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SEISMIC DESIGN OF LOW-RISE RC SHEAR WALL BUILDINGS IN SOUTH ICELAND LOWLAND

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ABSTRACT

The purpose of this study is to compare three prominent sets of standards for seismic design of new structures: The Eurocode 8, (CEN, 2004). US standards: ASCE/SEI 7-05 (ASCE,2005), International Building Code (ICC, 2009) and ACI 318-08 (ACI,2008) and the New Zealand standard (NZS, 2004). The standards are compared for the design of two low-rise RC wall structures in two case studies. The building sites selected are in a high seismic risk area on South Iceland Lowland with near fault effects from vertical acceleration.

The buildings were designed according to Eurocode 8 using both ductility class medium, DCM, and large lightly reinforced walls, LLRW, which is a special design class for low-rise buildings and is in fact a subset within ductility class DCM. For the New Zealand Standard the buildings were designed using a ductility class for structures of limited ductility (SLD). The building design according the US standard assumed a seismic design category (SDC) E, which requires the use of special reinforced shear walls (SRSW) and regularity in the mass and stiffness distribution of the structures.

It can be concluded from the case studies that the LLRW design option according to Eurocode 8, gives quite reasonable reinforcement ratio compared to the Eurocode 8 DCM shear walls. It is therefore recommended to use the LLRW design for low-rise shear wall buildings if they fulfil the established requirements for its application.

All three standards use inter-storey drifts as a performance requirement to prevent severe damage to structural elements and non-structural elements e.g. partitions and cladding. The buildings are well within the allowable drift limits set by the standards, as may be expected for low-rise RC wall buildings with high wall-to-floor surface ratio.

INTRODUCTION

Buildings in Iceland are mostly low-rise cast-in-place concrete shear wall buildings and over 90% of them are four storeys or lower. Large majority of the concrete buildings are constructed after the Second World War. These low-rise wall buildings have plain concrete shear walls at the perimeter

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and reinforced concrete slabs. From 1920 – 1960, only important buildings, such as schools, hospitals and governmental buildings had reinforced concrete (RC) walls. This building practice continues to this day, except with the improvement that almost all low-rise wall buildings have reinforced concrete walls since the late seventies. This building practice is different from many other earthquake prone countries where RC frame structures are commonly used instead of shear wall structures. Low-rise shear wall buildings have large horizontal stiffness against lateral loading and therefore provide an efficient resistance to earthquake action, at least in zones of moderate seismicity. Also, they generally have a reserve strength capacity because for practical reasons their thickness is usually greater than needed to resist common design loads. Low-rise shear wall buildings performed well in three large earthquakes in the South Iceland Lowland of magnitude M_w 6.6 and M_w 6.5 in June 2000 and a M_w 6.3 in May 2008 as discussed in the following and shown in figure 1.

