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COMPARISON OF DUCTILITY CLASS REQUIREMENTS FOR SEISMIC DESIGN OF REINFORCED CONCRETE WALLS IN A TALL BUILDING

Luka Čičak (1), Anđelko Vlašić (2), Dominik Skokandić (3)

(1) *Mag.Ing.Aedif., Studio Arhing l.t.d., luka@studio-arhing.com*
(2) *Assoc. Professor, University of Zagreb Faculty of Civil Engineering, andjelko.vlasic@grad.unizg.hr*
(3) *Post Ph.D. Assistant, University of Zagreb Faculty of Civil Engineering, dominik.skokandic@grad.unizg.hr*

Abstract

The seismic design of reinforced concrete walls is greatly influenced by the selected ductility class. The ductility class directly determines the reduction of seismic loading by reducing the response spectrum using an appropriate behaviour factor. A higher ductility class can only be achieved by following more stringent design rules related to the type, amount, and detailing of reinforcement. In this paper, an overview of the differences in the design criteria required to achieve a medium (DCM) and high (DCH) ductility class is presented. The difference in behaviour factor between these two ductility classes is 50%. Three buildings of different heights (40 m, 50 m, and 60 m) in which reinforced concrete walls are arranged around a central core are analysed. The dimensions of the core and the thickness of the walls were chosen primarily to achieve the required stiffness of the building and to meet the conditions for total and inter-story displacements. Seismic loads for ductility classes DCM and DCH were calculated for all buildings, and a seismic response spectrum analysis was performed. Finally, a seismic design of selected walls was performed for each ductility class according to EN 1998-1-1. The results show a difference in the required amount of reinforcement, and its placement, depending on the reinforcement type (B500B or B500C). The main difference in design was found to be the resistance to sliding shear failure, which requires additional angled reinforcement for DCH ductility class. Examples of reinforcement detailing are shown graphically for each ductility class. A conclusion is drawn regarding the advantages of choosing the highest ductility class, taking into account the cost of reinforcement.

Keywords: tall building, shear walls, reinforced concrete, seismic analysis, ductility

1. Introduction

Given the resurgence of seismic activity in Croatia in recent years, there is increasing interest in researching various earthquake-related topics. In Croatian practise, the modelling and design of structures is usually done according to the DCM rules, which place the structure in the medium ductility class. The high ductility class DCH is not a common requirement for structures, and for this reason there are very few case studies on its benefits and related design challenges.

The structural system with the reinforced concrete core and surrounding frames is one of the most widely used in high-rise building construction. The use of shear walls is equally represented in both high and low buildings. They are important parts of the structure that contribute significantly to lateral stiffness. They can be compared in their behaviour to vertical consoles, i.e., it can be said that these are "high consoles" fixed to the foundation. Together with the columns, they contribute to the transfer of vertical loads, and thus have a significant normal force. In some structures, coupled shear walls may be required, where the beams and the floor system connect the two or more walls together as a coupled system to provide greater stiffness. In tall buildings, shear walls are usually located in the centre of the building and typically form a central core that houses the vertical communication system such as stairwells and elevators. As such, they are a very common form of lateral load support system in tall buildings [1].

The response of high-rise buildings in earthquake areas is an essential criterion for seismic design. If the structure is "too stiff", greater internal forces will occur, and if it is "too soft", excessive displacements will occur. By properly selecting the type, quantity, position, and dimensions of

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structural elements, we can achieve a balance between these two limitations. According to research [2] carried out on more than 2500 high-rise buildings in China, a diagram of typical relationships between building heights and fundamental vibration periods was given. Using this diagram, we can estimate how the building will react based on its stiffness.

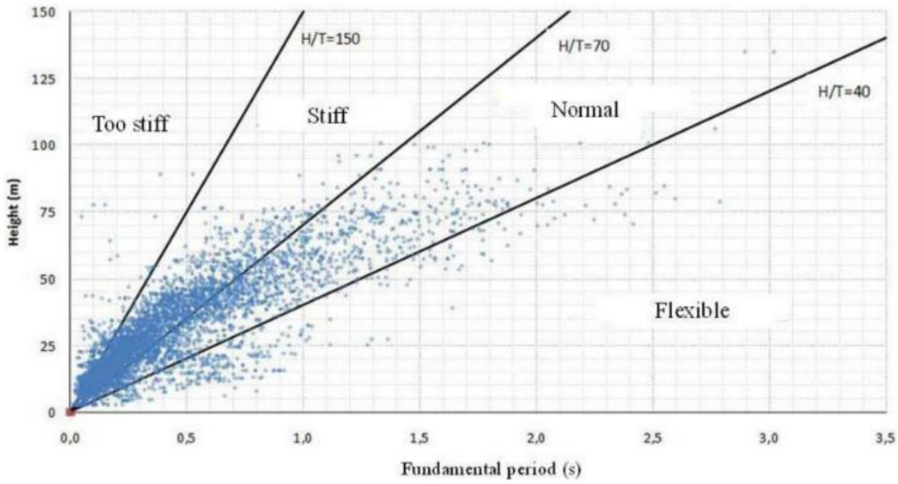

Figure 1. Relationship between fundamental periods and structural heights [2]

