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Seismic shear design of twenty-story RC building with ductile wall system

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ABSTRACT

This paper presents analysis of seismic shear design of twenty-story RC building designed in accordance with EN 1998-1. For this analysis, uncoupled ductile wall system is selected as structural system of building. Preliminary seismic analysis of structure is carried out using modal response spectrum analysis. The nonlinear time-history analysis is performed on the spatial model of the structure where the structure is exposed to seven real earthquake records selected in accordance with the rules defined in EN 1998-1. The subject of performed nonlinear time-history analysis is seismic shear design of DCH ductile walls in accordance with EN 1998-1. The analysis of determining design shear forces using magnification factor and analyses of diagonal compression and diagonal tension failure of the web due to shear for DCH ductile walls are performed. Based on the derived results, corrections for the magnification factor and for shear resistance of ductile walls are proposed. The analysis leads to conclusions regarding the design procedure for "large" ductile walls (L=6.0m), walls that accept the dominant part of seismic force, in relation to the "small" walls (L=3.0m), walls in which minimum reinforcement is relevant.

Contents

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Introduction

Ductile wall system (high ductility class DCH according to EN1998-1 is selected for this analysis) is a very common choice for earthquake-resistant reinforced concrete RC medium-rise and high-rise buildings. In this structural system, DCH walls usually have high seismic design shear forces. Seismic shear design of DCH walls in accordance with EN 1998-1 proved to be a limitation for the application of the DCH wall system.

This happens because the design shear forces V_{Ed} obtained by increasing the shear force obtained from the linear analysis V_{Ed}' using magnification factor ε (see Eq.(1)) is often exceed characteristic values of shear resistance.

$$V_{Ed} = \varepsilon \cdot V_{Ed}^{'}, \text{ where } 1.5 \le \varepsilon = \sqrt{\left(1.2 \cdot \frac{M_{Rdo}}{M_{Edo}}\right)^2 + 0.1 \cdot \left(q \cdot \frac{S_e(T_c)}{S_e(T_1)}\right)^2} \le q \tag{1}$$

The first term in the expression for the magnification factor is intended to account for overstrength due to the development of a single plastic hinge at the base of the wall, while the second term aims at capturing the increase of shear force over the elastic overstrength value represented by the first term due to higher-mode effects. Calculating magnification factor ε in this way, usually high values are obtained (Kappos and Antoniadis, 2007).

On the other hand, there are two charateristic shear failure control in EN 1998-1 for DCH wall. The first shear failure control is related to diagonal compression failure of the web. The design shear force V_{Ed} must be less than the design value of the shear resistance of ductile walls controlled by diagonal compression in the web $V_{Rd,max}$.

In the critical region of the ductile walls the design value of the shear resistance $V_{Rd,max}$ is 40% of the value outside of critical region. This large reduction of the design shear resistance $V_{Rd,max}$, when applied together with the magnification of shears by the magnification factor ε, might be prohibitive for the usage of ductile concrete walls in earthquake-resistant buildings.

The second shear failure control is related to diagonal tension failure of the web. The design shear force V_{Ed} must be less than the design value of the shear resistance of ductile walls controlled by diagonal tension in the web $V_{Rd,s}$. This kind of failure is rarer than diagonal compression failure. In view of the lack of information specific for this shear failure under the cyclic loading, the modification of the rule given in Eurocode 2 for the calculation of the shear reinforcement in members with $0.5<\alpha_s<2$ (α_s is shear ratio) under monotonic loading (short shear spans element) has also been adopted for the determination of $V_{Rd,s}$ in EN 1998-1. Thus defined, the rule is very conservative. Eurocode authors agree in the statement that there is certainly room for future improvement of these rule, once more data become available on the cyclic behaviour and failure of low-shear-span-ratio walls by diagonal tension (Fardis et al.,2005).

In this paper, analysis of seismic shear design of DCH walls in accordance with EN 1998-1 is performed in the example of twenty-story RC building with ductile wall system. For this purpose the nonlinear time-history analysis is performed on the spatial model where the structure is exposed to seven real earthquake records selected in accordance with the rules defined in EN 1998-1. The relevant design values of shear force for seven selected earthquake records represent the mean value of the individual shear forces obtained from independent ground motion records V_{na} is compared to: design shear force V_{Ed}, the design value of the shear resistance of ductile walls controlled by the diagonal tension in the web $V_{Rd,s}$ and the design value of the shear resistance of ductile walls controlled by diagonal compression in the web $V_{Rd,max}$. Based on the derived results, corrections when computing the magnification factor ε and controlling the shear resistance of ductile walls are proposed.