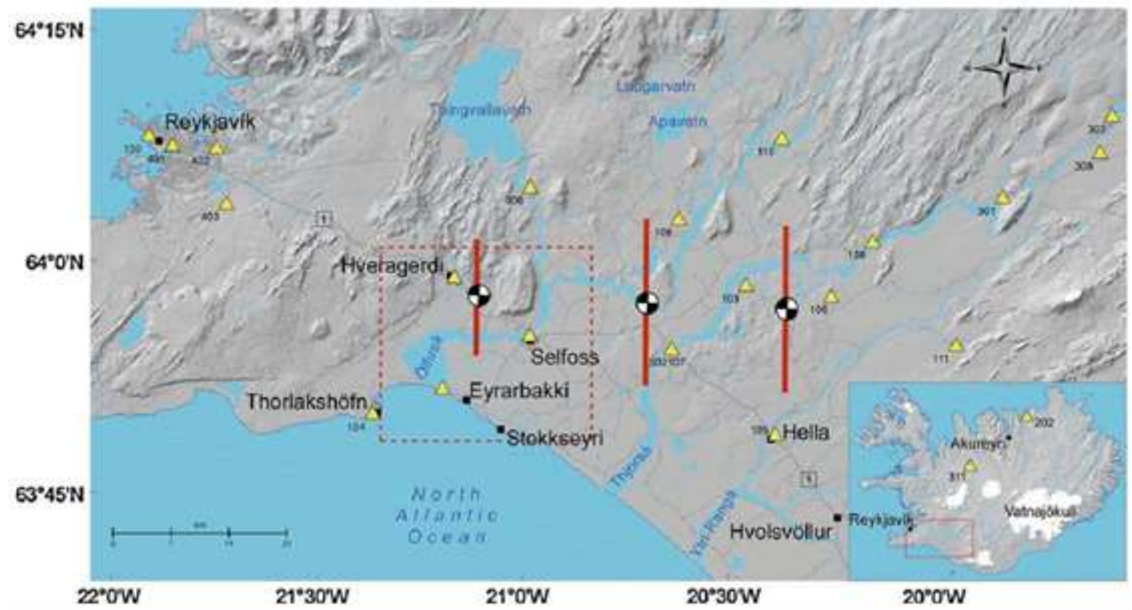

Figure 1. A map showing the South-Iceland-Seismic-Zone, the epicenters of the Earthquakes in 2000 and 2008 and the location of the town of Selfoss (Sigbjörnsson et al., 2009). The triangles show the locations of the recording stations of the Icelandic Strong-Motion Network.

Only relatively few poorly build low-rise wall buildings less than 15 km from the epicenter suffered damage beyond repair. These buildings were commonly older concrete or masonry buildings, with little or no reinforcement (Sigurbjörnsson, et al., 2007). The most common failure mode was otherwise uplifting of foundations and bottom floor slabs residing on gravel.

This good performance of lightly-reinforced shear wall buildings in Iceland is partly due to their excessive strength, high wall-to-floor surface ratio and also the fact that the earthquakes were of moderate magnitude with relatively short duration in spite of high acceleration levels. Therefore the need for ductile behaviour through plastic hinge forming at base of the wall was not really required.

On the other hand, most earthquake design codes relay on ductility as it is common in many countries that buildings have few shear walls bearing a small part of the vertical loads. This led to the development of model for design where these walls act as vertical beams subjected to flexure under the seismic action, which is essentially horizontal. With the assumption that each wall is suitably anchored in its foundation, a plastic hinge may develop as its base. The design method (ductile walls) is included in the Eurocode 8 as in others earthquake engineering design standards. However Eurocode 8 also includes a design option that is not found in other building standards, i.e. the Large, Lightly Reinforced Walls (LLRW) option mentioned before (Bisch & Coin, 2007).

EARTHQUAKES IN ICELAND

Iceland is situated in the North Atlantic Ocean at the junction of two large submarine structures, the Mid-Atlantic Ridge and the Greenland–Iceland–Faeroe Ridge. The present day seismotectonic activity of Iceland is primarily related to the Mid-Atlantic Ridge, which crosses the island and shifts eastward through two major fracture zones, one in the south called the South Iceland Seismic Zone, and one in the north, commonly called the Tjörnes Fracture Zone. All major damaging earthquakes in Iceland have originated within these two zones (Sigurbjörnsson, et al., 2007) as shown in figure 2.

