural walls having complex structural configuration was developed. Additional features enabling the study of the confinement of the edges and cracking of concrete were added. The extended model has been implemented into the up-to-date performance simulation framework OpenSees developed by PEER [10].
- The model has been verified using results of the CAMUS 2000 shaking table experiment.
- Shaking table experiment of the H-shaped coupled wall in the frame of the ECOLEADER program is under way.
- Blind prediction of the H-shaped coupled wall under simultaneous 2D earthquake excitation was made.
- Test results and post-experiment analyses will be presented at the Conference.

MULTIPLE-VERTICAL-LINE-ELEMENT MODEL FOR RC WALLS

Following full scale tests on 7-story RC frame-wall building in Tsukuba Japanese researchers (i.e. Kabeyasawa [11]) developed special macro element to simulate specific seismic response of RC walls (i.e. uplift of tension corners). The model consisted of rotational spring for the central panel and two external vertical springs modeling the force deformation relationship for boundary columns. Within subsequent development of the model the central rotational spring was replaced by additional vertical springs – resulting in multiple-vertical-line-element model (MVLEM). The research group in Ljubljana modified the hysteretic rules for the springs and incorporated the model into the DRAIN-2D program and recently into the OpenSees framework.

In-plane (2D) model

In the multiple-vertical-line element model (Figure 2), several vertical springs are connected by rigid beams at the top and bottom level. They simulate axial and flexural behavior of the wall segment using simple hysteretic rules (see i.e. [7]). Horizontal spring is modeling shear behavior.

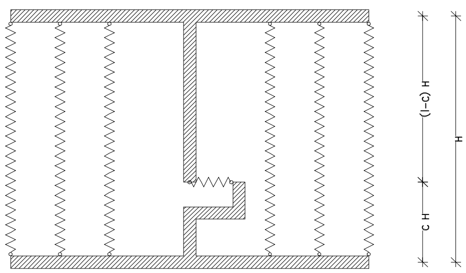


Figure 2. In-plane (2D) version of the Multiple-vertical-line-element model (MVLEM)

Bi-directional (3D) model

Although MVLEM has proved to be a suitable tool, successfully balancing the simplicity of a macroscopic model and the refinements of a microscopic model, all the applications were limited to the in-plane response of cantilever walls. However, there has been need in research and practice for bi-directional analyses of walls having more complicated structural configurations. Therefore a bi-directional version of the MVLEM enabling 3-D analysis of structural walls having complex structural configuration was developed (Figure 3). Additional features enabling the study of the confinement of the edges and cracking of concrete were added. The extended model has been implemented into the up-to-date performance simulation framework OpenSees developed by PEER [10].

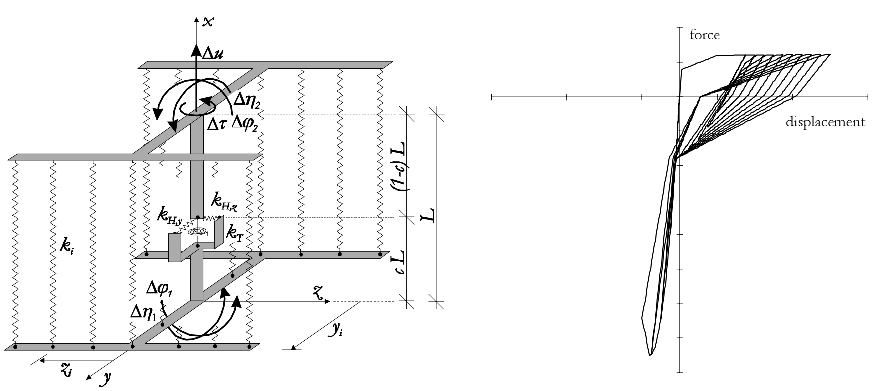


Figure 3. Bi-directional (3D) version of the Multiple-vertical-line-element model (MVLEM) with improved vertical spring hysteretic properties

CAMUS PROJECTS

Large-scale RC cantilever structural walls (Figure 4) were studied in the frame of the CAMUS (Conception et Analyse des Murs sous Séisme) program on the shaking table at the CEA in France. Lightly reinforced wall, designed according to the French practice, was tested in the frame of the CAMUS 1. Within the CAMUS 3, the wall was designed according to the EC8 standard. Bi-directional response was studied in the frame of CAMUS 2000. Research group in Ljubljana participated in the benchmark studies in the frame of the CAMUS 1 [12] and CAMUS 3 [13] and in post-experiment evaluations in the frame of the CAMUS 2000 [14]. Relatively simple analytical tool (MVLEM) was able to predict operational/life safe performance in all cases. There have been some difficulties in prediction of some types of near collapse performance (i.e. fracture of the reinforcement initiated in the relatively early stage of the CAMUS 3 program). The response to simultaneous excitation in two horizontal directions was well modeled. Some most important parallel conclusions regarding seismic behavior and performance of the analyzed structural walls are summarized below.