The analysis leads to conclusions regarding the design procedure for "large" ductile walls (L=6.0m), walls that accept the dominant part of seismic force, in relation to the "small" walls (L=3.0m), walls in which minimum reinforcement is relevant [1-19].

Description of the considered RC structure

RC structure that is analysed in this paper, presents twenty-story building with total height of 60.0m and the storey heigh of 3.0m. Floor plan of the building is shown on Fig.1. Building has dimensions in the base 28.0m x 18.0m. Structural system of building is uncoupled ductile wall system. The floor structure is RC monolithic slab with thickness of 14cm. Thicknesses of the walls are 30cm and 40cm. The class of concrete is adopted C40/50 and C45/55 according to Eurocode 2 for all elements. Yield strength of steel for longitudinal and transverse reinforcement is f_y=400 MPa and f_y=500 MPa for mesh reinforcement. The building is founded on the ground type C. The maximum horizontal ground acceleration for the considered location of the object is 0.32g for the return period of 475 years.

Design of the considered structure was done according to EN 1998-1 (CEN, 2004). Preliminary seismic analysis of structure was carried out using a multi-modal response spectrum analysis. The fundamental periods of structure for X and Y direction, obtained from linear analysis, are T_{1x}=2.592s and T_{1y}=1.954s. The building is designed for DCH ductility class (Bisch et al., 2012), (Fardis and Tsionis, 2011). The total seismic forces of the structure resulting from multi-modal spectrum analysis in the X direction is 6654kN, while in the Y direction is 6807kN.

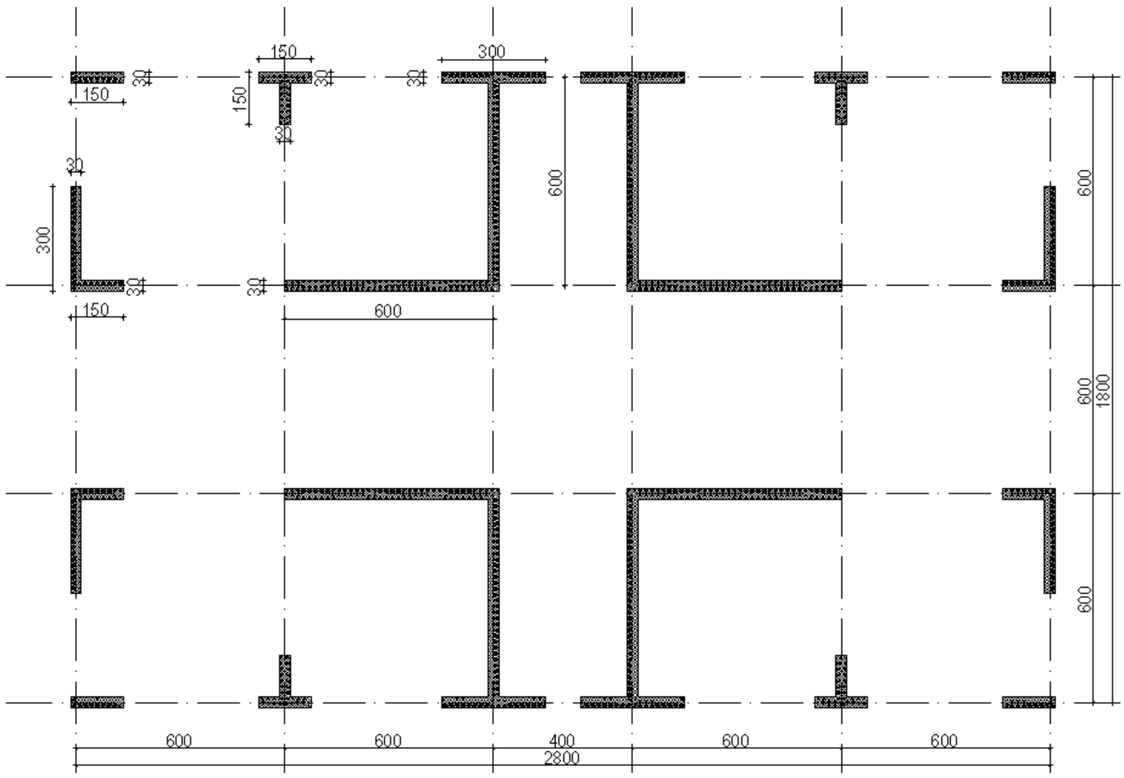

Figure 1. Floor plan of the building

Modeling of RC structure for nonlinear analysis

For the purpose of performing nonlinear time-history analysis, model of the structure was created using *Perform-a 3D* (Perform 3D Product of Computers & Structures, Inc., 2006). The nonlinear model of the structure is designed as a spatial model. The model includes strength of structural elements and their post-elastic behaviour.