2. Ductility classes DCM and DCH

The ductility of the structure reflects its ability to maintain load-bearing capacity high in the plastic range, exhibiting high deformations before failure. Ductile structures are particularly important in seismic areas due to their energy dissipation properties, reflected by high enclosed area of the load-displacement hysteresis loops. Ductile behaviour in reinforced concrete structures can be achieved by special reinforcement design and detailing which results in adequate longitudinal reinforcement, stirrups for confinement of the compressed sections, and transverse or inclined reinforcement to prevent shear failure. The goal is to achieve ductile wall failure mode by yielding of the tensile reinforcement and wide cracks in the tension zone, rather than brittle compressive failure with crushing of the concrete and buckling of the reinforcement. In addition, shear failure modes such as diagonal tension and diagonal compression, as well as sliding shear in the region of the plastic joint, must be prevented. Sliding shear is attributed to the relatively low allowable compression, which is limited to only 40% of the concrete compressive strength for DCH according to the provisions of the Eurocode. By applying the rules for design and detailing of a selected ductility class, the desired behaviour of the structure can be achieved. As far as ductility class is concerned, HRN EN 1998-1 [3] makes the biggest difference in the design for shear, where DCH has much stricter rules compared to DCM. The multiplication factor for the design seismic shear force derived from the analysis is 1.5 for DCM, while for DCH it is calculated according to the expression (Table 1) and is limited to a value less than or equal to the behaviour factor. According to EN 1998 [3], the resistance to sliding shear failure must be checked for ductility class DCH. This resistance is composed of three parts: the resistance of the vertical bars, the resistance of the inclined bars and the frictional resistance. According to these design rules, the design was carried out for two case study walls and the results are presented in the continuation.

Table 1 (part 1) – EN 1998 rules for the detailing and dimensioning of ductile walls [4]

	DCH	DCM	DCL
Web thickness, $b_{w0} \geq$	max(150mm, h_{storey}/20)		-
critical region length, h_{cr}	$\geq$ max(l_w, H_w/6) $\leq$ min($2l_w$, h_{storey}) if wall $\leq$ 6 storeys $\leq$ min($2l_w$, $2h_{storey}$) if wall > 6 storeys		-

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Table 1 (part 2) – EN 1998 rules for the detailing and dimensioning of ductile walls [4]

	DCH	DCM	DCL
Boundary elements:			
a) in critical region:			
- length l_c from edge $\geq$	0.15lw, 1.5bw, length over which ε_c>0,0035		-
- thickness b_w over o $l_c \geq$	0.2m; h_{st}/15 if $l_c \leq \max(2b_w, l_w/5)$, h_{st}/10 if $l_c > \max(2b_w, l_w/5)$		
- vertical reinforcement:			
ρ_{min} over $A_c=l_cb_w$	0,5%		0,2%
ρ_{max} over A_c	4%		
- confining hoops (w):			
$d_{bw} \geq$	6mm, $0.4(f_{yd}/f_{ywd})^{1/2}d_{bL}$	6mm	in the part of the section where ρ_L>2: as over the rest of the wall (case b, below)
spacing $s_w \leq$	$6d_{bL}$, $b_0/3$, 125mm	$8d_{bL}$, $b_0/2$, 175mm	
$\omega_{wd} \geq$	0,12	0,08	
$\alpha\omega_{wd} \geq$	$30\mu_\phi(\nu_d+\omega_v)\varepsilon_{sy,d}b_w/b_0-0{,}035$		
b) over the rest of the wall height:	In parts of the section where ε_c>0,2%: $\rho_{v,min}$=0,5%; elsewhere 0,2% In parts of the section where ρ_L >2%: distance of unrestrained bar in compression zone from nearest restrained bar $\leq$150mm; hoops with $d_{bw} \geq \max(6mm, d_{bL}/4)$ & spacing $s_w \leq \min(12d_{bL}, 0.6b_{w0}, 240mm)$ up to a distance of $4b_w$ above or below floor beams or slabs, or $s_w \leq \min(20d_{bL}, b_{w0}, 400mm)$) beyond that distance		
Web:			
- vertical bars (v):			
$\rho_{v,min}$	Wherever in the section ε_c>0,2%: 0,5%; elsewhere 0,2%		0,2%
$\rho_{v,max}$	4%		
$d_{bv} \geq$	8mm	-	
$d_{bv} \leq$	$b_{w0}/8$	-	
spacing $s_v \leq$	$\min(25d_{bv}, 250mm)$	$\min(3b_{w0}, 400mm)$	
- horizontal bars:			
$\rho_{h,min}$	0,2%	$\max(0.1\%, 0.25\rho_v)$	
$d_{bh} \geq$	8mm	-	
$d_{bh} \leq$	$b_{w0}/8$	-	
spacing $s_h \leq$	$\min(25d_{bh}, 250mm)$	400mm	
axial load ratio $\nu_d=N_{Ed}/A_cf_{cd}$	$\leq$0,35	$\leq$0,4	-
Design moments M_{Ed}:	If $H_w/l_w \geq 2$, design moments from linear envelope of maximum moments M_{Ed} from analysis for the "seismic design situation", shifted up by the "tension shift" a_l		from analysis for design seismic action & gravity

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Table 1 (part 3) – EN 1998 rules for the detailing and dimensioning of ductile walls [4]