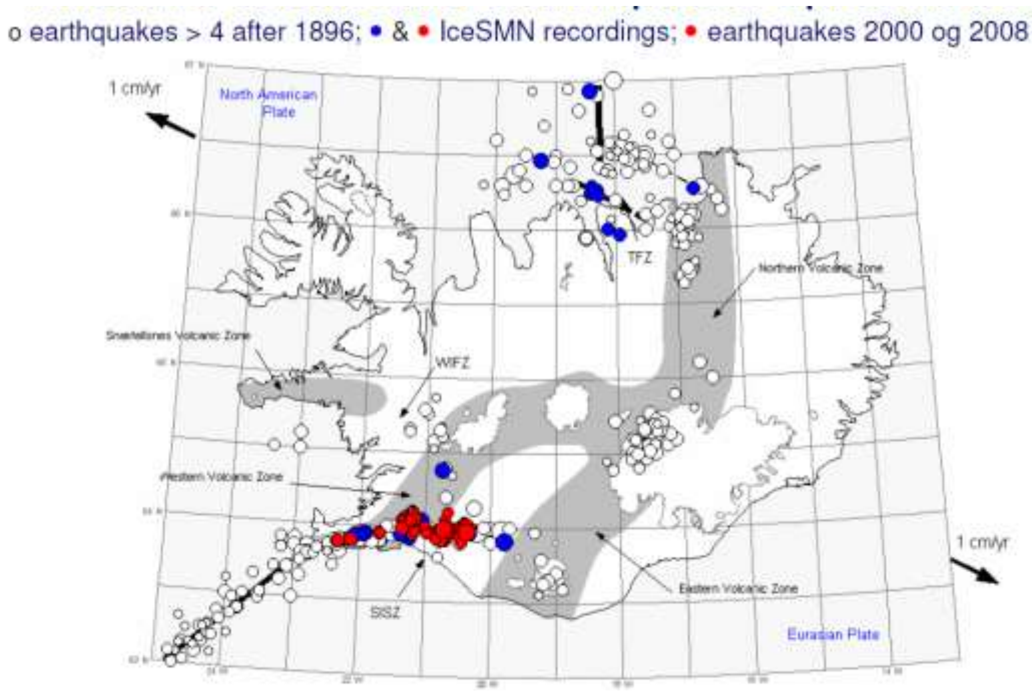

Figure 2: Volcanic areas and earthquake epicenters in Iceland 1896-2008.

Three large earthquakes have struck fairly recently in Iceland causing some structural damages. Two struck in June 2000 around the village Hella in southern Iceland, with moment magnitudes 6.5 and 6.4 and the most recent in 2008 in the Ölfus districts south of Reykjavík, having moment magnitude 6.3 (Sigurðsson, et al., 2013), see figure 1.

Measurements of strong earthquakes in Iceland have shown high acceleration near source, but for rather short duration compared to large earthquakes in Europe, the Middle East and America. Attenuation is also rapid. The high acceleration may be contributed to shallow earthquakes and short duration with rather short fault lengths. The higher attenuation may be contributed to young cracked lava bedrock that dissipates the earthquake energy (Sólnes, et al., 2013). Because of the short duration of the earthquakes, damage to buildings may be less than expected for the high near source acceleration.

Figure 3 shows a normalized linear elastic earthquake acceleration spectra derived from ground components recorded in the May 2008 earthquake in the basement of a three-storey office building in the town of Selfoss, about 5 km from the epicentre. The seismic coefficients shown are for transverse (north–south) ground acceleration (thin blue line), longitudinal (east–west) acceleration (red line) and vertical ground acceleration (thin green line). The Eurocode 8 type 1 elastic acceleration response spectra for ground type A (rock) are shown for comparison by a thick blue line representing lateral action and a thick green line for vertical action. The critical damping ratio is 5% in all cases (Sigbjörnsson, et al., 2009).

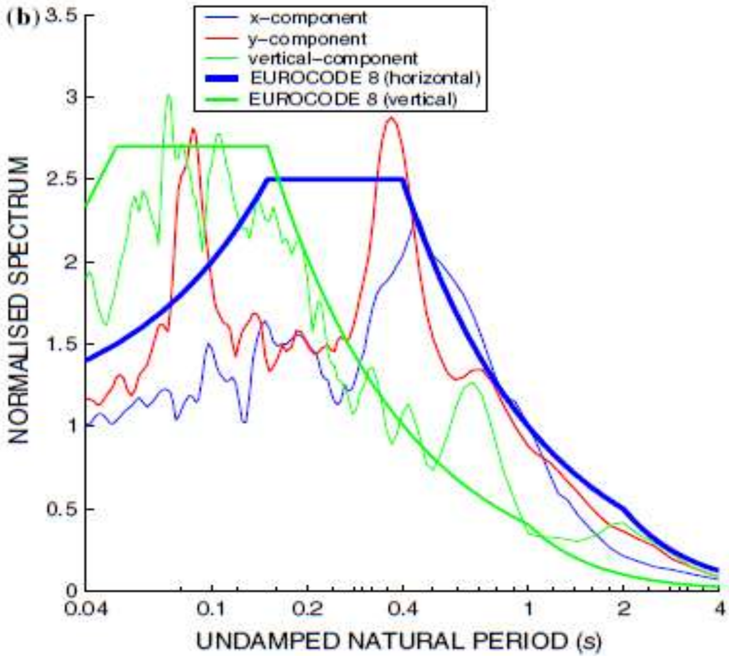

Figure 3: Normalized linear elastic earthquake spectra.