Element properties are based on mean values of the properties of the materials. Stress-strain relationship for unconfined concrete, confined concrete and reinforcement steel are adopted accordance with the recommendations of EN 1998.

Shear wall element (ASCE41, 2007) was used for the modeling of wall. Walls are modeled by defining the cross-section composed of a number of fibers. The area and location of reinforcement within the cross-section and the properties of the concrete are defined using individual fibers. Concrete and reinforcing steel are modeled with nonlinear characteristics.

For purpose of estimating structure performance capacity of the plastic hinge deformation is calculated in accordance with EN 1998 for Collapse prevent performance level. Calculated capacities of deformation are in the range 0.005-0.01 radians.

Selection of ground motions

Ground motions used in this paper for nonlinear time-history analysis are chosen from European strong-motion database (Ambraseys et al., 2000). Selection of ground motions are made in accordance with EN 1998-1.

It is concluded that is very difficult to find seven ground motions that meet the provisions of EN 1998-1. Mean response spectrum that significantly deviates from the Eurocode elastic response spectrum is obtained with arbitrary choice of seven ground motions from European strong-motion database. The problem of choosing the earthquake records in accordance with the rules from EN 1998-1 has been recognized by a number of researchers (Iervolino et al.,2008). For this reason, REXEL software (Iervolino et al.,2009) is used as helpfull tool for selecting ground motions. There are found seven ground motions on the soil type C with this software, scaled to the value of agS (soil factor S is 1.15 for soil type C) whose mean response spectrum roughly correspond to target eurocode’s elastic response spectrum.

These motions were characterized by surface-wave magnitudes, M, in the range between 5.5 and 6.6 and epicentral distance, R, between 8 and 35 km. Selected seven ground motions with the their basic characteristics are shown in Table.1.

Table 1. Selected ground motions with their basic characteristic

Earthquake Name	Date	M	Epicentral Distance R [km]	PGA_X [m/s^2]	PGA_Y [m/s^2]
Alkion	2/24/1981	6.6	19	2.8382	1.6705
Friuli (aftershock)	9/15/1976	6.0	9	1.0686	0.9324
Dinar	10/1/1995	6.4	8	2.6739	3.1306
Ishakli (aftershock)	2/3/2002	5.8	35	0.394	0.5069
Adana	6/27/1998	6.3	30	2.1575	2.6442
Cubuklu	4/20/1988	5.5	34	0.4095	0.4439
Izmit (aftershock)	9/13/1999	5.8	26	0.6464	0.512

Then, mean response spectrum of selected seven ground motions which are scaled according to the rules defined in EN 1998-1 is constructed for each of the directions of earthquake (Fig.2, Fig.3) The scaling factor for the ground motions in the X direction is 1.194, and for the ground motions in the Y direction is 1.00. Both horizontal components of ground motions are taken to acting simultaneously.