	DCH	DCM	DCL
Shear design:			
Design shear force V_{Ed} = shear force V'_{Ed} from the analysis for the design seismic action, times factor ε	if $H_w/l_w \leq 2$: $\varepsilon = 1.2 M_{Rd0}/M_{Ed0} \leq q$ if $H_w/l_w > 2$: $\varepsilon = \sqrt{\left(1.2\frac{M_{Rd0}}{M_{Ed0}}\right)^2 + 0.1\left(q\frac{S_e(T_C)}{S_e(T_{C1})}\right)^2} \leq q$	ε=1.5	ε=1.0
$V_{Rd,max}$ outside critical region	As in EC2: $V_{Rd,max} = 0{,}3(1 - f_{ck}(MPa)/250)b_{w0}(0{,}8l_w)f_{cd}sin2\delta$, $1 \leq cot\delta \leq 2{,}5$		
$V_{Rd,max}$ in critical region	40% of EC2 value	As in EC2	
$V_{Rd,s}$ in critical region; web reinforcement ratios: ρ_h, ρ_v			
(i) if $\alpha_s = M_{Ed}/V_{Ed}l_w \geq 2$: $\rho_v = \rho_{v,min}$, ρ_h from $V_{Rd,s}$:	$V_{Rd,s} = b_{w0}(0{,}8l_w)\rho_h f_{ywd}$	As in EC2: $V_{Rd,s} = b_{w0}(0{,}8l_w)\rho_h f_{ywd} cot\delta$, $1 \leq cot\delta \leq 2{,}5$	
(ii) if $\alpha_s < 2$: ρ_h from $V_{Rd,s}$	$V_{Rd,s} = V_{Rd,c} + b_{w0}\alpha_s(0{,}75l_w)\rho_h f_{yhd}$	As in EC2: $V_{Rd,s} = b_{w0}(0{,}8l_w)\rho_h f_{ywd} cot\delta$, $1 \leq cot\delta \leq 2{,}5$	
ρ_v from:	$\rho_v f_{yvd} = \rho_h f_{yhd} - N_{Ed}/(0{,}8l_w b_{w0})$		
Resistance to sliding shear: via bars with total area A_{si} at angle α to the horizontal	$V_{Rd,s} = A_{si}f_{yd}cos\alpha +$ $A_{sv}\min\left(0{,}25f_{yd}, 1{,}3\sqrt{(f_{yd}f_{cd})}\right) +$ $0{,}3(1 - f_{ck}(MPa)/250)b_{w0}f_{cd}$		

3. Worked examples

3.1 Buildings for comparison – layout and height

A comparison of the analysis and design is given for buildings with three different heights, all of which have the same fixed arrangement of elements in the floor plan. The heights vary between 40 m, 50 m and 60 m. Fig. 2 shows the arrangement of the elements in the floor plan of each building. The floor plan is symmetrical in both directions, with a central reinforced concrete core and frames arranged around it. The reinforced concrete core is the primary horizontal load-bearing element and consists of a closed walled section 8.0 x 8.0 m in size. Access to the core, which houses all vertical communications is provided through two openings in opposing walls, thus creating a coupled wall system. The openings in the walls of the core are 2.0 x 2.1 m in size. The height of the floor ranges from 2.95 m to 3.1 m, depending on the height of the building. Since the focus of the design is on the walls, their thickness varies depending on the height of the model and the position of the walls along the height of the building. For the 40-meter-high building, the walls of the first five floors are 40 cm thick, and the walls of the next eight floors have a thickness of 30 cm. In the 50-meter building model, the first seven floors have a wall thickness of 50 cm, and the remaining ten floors have a wall thickness of 40 cm. In the last model, the tallest building, the first eight floors have a wall thickness of 60 cm, and the remaining eleven floors have a wall thickness of 50 cm. These wall thicknesses were chosen according to the dynamic analysis to calibrate the behaviour of the building to the expected periods of the modal shapes and to limit the amount of compressive stress in the wall according to the ductility class requirements. Inter storey drift and displacements at the top of the buildings were also checked and limited to H/500 restriction. All columns have the same constant cross-section, 50 x 50 cm, throughout the entire height of the building in all models. The dimensions of the beams are 50 x 50 cm and are the same throughout the height of

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the building. The horizontal diaphragms (slabs) are 16 cm thick. All vertical elements are fixed at the bottom to simulate a rigid underground foundation level, which was not modelled (Fig. 3). The material used was concrete of class C50/60.