It is interesting to note that the peaks in the May 2008 horizontal earthquake spectra for periods 0,1 s and 0,4-0,5 s. The longer period peaks are probably induced by softer sediment layers under the lava rock on the surface. Pulses in the period range 1 s to 2 s are known near fault effects (SEAOC, 1999) and may be noticed in the earthquake spectra for 0,7 s and 2 s periods. The peaks are typical for the earthquakes in South Iceland Lowland. Overall, the May 2008 vertical earthquake spectrum compares reasonably well with the Eurocode 8 spectrum except for the higher periods. Note that this earthquake produced near fault effects for structures in the towns of Selfoss and Hveragerði.

CASE STUDIES

Residential buildings in Iceland are commonly low- to medium-rise reinforced shear wall buildings, cast in-situ and founded on a gravel cushion or directly on a stiff, rock-type layer. However, the rock layer at the surface may be resting on soft sediment layers generated in between volcanic eruptions producing lava flow. This building stock resisted the recent earthquakes in southern Iceland surprisingly well, often without visible damage. Figure 4 shows a typical 4-storey multifamily apartments house in Selfoss that was damaged by the M_w 6.3 earthquake in May 2008. The building is a typical low-rise concrete wall building. However, the size and setting of the windows, creates a series of narrow pier column-spandrel beam systems and the basement windows result in a "short column" phenomenon. In Figure 4, a cracked pier in between two windows is shown. More cracks were found in the basement walls inside the building. The cracks were inclined shear cracks which are repairable. However, the capacity of the basement walls may be permanently impaired and some strengthening of the structural system for future earthquake action is advisable.

The study by Sigurdsson et al. indicates that founding a building on gravel cushion resulted in a more flexible system than if a structure was founded on rigid ground. The increased flexibility induces a lower base shear demand and higher displacement demands in a basic response spectrum design approach (Sigurðsson, et al., 2013). Non-linear analysis of a typical low-rise wall on 1 to 3 meters thick gravel cushions showed that, 25% less reinforcement was needed for a wall founded on stiff ground and allowed to rock versus a wall considered fixed to a stiff base and 50% less reinforcement was needed for a wall founded on gravel cushion versus a wall fixed to a stiff base (Thorhallsson & Ólafsson, 2010).

Figure 4: Low-rise apartment house in Selfoss damaged after the earthquake in May 2008.

To investigate this further, two low-rise in-situ cast reinforced concrete buildings are studied. The foundations are also cast in-situ reinforced concrete walls resting on 2 m thick, well-compacted gravel, resting on rock. The floors including the roof are cast in-situ reinforced concrete slabs. The building has a 20 cm thick cast in place reinforced concrete walls on perimeter, in addition to light internal partition walls, dividing each floor level into apartments and rooms. The building site is set in a high seismic risk area in Selfoss on south Iceland Lowland where near fault effects can be expected. Modal response spectrum analysis was chosen as the tool to analyse the structures using a linear 3D modal spectrum analysis within the framework of a finite element program. The buildings are designed for seismic action according to Eurocode 8, US codes International Building Code (ICC,2009) and ACI 318-08 (ACI,2008) and the New Zealand standard (NZS, 2004). The design peak ground acceleration PGA is taken as 0.5g, in accordance with the EN1998-National annex for Iceland.

Case no. 1 is a three-storey apartment house, where each floor is 13 m by 6.95 m or 90 m^2. The height of each storey is 2.81 m. The wall-to-floor surface ratio is 8%. The natural period of the structure is T_1 = 0,14 s in the x-direction and T_2 = 0,055 s in the y-direction. 3D model and ground plan for the building in case study 1 are shown in figure 5. As can be seen the shear walls in the y-direction are more or less without windows, which in addition to the aspect ratio of the plan, explains the difference between the first two modal periods of the building.