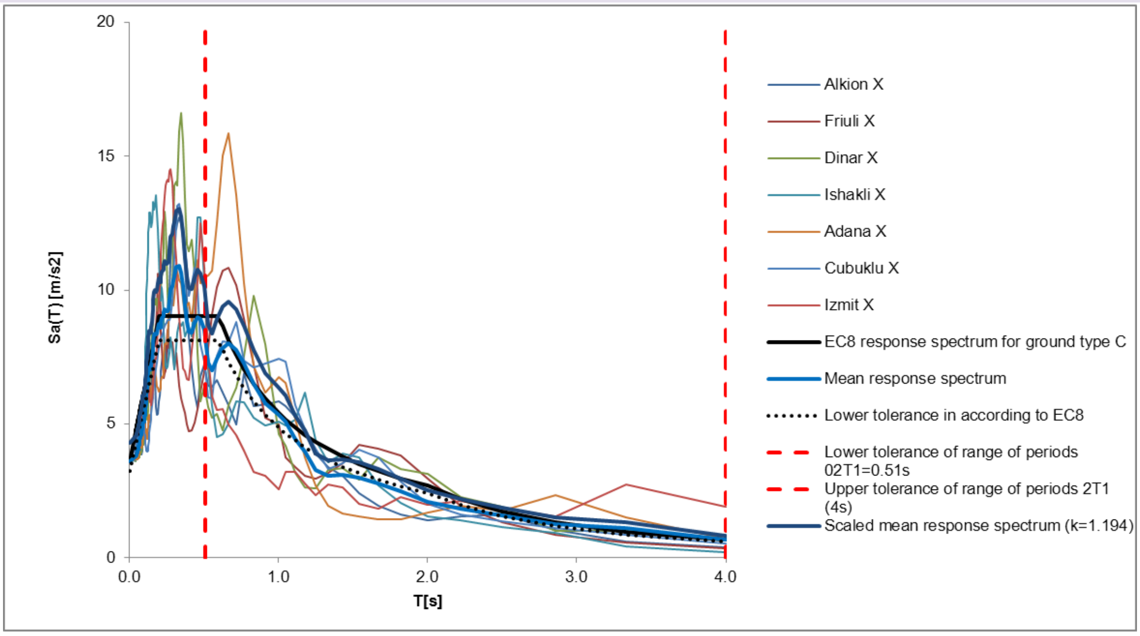

Figure 2. Selection of ground motions according the rules defined in EN 1998-1 for X direction

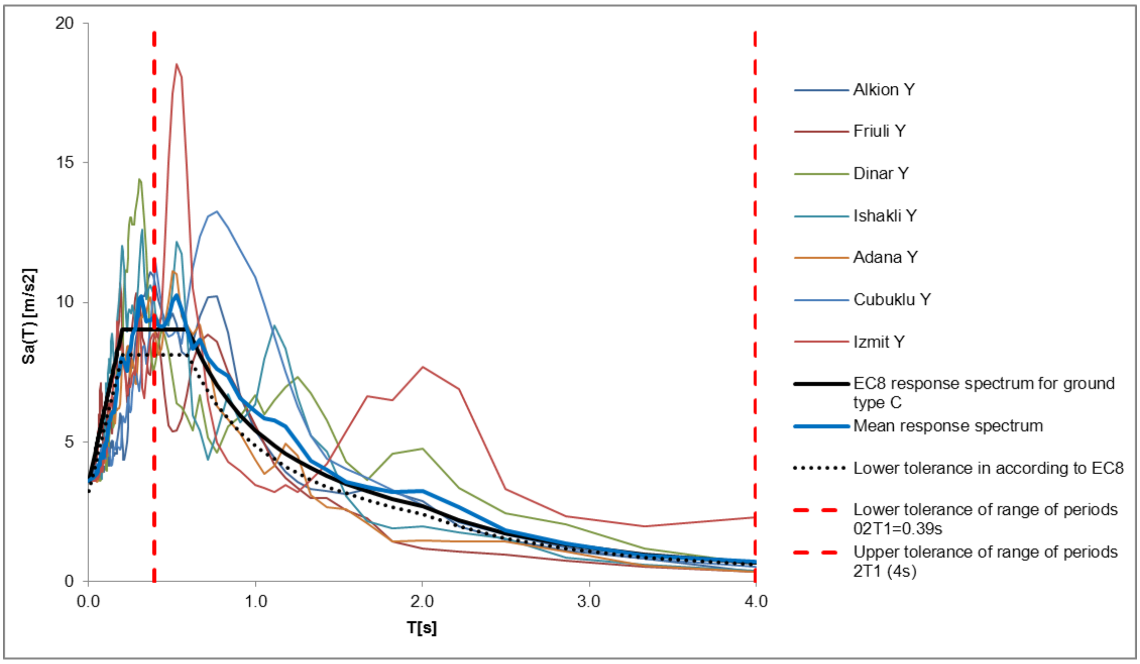

Figure 3. Selection of ground motions according the rules defined in EN 1998-1 for Y direction

Results of nonlinear time-history analysis

The nonlinear time-history analysis is performed on the spatial model where the structure is exposed to seven real ground motions selected in accordance with the rules defined in EN 1998-1.

Characteristic shear force diagrams of ductile walls for considered building are constructed and the appropriate conclusions are made. The design envelope of shear forces V_{Ed} is calculated according to EN 1998-1 using a magnification factor ε and shear force from the analysis V_{Ed}'. The relevant design values of shear force for seven selected earthquake records represent the mean value of the individual shear forces obtained from independent ground motion records V_{na} is compared to: design shear force V_{Ed}, the design value of the shear resistance of ductile walls controlled by the diagonal tension in the web $V_{Rd,s}$ and the design value of the shear resistance of ductile walls controlled by diagonal compression in the web $V_{Rd,max}$.