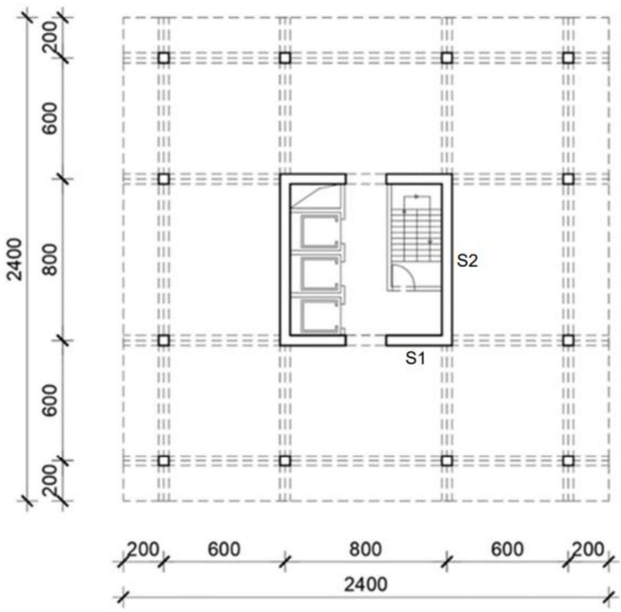

Figure 2. Characteristic floor plan of the building considered in the design

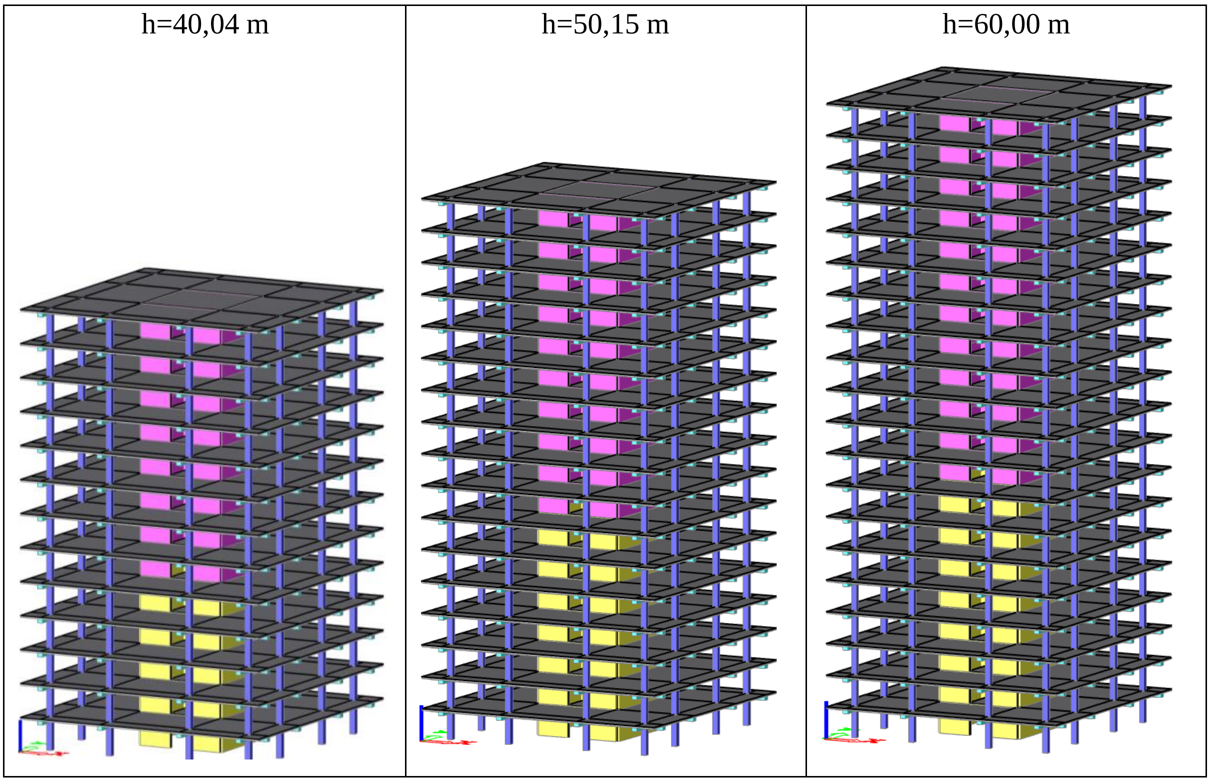

Figure 3. 3D view of three models

3.2 Modal analysis and behaviour

The behaviour factor q was determined according to the instructions given in EN 1998-1-1 and is calculated as 3.6 for DCM, and 5.4 for DCH. Based on these values, two spectres were defined, and a modal analysis was performed in the software. The 3D numerical model was developed using finite element method (FEM) in software for static analysis. All load bearing elements are defined in the model, while the non-bearing ones are taken into account with additional dead load. Walls and slabs are defined using shell elements, with the mesh size of 25x25 cm, while the 1D beam elements were defined with corresponding cross sections. All vertical elements are placed on rigid supports (in node for 1D and on bottom edge for 2D elements) on the bottom of the ground floor. The modulus of elasticity of the horizontal diaphragms, the slabs, is 37300 MPa. According to HRN EN 1998-1-4.3.1(7), the modulus of elasticity of all other concrete elements is to be taken as half the value, 18650 MPa, due to the assumption of the cracked concrete sections. Modal spectral analysis is performed, with 20 eigenvalues for each of the three models, to be in accordance with the minimum number of modes, equal to the $3\sqrt{n}$, where the n is the number of floors. Tables 2-4 show the results of the modal analysis, and Fig. 4 shows the design spectra with plotted corresponding values of buildings first periods.

Table 2 – Modal analysis results for building h = 40 m

Mode	Omega [rad/s]	Period [s]	Freq. [Hz]	W_{xi}/W_{xtot}	W_{yi}/W_{ytot}	W_{zi}/W_{ztot}	W_{xi_R}/W_{xtot_R}	W_{yi_R}/W_{ytot_R}	W_{zi_R}/W_{ztot_R}
1	6,87508	0,91	1,09	0,6911	0	0	0	0,2565	0,0001
2	7,4443	0,84	1,18	0	0,6507	0	0,2921	0	0,0005
3	8,22718	0,76	1,31	0,0001	0,0004	0	0,0002	0	0,7817
4	23,5615	0,27	3,75	0,1727	0	0	0	0,3901	0,0001
5	24,2208	0,26	3,85	0,0002	0	0	0,0001	0,0005	0,1198
6	28,1011	0,22	4,47	0	0,0921	0,001	0,3625	0	0
7	28,7503	0,22	4,58	0,0002	0,0003	0,6738	0,0005	0,0015	0
8	29,76	0,21	4,74	0,0081	0,0001	0,0152	0,0001	0,0496	0
9	32,158	0,2	5,12	0	0,1181	0,0001	0,0281	0	0
10	40,3878	0,16	6,43	0	0	0	0	0	0,0421
11	46,4411	0,14	7,39	0,0512	0	0	0	0,0939	0
12	55,93	0,11	8,9	0	0	0	0	0	0,0201
13	60,6207	0,1	9,65	0	0	0,0642	0	0,0005	0
14	60,8107	0,1	9,68	0	0,0258	0	0,0962	0	0
15	61,6163	0,1	9,81	0,0002	0	0,0026	0	0,0127	0
16	64,2709	0,1	10,23	0	0,0389	0	0,0286	0	0
17	68,8677	0,09	10,96	0,0226	0	0	0	0,0558	0,0001
18	69,1251	0,09	11	0,0001	0	0	0	0,0002	0,0128
19	69,8406	0,09	11,12	0,0001	0	0,0202	0	0,0001	0
20	71,5292	0,09	11,38	0	0	0	0	0	0
				0,9466	0,9263	0,7773	0,8084	0,8616	0,9772