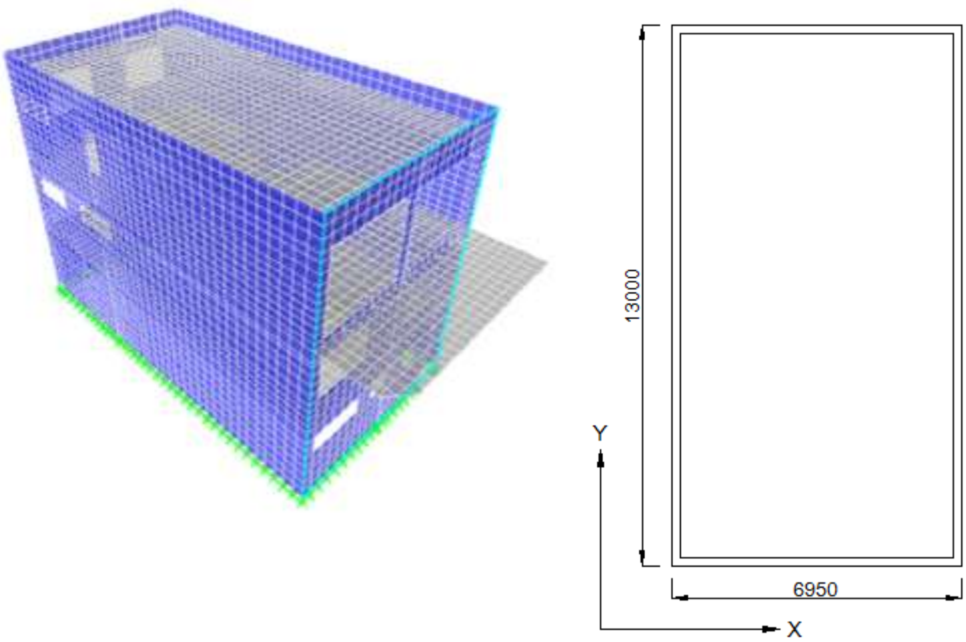

Figure 5: Case study 1. 3D view to left and ground plan of reinforced concrete walls to right.

Case no. 2 is a four storey multi-family apartment building with one storey basement. The area of the ground floor is 230 m^2. The basement height is 2.4 m. The height of the upper storeys is 2.7 m for all floors. The total height of the house is then 13.2 m. The wall-to-floor surface ratio is 9.6%. The first natural period of the structure is T = 0,13 s in both x- and y-directions. The 3D model and the ground floor plan for the building in case no. 2 is shown in figures 6 and 7.

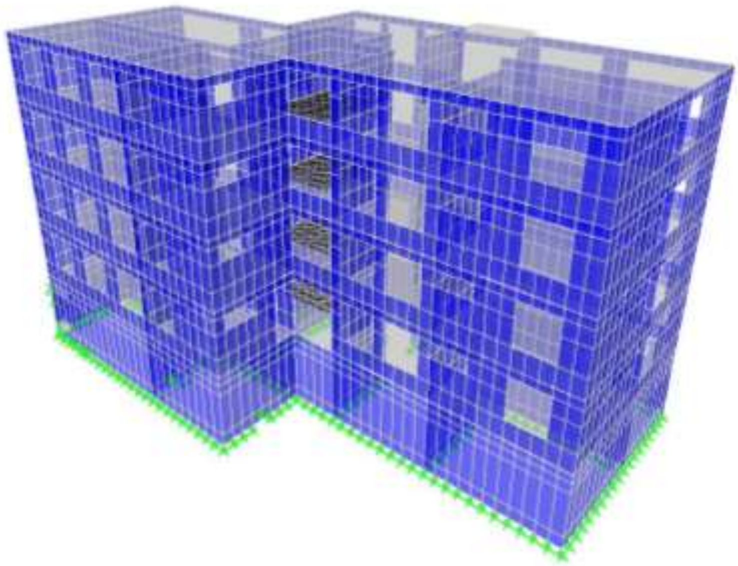

Figure 6: Case no. 2 - The 3D model.