Characteristic shear force diagrams of large ductile wall (L=6.0m) for considered building are shown on Fig.4 (a) and (b). The mean value of shear forces from all of the individual seven ground motions V_{na} along the height of the wall deviates from design shear forces V_{Ed} calculated in accordance with EN 1998-1 as presented in diagram on Fig.4(a). In the lower stories deviation is minimum, while in the upper stories deviations are higher. Fig.5 shows the variations along the height of the wall of the following ratios V_{na}/V_{Ed}', V_{na}/V_{Ed}, $V_{na}/V_{Rd,max}$ and $V_{na}/V_{Rd,s}$. The diagram relations V_{na}/V_{Ed} shows that in the lowest story $V_{na}/V_{Ed} = 0.85$, while in the upper stories this ratio decreases to a value of 0.49 (the largest deviations are from the 11th floor to the 13th floor). These differences in the relation V_{na}/V_{ed} over the height are due to deviations of actual distribution of shear forces over the height of the wall obtained from seven ground motions to the distribution of shear forces from the linear analysis. From the results of nonlinear analyses it is evident that the deviations in the distribution of shear forces over the height exist although shear forces is obtained using more accurate modal response spectrum analysis. These results indicate that it is necessary to consider the usage of unique and constant magnification factor ε in determining design shear forces.

The relation V_{na}/V_{Ed}' at the lowest level is 3.72, which is significently higher than the factor 1.2 from the first term in the expression for the magnification factor ε, $1.2M_{Rdo}/M_{Edo}$. This is attributed to higher realized flexural resistance at the base of the wall ($M_{Rdo}>>M_{Edo}$). In calculating in the linear analysis, for the computation of the magnification factor ε is taken that $M_{Rdo}=M_{Edo}$ respectively flexural resistance at the base of the wall is approximately the same as the design bending moment at the base of the wall for the relevant seismic design situation. As the vertical longitudinal reinforcement at the base of the wall is controlled by the case in which the bending moment from the alalysis, M_{Edo}, is combined with the minimum axial compression, the flexural capacity when the maximum axial compression is considered at the base, M_{Rdo}, is much greater than M_{Edo}. Then, the value of ε may be so high that the verification of the individual walls in shear (especially failure by diagonal compression) may be unfeasible.