Table 3 – Modal analysis results for building h = 50 m

Mode	Omega [rad/s]	Period [s]	Freq. [Hz]	W_{xi}/W_{xtot}	W_{yi}/W_{ytot}	W_{zi}/W_{ztot}	W_{xi_R}/W_{xtot_R}	W_{yi_R}/W_{ytot_R}	W_{zi_R}/W_{ztot_R}
1	5,16589	1,22	0,82	0,6806	0,0001	0	0	0,2878	0
2	5,5102	1,14	0,88	0,0001	0,6409	0	0,3242	0	0,0001
3	6,87628	0,91	1,09	0	0,0001	0	0,0001	0	0,7782
4	18,3599	0,34	2,92	0,1794	0	0	0	0,3623	0
5	20,501	0,31	3,26	0	0,0001	0	0,0002	0,0001	0,1158
6	22,8246	0,28	3,63	0	0,155	0	0,3286	0	0,0001
7	24,3841	0,26	3,88	0,0001	0,0001	0,6635	0	0,0006	0
8	25,4948	0,25	4,06	0,0048	0	0,0094	0	0,026	0
9	26,7396	0,23	4,26	0	0,052	0,0003	0,0023	0	0
10	34,9582	0,18	5,56	0	0	0	0	0	0,0423
11	37,2832	0,17	5,93	0,0506	0	0	0	0,0995	0
12	49,163	0,13	7,82	0	0	0	0	0	0,0218
13	51,6847	0,12	8,23	0	0,06	0	0,1335	0	0
14	54,1018	0,12	8,61	0,0002	0	0,0478	0	0,0018	0
15	54,5276	0,12	8,68	0,0007	0	0,0215	0	0,003	0
16	55,4021	0,11	8,82	0	0,0087	0,0001	0,0004	0	0
17	56,5972	0,11	9,01	0,0256	0	0	0	0,0657	0
18	61,5051	0,1	9,79	0	0	0	0	0	0,0129
19	66,9404	0,09	10,65	0	0	0,0216	0	0	0
20	69,0877	0,09	11	0,0001	0	0	0	0,0023	0
				0,9422	0,9169	0,7643	0,7894	0,8492	0,9712

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Table 4 – Modal analysis results for building h = 60 m

Mode	Omega [rad/s]	Period [s]	Freq. [Hz]	W_{xi}/W_{xtot}	W_{yi}/W_{ytot}	W_{zi}/W_{ztot}	W_{xi_R}/W_{xtot_R}	W_{yi_R}/W_{ytot_R}	W_{zi_R}/W_{ztot_R}
1	4,19184	1,5	0,67	0,6663	0,0002	0	0,0001	0,3126	0
2	4,31474	1,46	0,69	0,0002	0,6339	0	0,3422	0,0001	0
3	6,35601	0,99	1,01	0	0	0	0	0	0,7822
4	15,7529	0,4	2,51	0,1894	0	0	0	0,3382	0
5	18,9387	0,33	3,01	0	0,0206	0	0,0322	0	0,0981
6	19,0886	0,33	3,04	0	0,165	0	0,261	0	0,0124
7	21,9378	0,29	3,49	0	0	0,6545	0	0,0002	0
8	23,1493	0,27	3,68	0,0041	0	0,0059	0	0,0115	0
9	23,8354	0,26	3,79	0	0,0189	0,0003	0,0004	0	0
10	32,2195	0,2	5,13	0,0001	0	0	0	0,0003	0,041
11	32,7902	0,19	5,22	0,0525	0	0	0	0,1068	0,0001
12	44,318	0,14	7,05	0	0,0682	0	0,1349	0	0
13	45,5048	0,14	7,24	0	0	0	0	0	0,0216
14	48,943	0,13	7,79	0,0012	0	0,0193	0	0,0008	0
15	49,3369	0,13	7,85	0,001	0	0,0491	0	0	0
16	49,6946	0,13	7,91	0	0,0022	0,0004	0,0008	0	0
17	50,3042	0,12	8,01	0,0241	0	0,0003	0	0,0673	0
18	57,3445	0,11	9,13	0	0	0	0	0	0,013
19	63,8374	0,1	10,16	0	0	0,0215	0	0	0
20	65,1029	0,1	10,36	0,0002	0	0,0003	0	0,001	0
				0,9393	0,9092	0,7517	0,7715	0,8387	0,9684