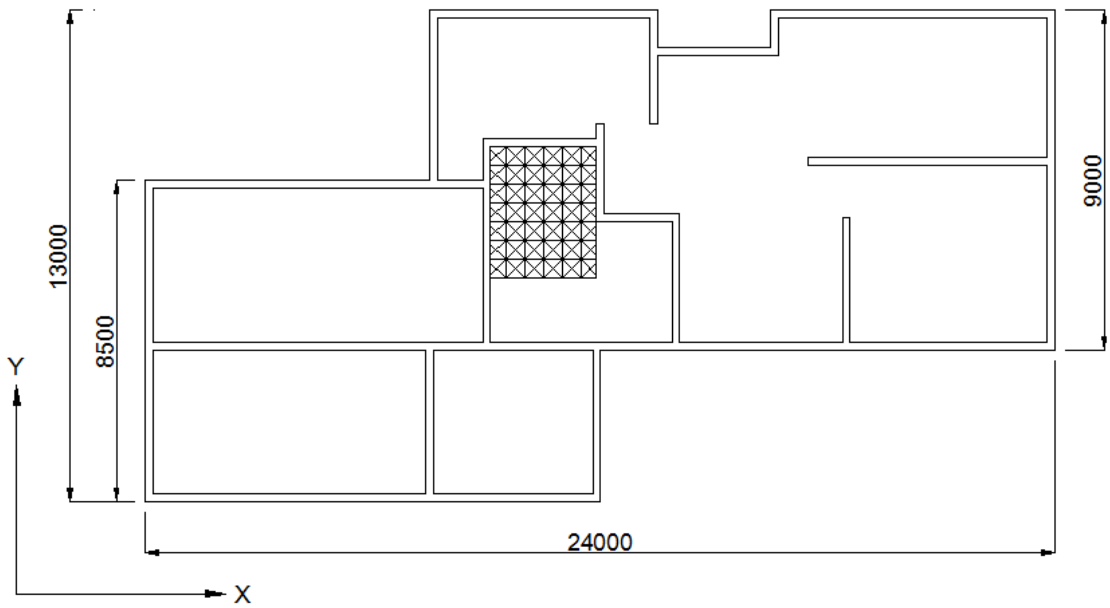

Figure 7: Case no. 2 - The ground plan showing the layout of the wall system.

Both of the two cases studied represent a fairly typical layout of Icelandic concrete reinforced shear wall buildings. As stated above the wall-to-floor surface ratio is 8% and 9.6%. Shear wall buildings with wall-to-floor surface ratio over 3-4% have shown a good performance in earthquakes (Pentangelo et al. 2010) According to this statement both of the cases studied should show good structural behaviour under seismic action.

STRUCTURAL DESIGN

In this study, the reinforcement design of concrete walls according to three different standards is compered both case 1 and case 2 were structural designed by using the FEM program ETABS. An example of typical results from shear stress evaluation is shown in figure 8, for the longitudinal wall W3 (24 meters long). The ground plan for the building is shown in figure 7.