Due to the rocking of the walls, the shift of the neutral axis occurs during response. Corresponding vertical accelerations lead to important variation of axial forces in cantilever walls. The flexural capacity is varying in correspondence with the axial force. The corresponding increase in the shear force may jeopardize the capacity design approach. Bi-directional horizontal loading in the case of CAMUS 2000 test substantially increased this effect.

High level of tensile strain can be induced in boundary areas of walls (in particular those having large wall-to-floor ratio and related low level of compression load). If brittle reinforcement is used, this may lead to serious problems in seismic performance, as demonstrated by CAMUS 3, which has provided serious warning in the time when the marked is flooded by cheap, brittle reinforcement.

Heavy damage of the wall's free edge, subjected to compression was observed at the end of all tests. The damage to the free edge of the CAMUS 3 rectangular wall after the final test is shown in Figure 5. Clearly, this wall would not survive (very strong) loading without substantial amount of confining reinforcement (designed according to EC8). Unfortunately older design practice in Europe provided far less confinement if any (note also typical design details in Figure 15).

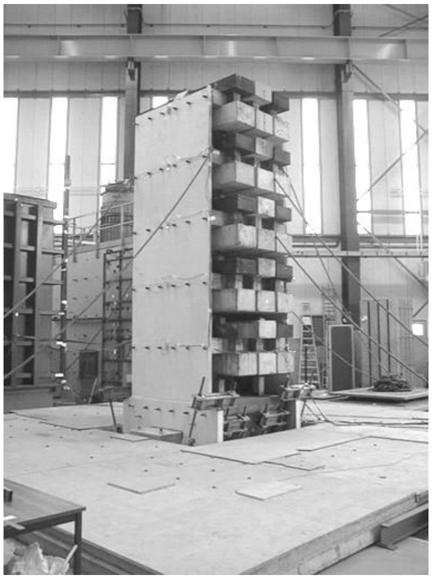
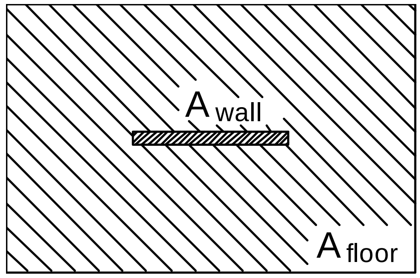


Figure 4. CAMUS test

Figure 5. Damage to the free edge of the wall at the end of the test sequence (CAMUS 3)

PARAMETRIC STUDY OF IDEALIZED BUILDINGS WITH STRUCTURAL WALLS

Simplified non-linear evaluation method N2 [15] and MVLE model were used in the parametric study of the idealized buildings with structural walls. The study has been done to evaluate the requirements of the Eurocode 8 standard as well as to estimate the inelastic response of buildings with structural walls. The floor plan of the idealized building (see Figure 6) was determined based on the floor plan of the typical building with structural walls (compare to Figure 1). The influence of the flanges and wall openings has not been considered.