(a) **(b)**

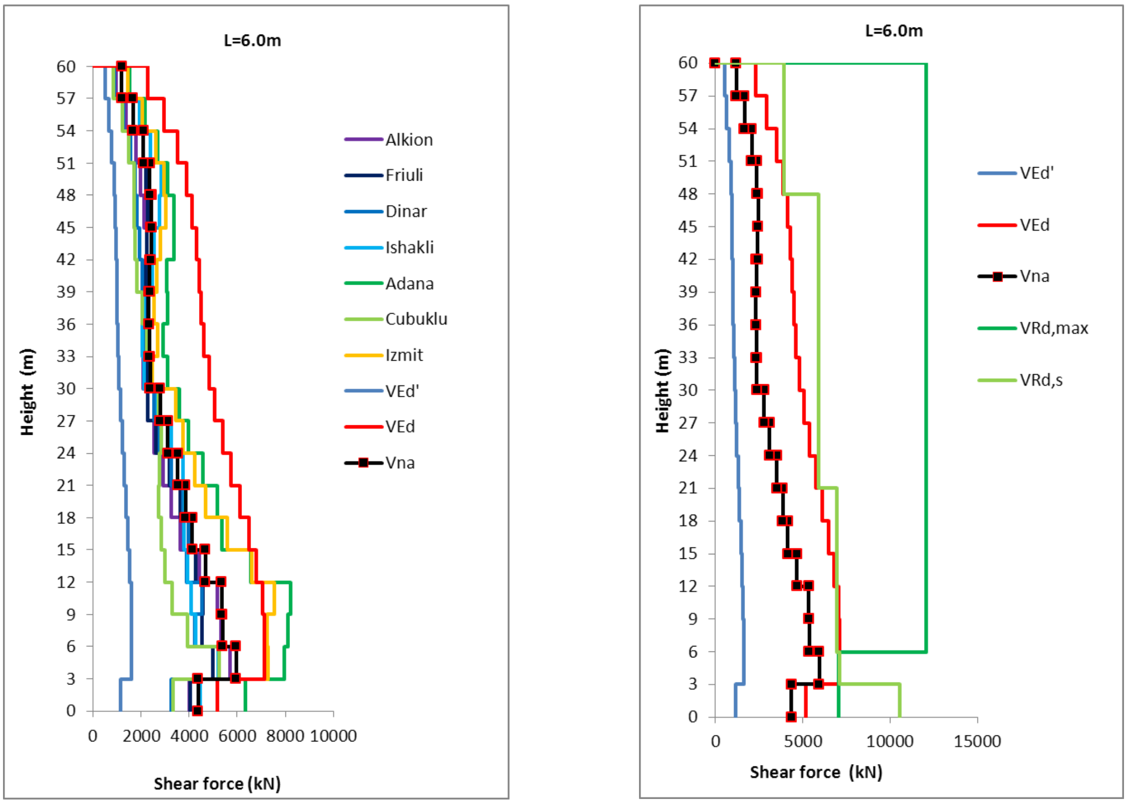

Figure 4. Shear force diagrams for ductile large (L=6.0m) wall: (a) $V_{Ed'}$, V_{Ed}, V_{na}; (b) $V_{Ed'}$, V_{Ed}, V_{na}, $V_{Rd,max}$, $V_{Rd,s}$

While determining the magnification factor ε for considered wall, it can be seen that a much larger contribution to the magnification factor ε is just from second term which relating to the higher-mode effects. According to some researchers, this term may even be excluded in the case when using modal response spectrum analysis for the calculation of seismic forces. In this paper, although the modal response spectrum analysis is used, it is interesting to note that second term had more significant impact on the final value of the magnification factor ε. In determining the relation M_{Rdo}/M_{Edo}, it has been taken into account the same seismic design situation for the case of determining the M_{Edo} and M_{Rdo} (the same value of the axial compression). Ratio M_{Rdo}/M_{Edo} is 1, so the impact of the first term in expression for the magnification factor ε with respect to second term is not important. Thus calculated magnification factor ε is not logical and consistent with the meaning and purpose of the individual terms. The second term in the expression for the magnification factor ε, when the calculation of seismic force used modal response spectrum analysis should be excluded from the equation and keep it only when the calculation of seismic force used methods of lateral force.

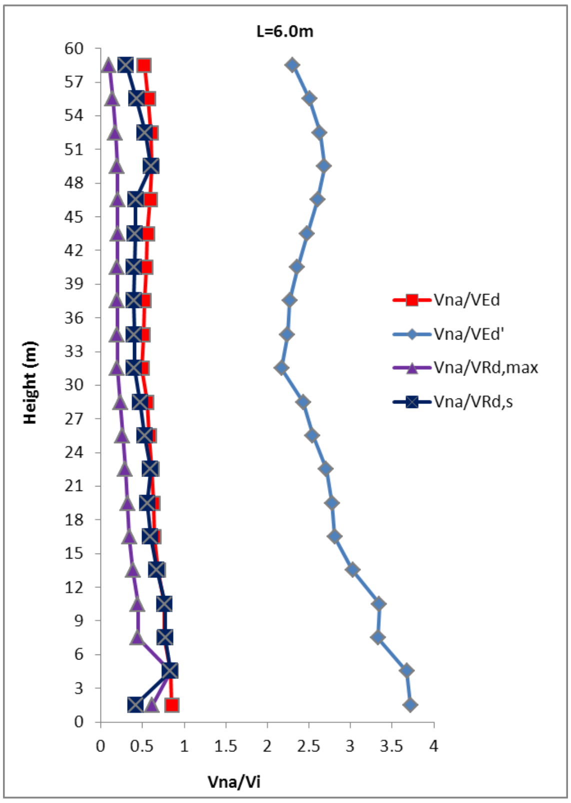

Figure 5. Ratios $V_{na}/V_{Ed'}$, V_{na}/V_{Ed}, $V_{na}/V_{Rd,max}$, $V_{na}/V_{Rd,s}$ for ductile large (L=6.0m) wall

The diagram in Fig.4(b) shows that the mean value of shear force from all of shear forces obtained from independent ground motions V_{na} in all srories over the height of the wall is less than the the design value of the shear resistance of ductile walls controlled by diagonal compression in the web $V_{Rd,max}$. In the critical region of the wall, ratio V_{na}/V_{Ed} is 0.84 (outside the critical region the ratio is even lower), so the design shear force could be reduced by 16% in this area and even higher outside the critical region. This indicates that the design shear forces could be smaller.

The diagram in Fig.4(b) shows that the mean value of shear force V_{na} in all stories over the height of the wall is less than the the design value of the shear resistance of ductile walls controlled by diagonal tension in the web $V_{Rd,s}$. This kind of failure is rarer than diagonal compression failure. In view of the lack of information specific for this shear failure under the cyclic loading, the modification of the rule given in Eurocode 2 for the calculation of the shear reinforcement in members with $0.5<\varepsilon_s<2$ under monotonic loading (short shear spans element) has also been adopted for the determination of $V_{Rd,s}$ in EN 1998-1. Value of $V_{Rd,s}$, defined on this way is very conservative with regard to the adoption of horizontal and vertical reinforcement in the ductile wall with the shear ratio $\alpha_s<2$. Unrealistically large and unnecessary amounts of horizontal and vertical reinforcement in the ductile wall of considered structure are obtained, which leads to undesirable performance of structure for selected earthquake records (developing of plastic hinges on the upper floors). For this reason revised structure, that has a smaller amount of vertical reinforcement in the ductile walls, include in analysis and compare it with original structure. Revised structure has a better behavior under seismic design earthquake records than original.