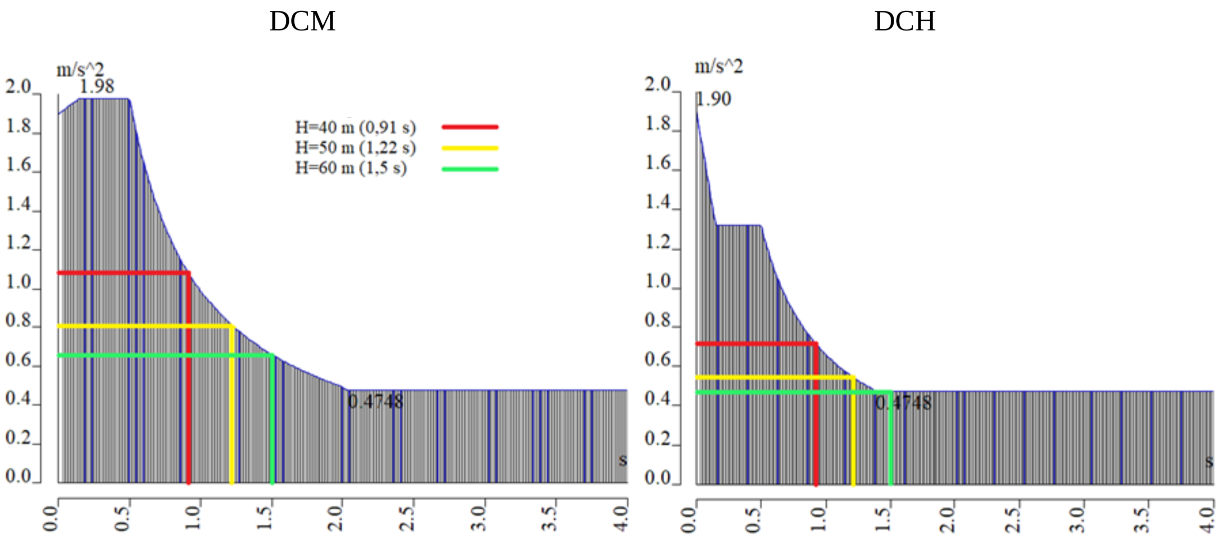

Figure 4. The relationship between the period and the response for each height of the building and ductility class

4. Reinforcement comparison for different ductility classes

Design and comparison of the results was done for two intersecting walls on the ground floor of each model - S1 shorter wall and S2 longer wall (Fig. 2). Reinforcement design results and comparison can be observed in Table 5. The main difference in the reinforcement concerning the ductility class is in the stirrups spacing. The curvature ductility factor μ_φ varies greatly depending on the final ductility of the structure we want to achieve, and it will affect the stirrups spacing. For DCM either B500B or B500C reinforcement can be used, while for DCH only B500C reinforcement is allowed. If B500B reinforcement steel is used, curvature ductility factor μ_φ must be taken 50% higher which places it close to curvature ductility factor μ_φ according to DCH design (Table 6). Thus, there is only a small difference in the stirrup spacing between DCM and DCH when B500B is used in DCM design compared to the case when B500C is used for both DCM and DCH design. Furthermore, to achieve a high ductility class (DCH) it is necessary to meet shear resistance requirements which are much demanding in relation to the requirements for the medium class of ductility (DCM). Conditions for both ductility classes are shown in Table 1 (part 3). Fig. 5 shows reinforcement for DCM design of a 50 m high building.

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Table 5 – Comparison of design reinforcement for DCM and DCH ductility class design

Model/Wall	Web		Boundary elements		
	Vertical bars	Horizontal bars	Vertical bars	Stirrups	Inclined bars
H40/S1 (DCM)	±Ø12/10 cm	±Ø12/10 cm	14Ø16	Ø12/12 cm	-
H40/S1 (DCH)	±Ø12/10 cm	±Ø12/10 cm	12Ø16	Ø12/8 cm	±7Ø32
H50/S1 (DCM)	±Ø12/10 cm	±Ø12/10 cm	16Ø16	Ø12/12 cm	-
H50/S1 (DCH)	±Ø12/10 cm	±Ø12/10 cm	16Ø16	Ø12/9 cm	±9Ø32
H60/S1 (DCM)	±Ø12/9 cm	±Ø12/9 cm	18Ø16	Ø12/12 cm	-
H60/S1 (DCH)	±Ø12/10 cm	±Ø12/10 cm	18Ø16	Ø12/9 cm	±8Ø32
H40/S2 (DCM)	±Ø12/10 cm	±Ø12/10 cm	28Ø16	Ø12/18 cm	-
H40/S2 (DCH)	±Ø12/10 cm	±Ø12/10 cm	28Ø16	Ø12/9 cm	±16Ø28
H50/S2 (DCM)	±Ø12/10 cm	±Ø12/10 cm	24Ø20	Ø12/18 cm	-
H50/S2 (DCH)	±Ø12/10 cm	±Ø12/10 cm	24Ø20	Ø12/10 cm	±17Ø28
H60/S2 (DCM)	±Ø12/9 cm	±Ø12/9 cm	28Ø20	Ø12/20 cm	-
H60/S2 (DCH)	±Ø12/10 cm	±Ø12/10 cm	28Ø20	Ø12/12 cm	±15Ø28

Table 6 – The curvature ductility factor and stirrups spacing for each ductility class and reinforcement type

Model	DCM/DCH (B500B/B500C)	Wall	Curvature ductility factor μ_φ	Stirrups
H40	DCM (B500B)	S1	9,3	Ø12/8 cm
		S2	9,3	Ø12/10 cm
	DCM (B500C)	S1	6,2	Ø12/12 cm
		S2	6,2	Ø12/18 cm
	DCH (B500C)	S1	9,8	Ø12/8 cm
		S2	9,8	Ø12/9 cm
H50	DCM (B500B)	S1	9,3	Ø12/9 cm
		S2	9,3	Ø12/10 cm
	DCM (B500C)	S1	6,2	Ø12/12 cm
		S2	6,2	Ø12/18 cm
	DCH (B500C)	S1	9,8	Ø12/8 cm
		S2	9,8	Ø12/10 cm
H60	DCM (B500B)	S1	9,3	Ø12/9 cm
		S2	9,3	Ø12/12 cm
	DCM (B500C)	S1	6,2	Ø12/12 cm
		S2	6,2	Ø12/20 cm
	DCH (B500C)	S1	9,8	Ø12/9 cm
		S2	9,8	Ø12/12 cm