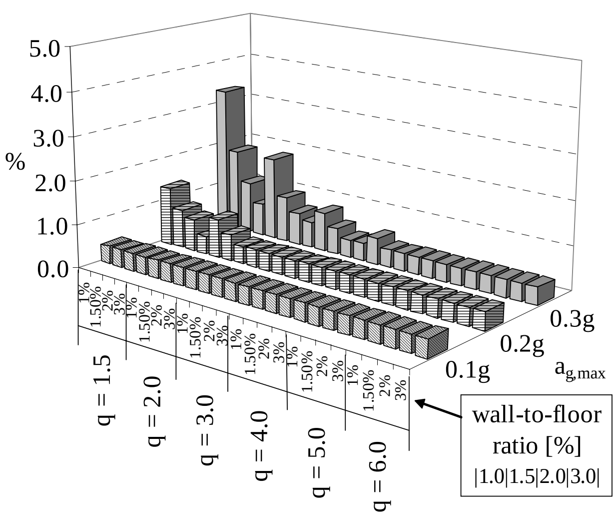


$$\rho_1 = \frac{A_{wall}}{A_{floor}}$$

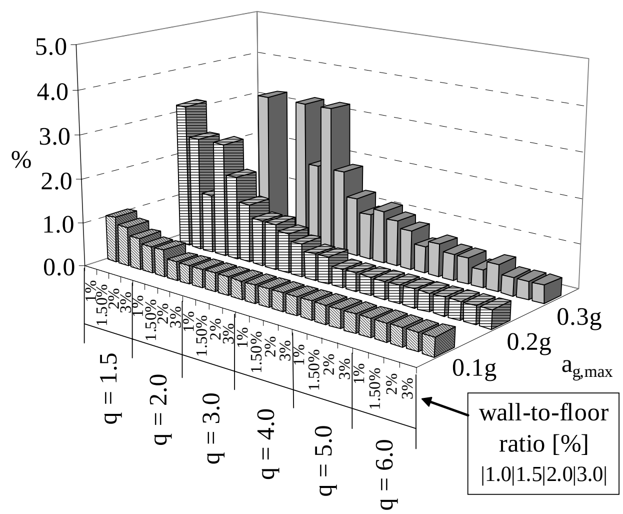
Figure 6. Idealized floor plan

The area of the wall was kept constant (A_{wall} = 1.0 m^2). The varied parameters included: maximum ground acceleration ($a_{g,max}$ = 0.1 g, 0.2 g, 0.3 g), seismic force reduction factor (behavior factor q = 1.5, 2, 3, 4, 5, 6), number of stories (n = 5, 10, 15) and wall-to-floor ratio (ρ_1 = 1 %, 1.5 %, 2 %, 3 %). Axial force was calculated based on one half of the tributary area (considering the other half is carried by walls in the perpendicular direction).

All buildings were designed according to Eurocode. The required percent of the longitudinal reinforcement in boundary areas is shown in Figures 7a and 7b. The results are plotted as the function of the behavior factor (q) and wall-to-floor ratio (ρ_1) represented together on one axis, and $a_{g,max}$ represented on the other axis.



a) 5-story walls



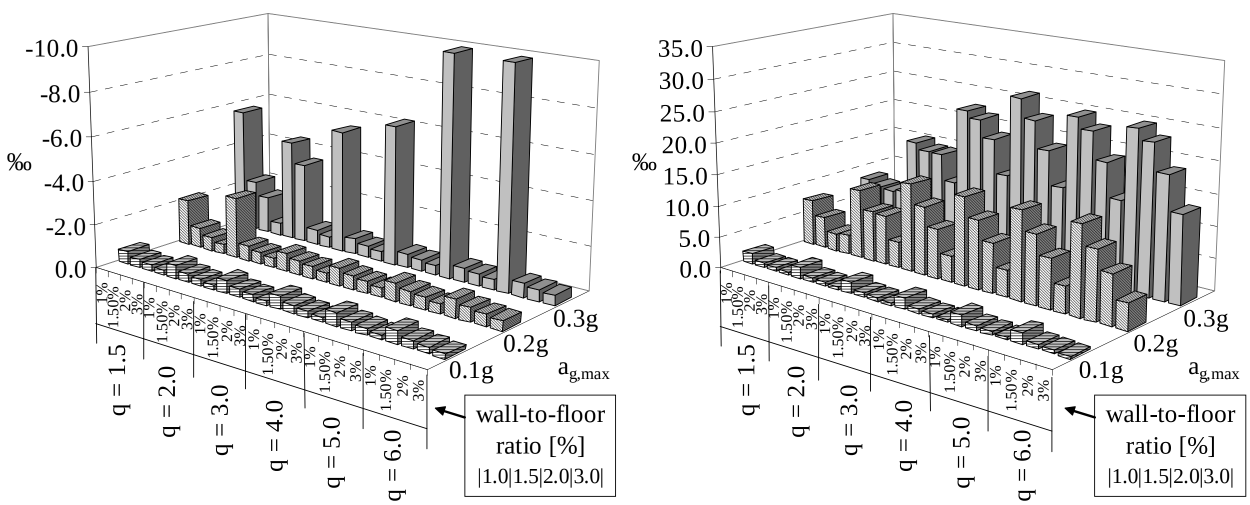
b) 10-story walls

Figure 7. The required percent of the longitudinal reinforcement in boundary columns.

It can be observed that the minimum required longitudinal reinforcement (0.4 %) in the boundary areas satisfies design requirements for practically all walls designed for maximum ground acceleration 0.1 g and for those walls designed with high behavior factors. However, when the strength provided by the wall having minimum reinforcement is exhausted, the required reinforcement increases very rapidly. Note that for those cases, where the required reinforcement is larger than maximum amount permitted by EC8 (4 %) results are not shown in Figures 7, 8 and 9 – some columns in the diagrams are omitted.