Comparison of results for the original and revised structure are shown in Table.2, wherein D/C ratios (demand/capacity ratios) for certain deformation limit states are presented. In the case of the revised structures tension strains and rotations in the plastic hinge zones are increased and tension strains outside the plastic hinge zone are reduced. Significant rotations is moved in hinge region in the case of the revised structure.

Table 2. Comparations of results for the orginal and revised structure

	D/C ratios	
Deformation limit state	**Original structure**	**Revised structure**
Rotations in plastic hinges	0.40	0.63
Tension strain, plastic hinge region	0.21	0.60
Compression strain, plastic hinge region	0.18	0.30
Tension strain, outside plastic hinge region	1.00	0.50
Compression strain, outside plastic hinge region	0.40	0.25

Characteristic shear force diagrams of small ductile wall (L=3.0m) for considered building are shown on Fig.6 (a) and (b). The mean value from all of the individual seven ground motions V_{na} along the height of the wall deviates from design shear forces V_{Ed} calculated in accordance with EN 1998-1 as presented in diagram on Fig.6 (a). At the base of the wall shear force V_{na} is less than the design shear force V_{Ed} (V_{na}=0.75V_{Ed}) while in the upper stories V_{na} exceeds the design values of shear force V_{Ed}. This excess is happened because in “small” ductile walls minimum reinforcement is relevant. For that reason, realized flexural resistance at the base of the wall is greater than design bending moment at the base of the wall.

(a) **(b)**

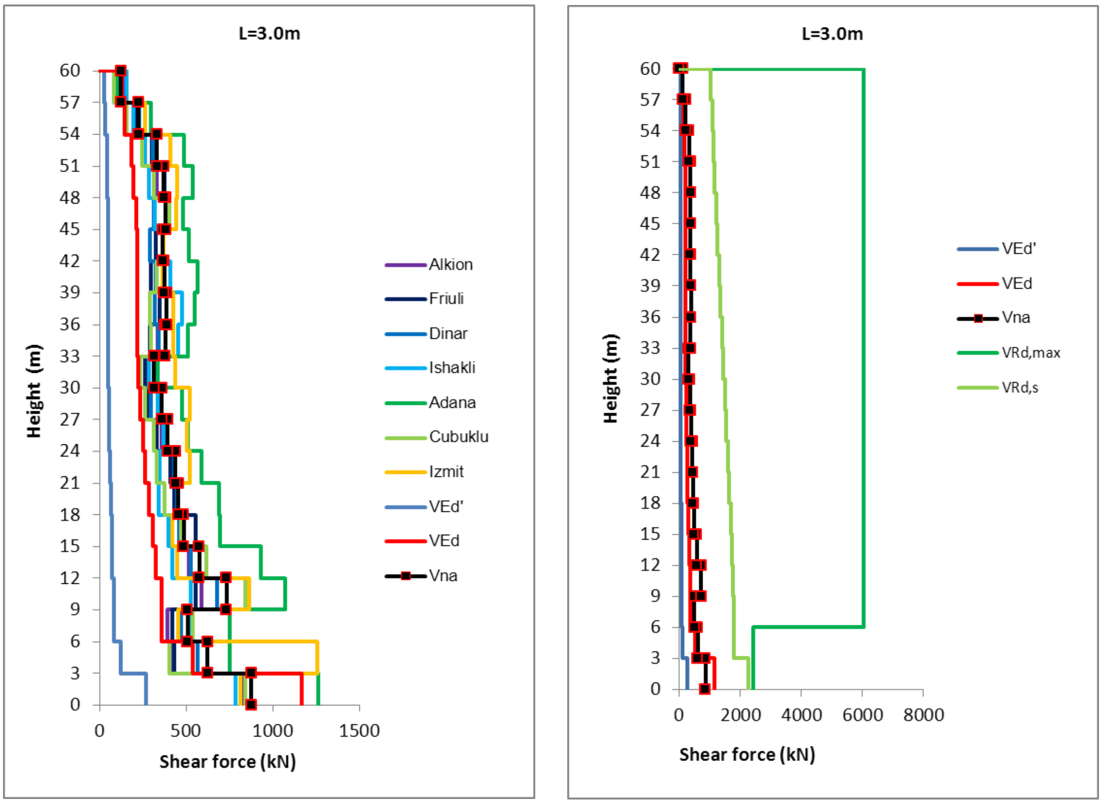

Figure 6. Shear force diagrams for ductile small (L=3.0m) wall: (a) $V_{Ed'}$, V_{Ed}, V_{na}; (b) $V_{Ed'}$, V_{Ed}, V_{na}, $V_{Rd,max}$, $V_{Rd,s}$