Sliding shear resistance is particularly important in this comparison because this proof is needed only in DCH design. Sliding shear can occur only in the region of a plastic hinge and particularly at the position of a construction joint (Fig. 6) [5]. The design shear force is to be multiplied with a factor ε which is 1.5 for DCM, while the expression for DCM yields a much larger factor equal to DCH behaviour factor of 5.4. To achieve a high sliding shear resistance needed for such a large design shear force, it is necessary to use an angled "X" shaped reinforcement in the region of the plastic hinge (Fig. 6). Without this reinforcement it is impossible to meet the sliding shear design requirements for DCH ductility class. Fig. 7 shows DCH reinforcement with bidiagonal bars for a 50 m high building.

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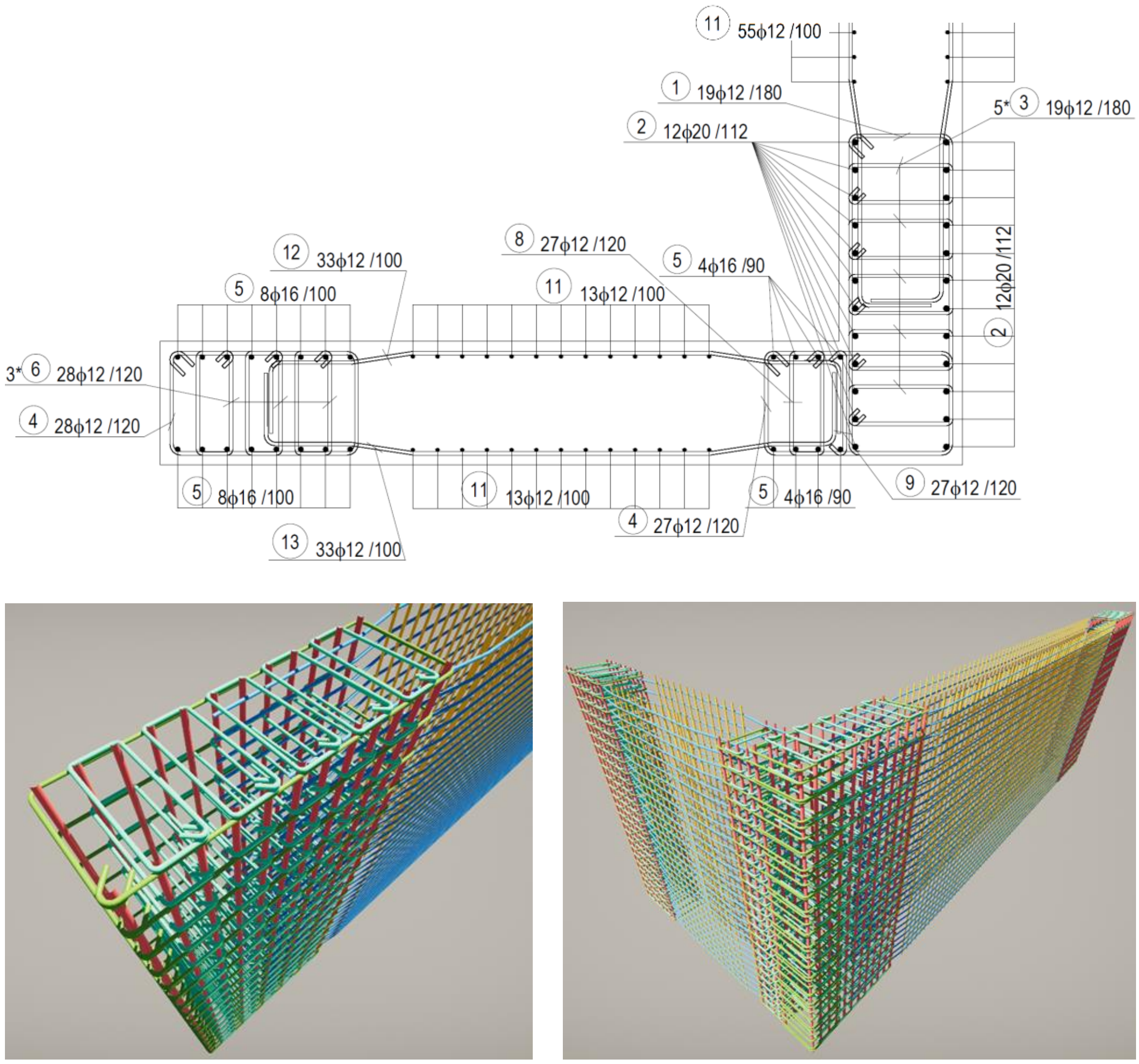

Figure 5. Reinforcement: boundary element and the crossing of two walls (DCM design, building height 50 m)