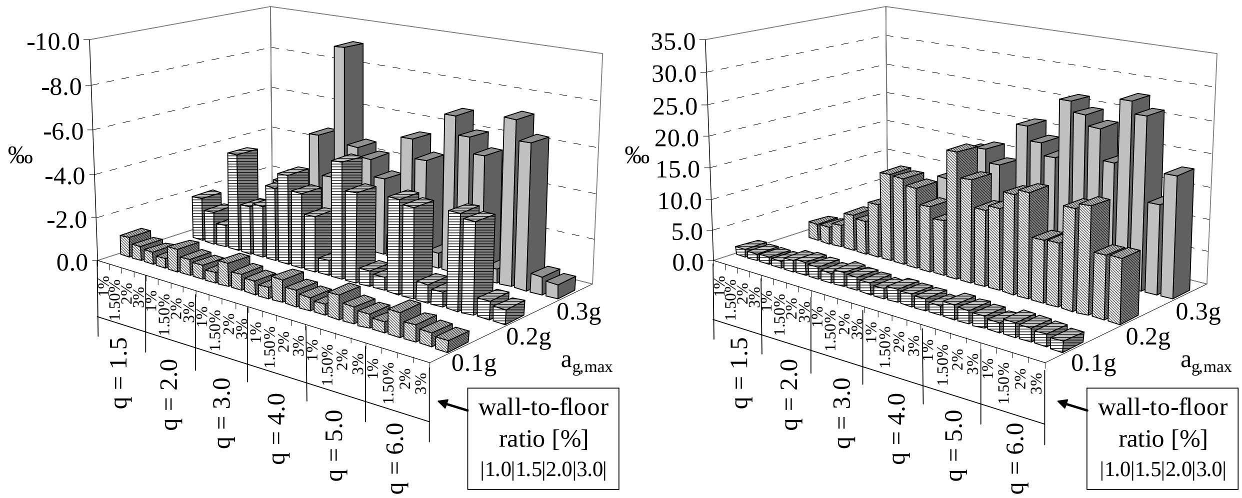
The N2 method enables analysis of the global response of the structure and local behavior of structural elements. While drift was chosen as the basic parameter describing the global behavior, maximum deformations of tension and compression edges of the wall were used as the measure for the local response.

The global response of all analyzed buildings was practically elastic in the case of weaker earthquake load ($a_{g,max}$ = 0.1 g) and low (five-story) buildings (Figures 8 and 9). For higher buildings (10 stories), subjected to moderate earthquakes ($a_{g,max}$ = 0.2 g, 0.3 g) the analysis with the N2 method resulted in larger nonlinear deformations. Still the drift did not exceed the value of 1 %, which is typically considered as acceptable value. Nevertheless several walls had problems in local response. Large compression deformations (more than 0.5 %) as well as tension deformations (about 2.5 %) were observed in higher buildings subjected to moderate earthquakes. In these cases, wall to floor ratio becomes very important. If this ratio is more than 2 %, compression and tension deformations are within acceptable range, while walls in buildings with lower wall-to-floor ratio have got problems.



a) compression b) tension

Figure 8. Maximum deformations on the compression and tension edges of the walls (5-story buildings)



a) compression b) tension

Figure 9. Maximum deformations on the compression and tension edges of the walls (10-story buildings)

SEISMIC PERFORMANCE ASSESSMENT OF A TYPICAL RECTANGULAR STRUCTURAL WALL

Although the parametric study has not indicated any problems for the analyzed 5-story buildings with structural walls, it was decided to make more rigorous assessment of seismic performance for a typical wall shown in Figure 10. The main properties of the wall has been:

- number of stories: 5
- wall-to-floor ratio: 1.5 %
- minimum reinforcement according to EC8

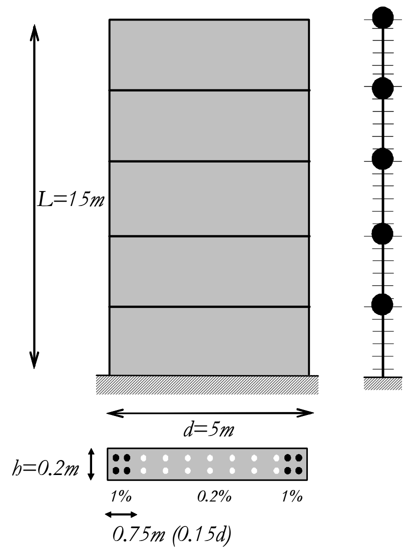


Figure 10. Typical 5-story structural wall

Seismic performance assessment methodology developed by Cornell and Krawinkler [16] was applied in the study. The annual and 50-years probabilities of exceeding the estimated capacity of the structure were obtained as the final result. In the paper the two shortened terms for those probabilities are used: annual and 50-years probability, respectively. In general, the presented problem is characterized by three random elements: the ground motion intensity, the demand on the structure and capacity of the structure.