On the other hand, it is interesting to notice although design envelope of shear force V_{Ed} is not adequate calculated (V_{na} value is greater than the design shear force V_{Ed}), V_{na} does not exceed characteristic value of shear resistance $V_{Rd,max}$ and $V_{Rd,s}$ in any section of wall (Fig.6(b)). Conservative minimum reinforcement requirements compensated for the unconservatism in the estimation of design shears in the small ductile walls.

Comparing the results obtained for the case of “large” walls (L= 6.0m) with the results obtained for “small” walls (L=3.0m) the following conclusion can be carried out. Calculation procedure that correspond to "large" ductile walls, walls that accept the dominant part of seismic force, are conservative when applied to the case of "small" ductile walls, walls in which the minimum reinforcement is relevant.

Conclusions

The main conclusions carried out on the example of the considered structure are as follows:

Computation of $V_{Rd,s}$ is conservative when using the modified equations and conditions of EN 1992-1-1:2004 related to the calculation of the shear reinforcement in short shear spans with direct strut action, as defined in EN 1998-1. Value of $V_{Rd,s}$ defined in this way is very strict with regard to the adoption of horizontal and vertical reinforcement in the ductile wall in the case when the shear ratio α_s<2. Unrealistically large and unnecessary amounts of horizontal and vertical reinforcement in the ductile wall of considered structure are obtained, which leads to undesirable performance of structure for selected earthquake records (developing of plastic hinges on the upper floors). Revised structure, that has a smaller amount of vertical reinforcement in the ductile walls, has a better behavior under seismic design earthquake records.

Calculation procedure that correspond to "large" ductile walls, walls that accept the dominant part of seismic force, are conservative when applied to the case of "small" ductile walls, walls in which the minimum reinforcement is relevant. In "small" ductile wall (L = 3.0m), although design envelope of shear force V_{ed} is not adequate calculated (V_{na} value is greater than the design shear force V_{ed}), V_{na} does not exceed characteristic value of shear resistance $V_{Rd,max}$ and $V_{Rd,s}$ in any section of wall. Conservative minimum reinforcement requirements compensated the unconservatism in the estimation of design shears in the small walls. In EN 1998-1 should be recognised diference procedure for shear calculation of "large" walls and "small" walls.

The second term in the expression for the magnification factor ε, which aims to capture the increase of shear force due to higher-mode effects, should be excluded from the equation when the calculation of seismic force with modal response spectrum analysis is used and keep it only when lateral force is used.

Analysis of buildings of a similar structural system and different heights, that is in progress, should confirm this results.

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Сейсмический расчет на сдвиг двадцатиэтажного монолитного здания с дуктильными стенами

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Информация о статье	История	Ключевые слова
УДК 69	Подана в редакцию 10 мая 2015 Принята 30 мая 2015	железобетонное здание, нелинейный анализ, поперечная сила, железобетонная стена, Еврокод 8

АННОТАЦИЯ

Тема данного исследования - анализ расчета на сдвиг армированного монолитного двадцатиэтажного здания, спроектированного по стандарту EN 1998-1. Для анализа выбрано здание с конструктивной схемой с дуктильными стенами. Преламинарный сейсмический рачет конструкции сделан с помощью мультимодульного спектрального анализа. Нелинейный динамический анализ сделан на пространственной модели конструкции, причем, данная конструкция подвергалась действию семи различных реальных землетрясений, имеющихся в записи. Анализ произведен в соответствии с правилами EN 1998-1. Цель сделанного нелинейного анализа - анализ расчета на сдвиг стеновых поверхностей высокого класса дуктильности в соответствии с EN 1998-1. На основании полученных результатов предложены корректировки для случая, когда расчет фактора увеличения "е" находится под вопросом. Данный анализ приводит к выводам о процедуре расчета для протяженных армированно-монолитных стен (L=6.0 m), которые доминируют при восприятии сейсмических реакций по отношению к коротким армированно-монолитным стенам (L=3.0 m), с относительно малым содержанием арматуры.

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