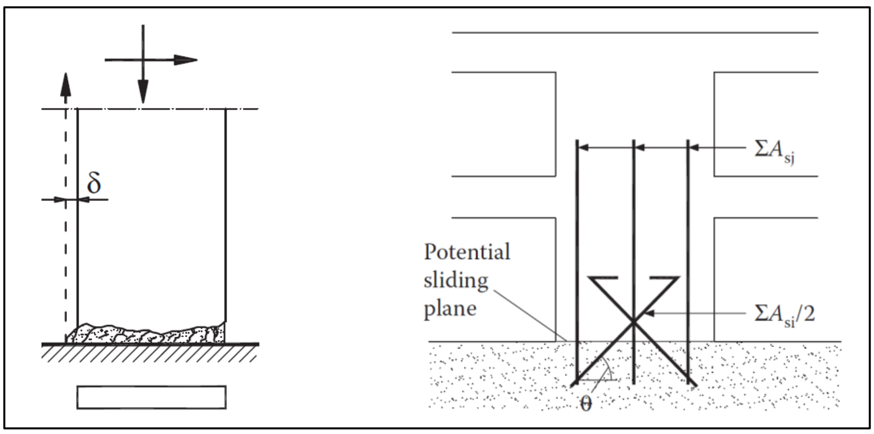

Figure 6. Sliding shear failure and bidiagonal X shaped reinforcement for added resistance [5]

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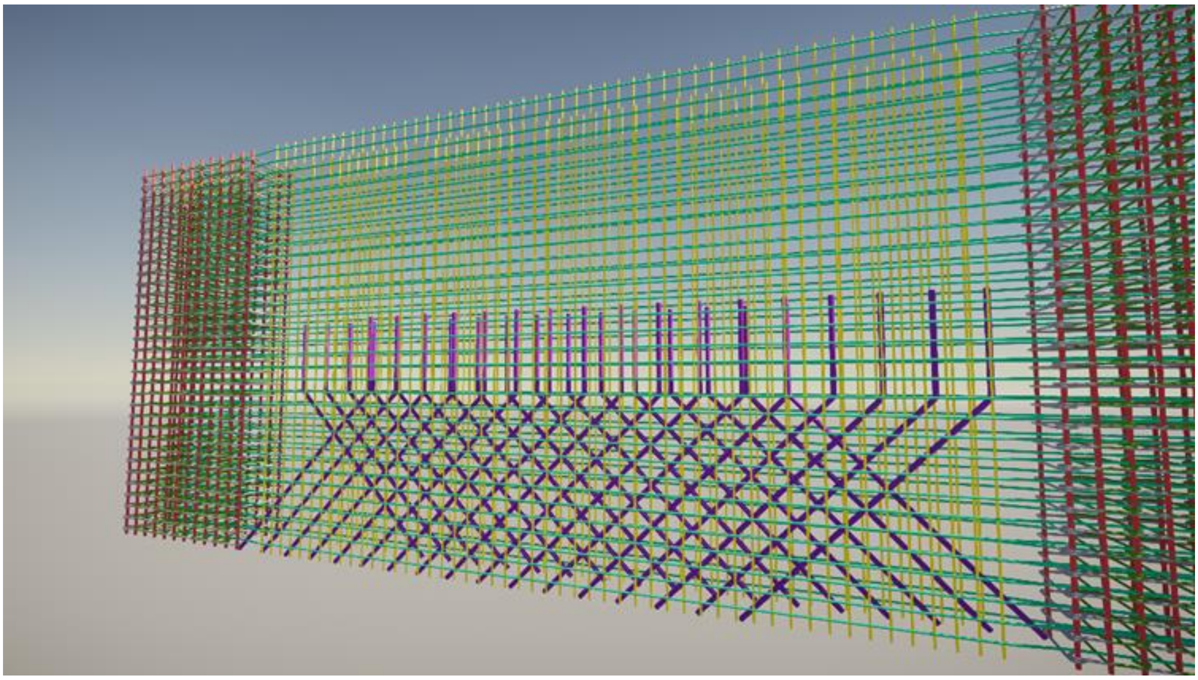

Figure 7. Bidiagonal X shaped reinforcement for sliding shear (DCH design)

5. Conclusion

Worked examples show that when the element is designed according to the DCM rules with the use of B500B reinforcing steel compared to the design of the same element according to the DCH rules with B500C reinforcing steel, the only major difference is the need for inclined reinforcement to prevent sliding shear failure. The DCM design, such verification is not required, so no inclined reinforcement is needed. When reinforcement steel class B500C is used for both designs, the possible stirrup spacing in the boundary element is significantly larger when designing according to the DCM rules. The reason for this is the required ductility, which is higher for the DCH design than for the DCM design when B500C class reinforcing steel is used. The required ductility depends on the behaviour factor and the first period. Since the behaviour factor for DCM and DCH is not the same, the required ductility is also not the same. Using B500B steel in the DCM design according to HRN EN 1998-1 [3] increases the required ductility by 50%, which brings it significantly closer to the ductility calculated for DCH. This ultimately leads to equal spacing of stirrups in the boundary element.

As mentioned earlier, one of the main differences in the design between DCM and DCH is the design for shear. The DCH requires a much more stringent constraint on shear resistance compared to the DCM method. Under the DCH design, shear resistance must satisfy three checks: diagonal compression failure of the web due to shear, diagonal tension failure of the web due to shear, and sliding shear failure. The DCM method also provides checks for diagonal compression and diagonal tension failure of the web due to shear, but the major difference is in the factorization of the shear force. In DCM, the factor by which the design shear force from the seismic analysis is multiplied is 1.5, while in DCH it is much higher and is limited only to the maximum value of q (Table 1).

At the time of writing, the price difference between B500B and B500C reinforcing steel is about 15%, with B500C being more expensive. If we add the necessary inclined reinforcement, the placement of which significantly complicates the execution and concreting, thus increasing the construction costs, the economic viability of such a design and construction for the seismic zone as analysed in this work, is questionable.

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