In the presented study the peak ground acceleration (PGA) has been used to specify the ground motion intensity. The acceleration hazard curve used in the study is presented in Figure 11. It represents the annual probability that the random acceleration of a_g at the site will equal or exceed some specified level of ground acceleration. A standard deviation of the acceleration hazard has not been taken into account in the presented study.

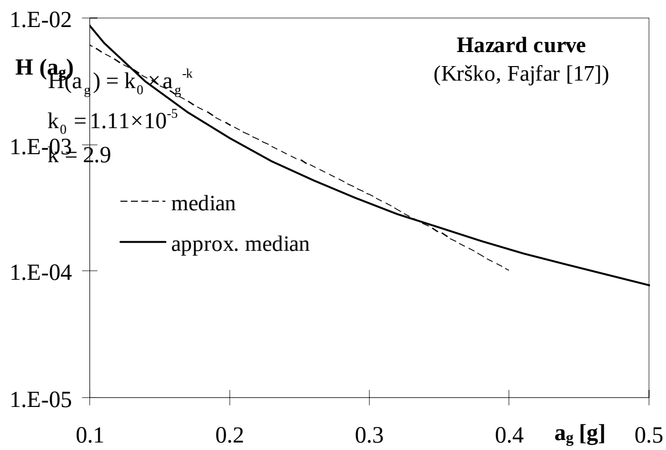


Figure 11. Hazard curve corresponding to one of the typical sites with moderate seismic intensity (50-years probability of occurrence of an earthquake intensity of 0.2 g is about 10 %)

Two variables, maximum displacement at the top of the wall and the Park-Ang damage index have been used to characterize the demand on the structure. To obtain the correlation between the PGA and the two measures of the demand, series of nonlinear analyses have been performed. The elastic spectra of the accelerograms used in the study are presented in Figure 12. The results of the nonlinear analyses are presented in Figure 13. The predicted value of the top displacement or damage index (median value) at the given level of PGA has been estimated using the results of the nonlinear analysis.

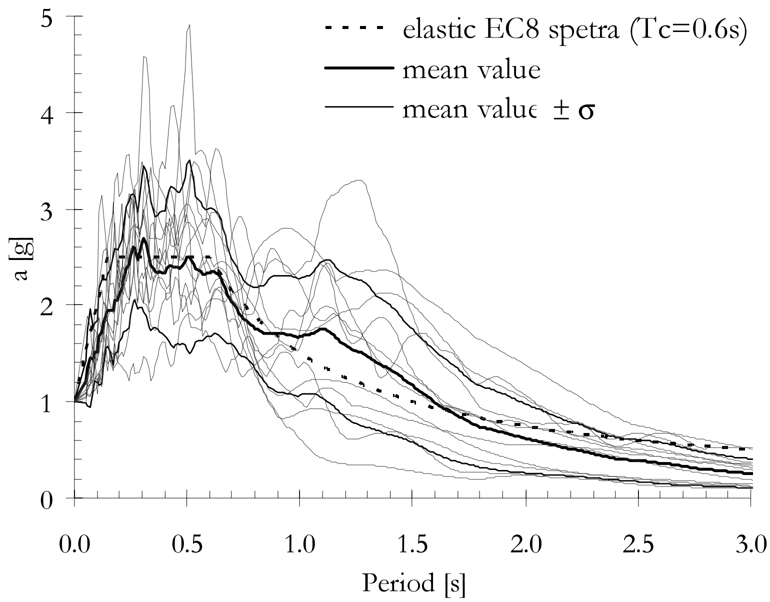


Figure 12. Elastic acceleration spectra corresponding to the earthquakes, used in the study

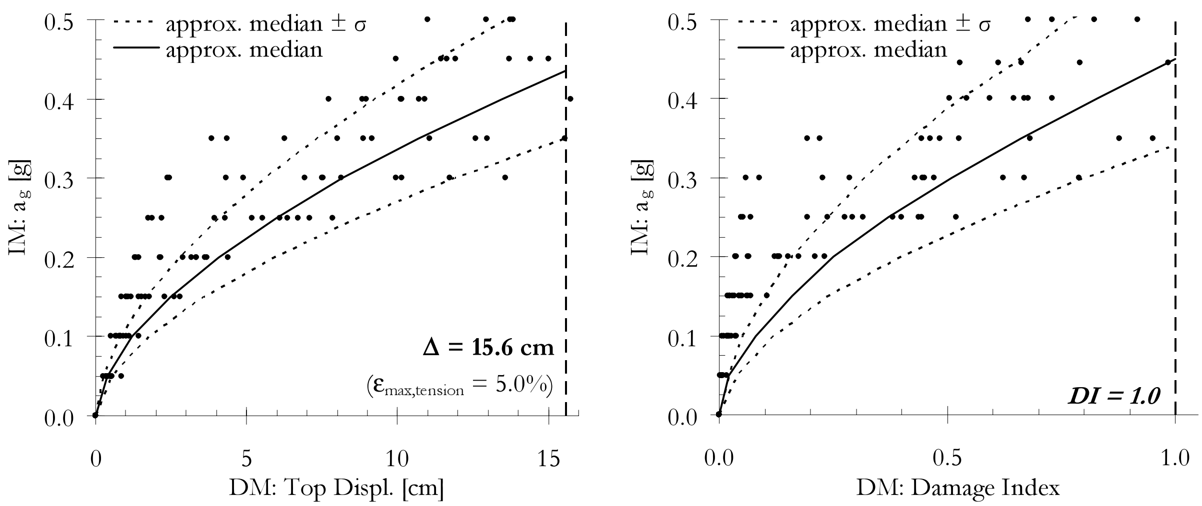


Figure 13. The results of the nonlinear analyses and estimated correlation between peak ground acceleration and demand on the structure.

Finally two criteria for the collapse of the structure have been defined. The collapse has been estimated either by a top displacement of the wall (15.6 cm), corresponding to a 5 % deformation in the tension reinforcement of the critical cross-section or by Park-Ang damage index equal to one. Due to a lack of relevant data, in this, initial phase of study a deterministic value for both measures of collapse has been used (all uncertainties, e.g. those regarding the material properties have been neglected).

Using the top displacement as a demand measure, the annual probability of $1.5 \cdot 10^{-4}$ and 50-years probability of $7.5 \cdot 10^{-1}$ at the location corresponding to the presented acceleration hazard curve has been obtained. Similar results (annual probability of $1.6 \cdot 10^{-4}$ and 50-years probability of $7.8 \cdot 10^{-1}$) have been obtained based on the damage index as a demand measure.

Although the presented estimation of the probability of failure is approximate (the randomness of several variables have been neglected), it can be concluded that the five story structural wall, design with the minimum reinforcement according to the Eurocode 8, situated in regions with moderate seismic intensity and similar earthquake hazard as the investigated one (the 50-years probability of occurrence of an earthquake with the peak ground acceleration a_g = 0.2 g is about 10 %) is considerable.

Note that the conclusions based on the results of the parametric study in the previous section do not directly support this observation. Nevertheless, comparison of both results proves that the response of this type of walls in the regions of moderate seismicity is in general adequate for design level of load. However, when the (over)strength of the structure is exhausted, there is no mechanism to distribute the demand. As a consequence even a small increase of the load results in a large increase in the demand, leading to significant increase of the possibility of failure.

ECOLEADER SHAKING-TABLE EXPERIMENTAL PROGRAM »SEISMIC PERFORMANCE OF LIGHTLY REINFORCED STRUCTURAL WALLS IN LOW TO MODERATE SEISMICITY AREAS«

Objectives and scope of the program

University of Ljubljana, L3S Grenoble, INSA Lyon and University of Patras proposed a research project using shaking table facilities at the LNEC, Lisbon to evaluate seismic performance of lightly reinforced structural walls in low to moderate seismicity areas.

The aim of the research has been:

(a) To investigate the influence of simultaneous loading in both horizontal directions.
(b) To address walls with T (H) cross-sections.
(c) To investigate the free edge of a T (H) shaped walls in compression.
(d) To investigate the behavior of coupled walls.
(e) To evaluate the behavior of coupling beams in thin walls.
(f) To calibrate and further develop numerical models.

Shaking table tests of two 1:3 specimens, one representative for Slovenia and Central Europe (SLO; Figure 14) and one for France are proposed. Both specimens will be loaded by a sequence of accelerograms with increasing intensity in both horizontal directions simultaneously. Representative earthquake for the two regions (Tolmezzo, Friuly; modified to match EC8) has been used in analyses and experiment.

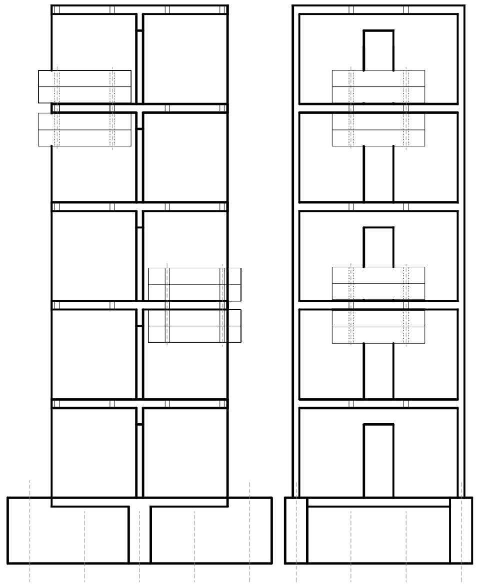


Figure 14. “SLO” 1:3 model used in the ECOLEADER test

Cross section and the details of the wall are shown in Figure 15. These details were chosen based on the information on the current and past practice in detailing structural walls, provided by leading Slovenian designers.

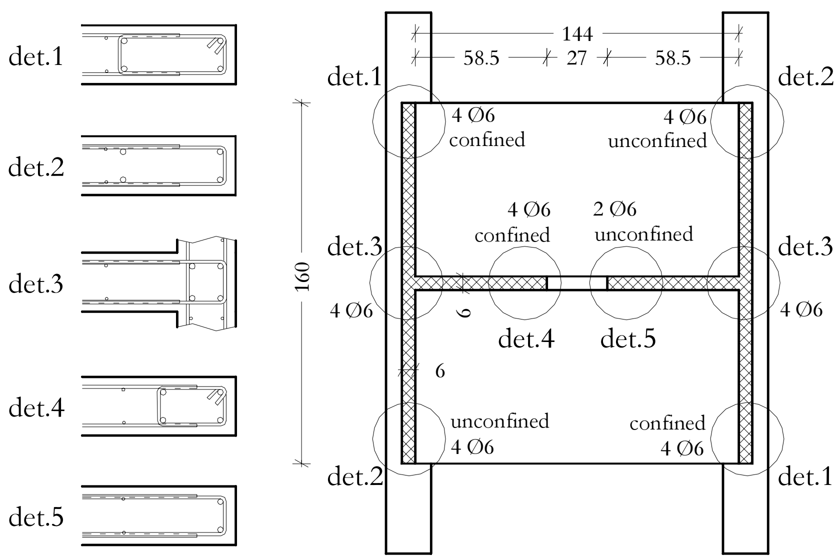


Figure 15. Cross section and the details of the wall

There has been a special interest to investigate the behaviour of the potentially dangerous unconfined edges of thin walls (note that some edges are confined and the others are not). The coupling beams are diagonally reinforced with two crossed bars. Possible bucklinng and/or slip has been studied.

Unfortunately, the experiment has been postponed until April 2004 by LNEC, Lisbon. Therefore, only the analytical prediction of the response is discussed in the next paragraphs, while the experimental results and subsequent analyses will be presented at the Conference.

Analytical prediction of the response

The wall was subjected to a series of earthquakes acting in both directions simultaneously. The bi-axial MVLE model, incorporated into OpenSees was used in the analysis.

Among several important parameters, only the influence of confinement on the response has been chosen for presentation and discussion in this paper. It has been expected that unconfined free edges represent hazard in thin structural walls. In fact, it has been clearly demonstrated that the weakest point of the structure is the unconfined edge of the wall along the openings (detail “5” in Figure 15). This observation is even more important, since many designers consider this “central” portion of the wall as non-problematic. However, in fact this edge is subjected to strong demand in compression as well as in tension. The MVLE model successfully identified the failure of the unconfined edge in coupled wall (Figure 16) at maximum ground acceleration 0.6g in both directions. When the edge was hypothetically confined (detail “5” was replaced by detail “4”; in Figure 15), an important improvement of the behavior has been predicted (Figure 17).

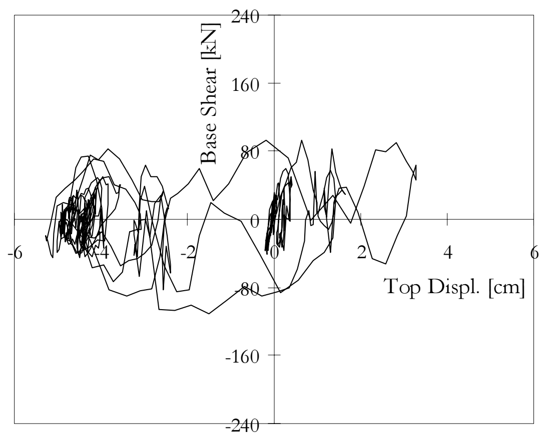


Figure 16. Base-shear/top displacement relationship in the direction of the coupled wall, one edge along the openings in the coupled wall is confined, the other is unconfined

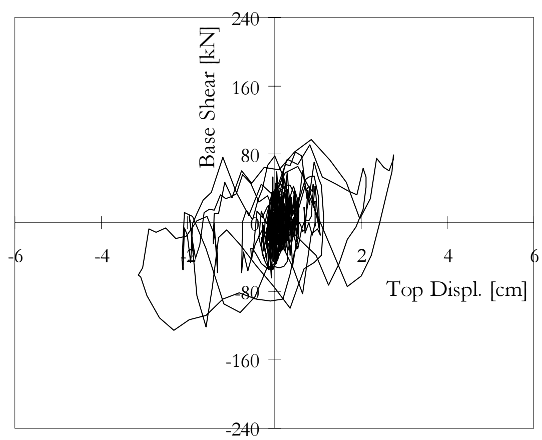


Figure 17. Base-shear/top displacement relationship in the direction of the coupled wall, both edges along the openings in the coupled wall are confined

CONCLUSIONS

The paper summarizes the results of the 15 years of research work at the University in Ljubljana related to seismic behavior and vulnerability of thin – limited ductile structural walls used in buildings with high wall-to-floor ratio. The discussion has been focused on the key question whether or not strict requirements for ductile design of RC structural walls are needed in low to moderate seismic regions. The question is related to the problem of choosing adequate level of response modification (behavior) factor. It has been also investigated what is the resistance, which is provided by the minimum flexural reinforcement required by codes (e.g. Eurocodes) and what level of performance such reinforcement provides?

There is no simple black and white answer to these questions. A number of parameters were considered in the paper, including actual and design level of seismic demand, behavior factor, number of stories, wall-to-floor area ratio, shape of the cross section and structural details. Experimental and field evidence, as well as analytical simulations, parametric studies and seismic assessment methodology developed by Cornell and Krawinkler [16] were considered and used. Majority of the research was limited to 5- and 10-story RC structural walls of rectangular cross section in the areas, where the expected ground acceleration was 0.1 – 0.3 g.

In many cases (practically for all 5-story walls in any of the investigated seismic zones and for all walls in the areas of low (a_{gmax} = 0.1 g) and moderate (a_{gmax} = 0.2 g) seismic demand) the resistance provided by minimum flexural reinforcements was considerably higher than required by code. This demonstrates the fact that many walls possess considerable overstrength related to code requirements. Similar conclusion has been provided by extensive parametric study using pushover based N2 procedure. The response of low (5-story) structures exposed to code design level of earthquake was practically elastic (with low ductility demand) in most cases. The same was valid for all buildings in areas with low seismic demand. Even higher walls in areas of higher seismicity may experience little inelastic demand, if the wall-to-floor area in the building is high (i.e. 2-3 % in each direction).

These observations may imply the conclusion that rigorous requirements providing high ductility may not be justified in many cases. Furthermore, based on similar studies and field observations many researchers (also the first author of this study) advocated for relatively high behavior factors for both coupled walls, as well as cantilever walls.

However, the research presented in this paper has clearly demonstrated that extreme caution is needed. There is an abrupt limit (in the height of the building, wall-to-floor ratio and/or seismic intensity), beyond which even a small increase in load increases demand considerably. Accordingly, approximate seismic performance assessment demonstrated considerable seismic vulnerability (the probability of failure was about 10^{-2} in 50 years) even for the 5-story wall, for which pushover analysis had predicted nearly elastic response at the design level of the earthquake. To understand this, it should be noted that, when the (over)strength of a cantilever wall is exhausted, there is no mechanism to distribute the demand. As a consequence even a small increase of the load results in a large increase in the demand, leading to significant increase of the possibility of failure.

The modes of failure identified in the presented research include (but are not limited to; additional information is expected from the shaking table test of the H-shaped coupled wall scheduled for April 2004):

- Fracture of the (brittle) longitudinal reinforcement in the boundary columns. This is a very serious warning in the time when the market is flooded by cheap brittle reinforcement.

- Compression failure of poorly confined edges of the walls, in particular along the openings in coupled walls.
- The research also demonstrated that substantial variations of axial force could be induced into the cantilever wall due to its rocking response. This considerably influences moment resistance as well as maximum induced shear force.

To investigate such, and similar failure modes special numerical models are needed. Multiple-vertical-line-element macro model proved to be successful in several post-experimental studies and blind predictions, presented in the paper. The in-plane (2D) version has been extended to 3D version.

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