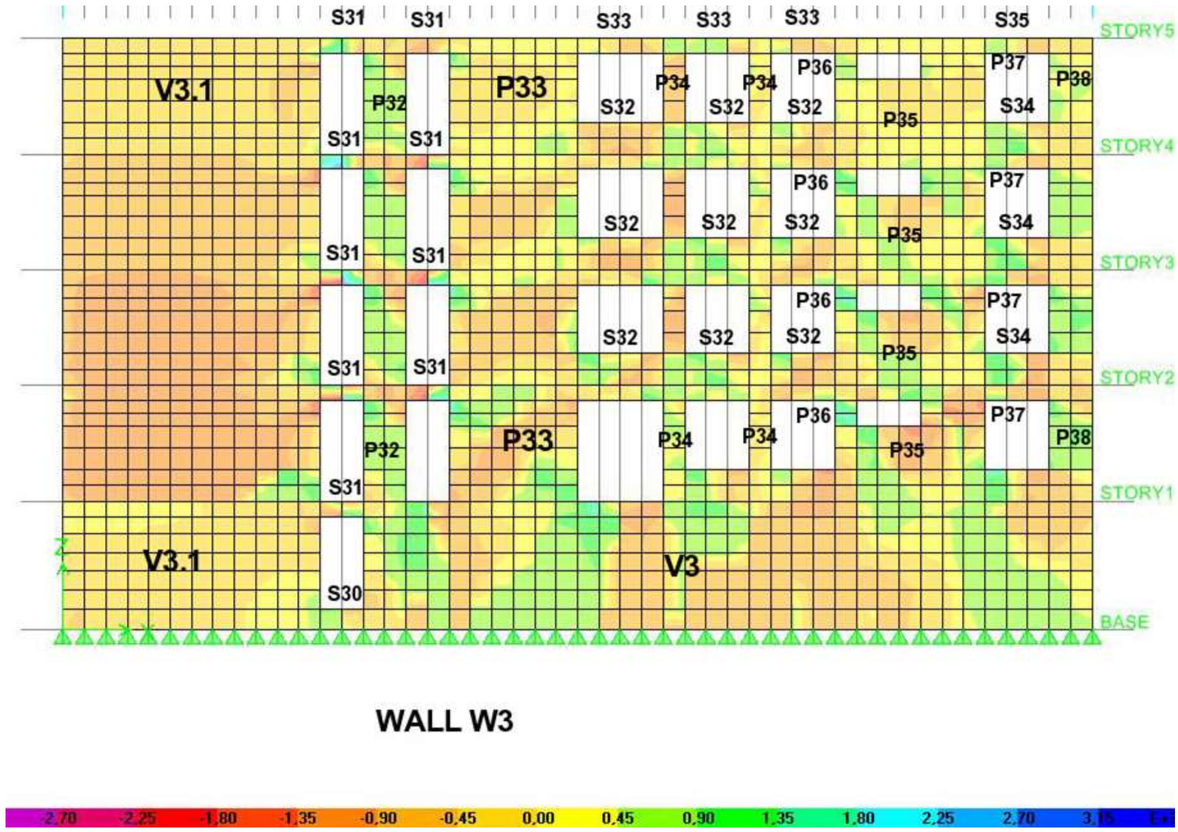

Figure 8. Case 2 - Wall W3, Elevation view. Piers and spandrels labels. Shear stress in MPa.

The design guidelines tested and the appropriate design assumptions are described in the following paragraphs.

<u>Eurocode 8, DCM:</u> In Eurocode 8 (CEN, 2003) concrete buildings are classified in two ductility classes, depending on their hysteretic dissipation capacity, ductility class medium (DCM) and ductility class high (DCH), with each class having different values of the behaviour factor q. Both cases studied are classified DCM. For the building in Case no. 1 the behaviour factor was selected as q = 1,5 for action in both lateral directions. For the building in Case no. 2 the behaviour factor was selected

q = 1,5 for design in the x-direction but q = 2,0 in for the y-direction.

<u>Eurocode 8, LLRW:</u> Large, lightly reinforced walls are a special design class for low-rise buildings, which is subset within ductility class DCM, but with less demanding detailing requirements, less minimum shear reinforcement requirements and less vertical reinforcement. The behaviour factor was selected as q = 2 for case no 1 and q = 3 for Case no 2. Behaviour factor q = 3 may be used in design if there are at least 2 walls in resisting the action in the direction of interest, but if there is just one wall then q = 2 should be used. To qualify for the design provision for LLRW a building should have the following:

- A fundamental period in each horizontal direction shorter than 0,5 s, under presumed fixity at the base of all vertical elements against rotation.

- Primary seismic walls in each horizontal direction qualifying as large walls by:
    - Having a length, l_w, of at least 4m, or 2/3 of the total height of buildings shorter than 6 m,
    - resisting at least 65% of the seismic base shear in the horizontal direction of interest,
    - supporting at least 20% of the total gravity load (Fardis, 2009).
- At least two primary walls fulfilling the conditions above should resist the seismic action in each lateral direction.

The LLRW option requires less demanding dimensioning and detailing which implies lower cost. If a structural system does not qualify as large lightly reinforced walls then all its walls must be designed and detailed as DCM ductile walls. For large lightly reinforced walls, the vertical reinforcement steel is tailored to provide capacity for demands due to moment and normal forces.

NZS 1170.5: In the New Zealand Standard 1170.5 (NZS, 2004) buildings may be designed using different ductility classes with structural ductility factor, μ, in the range of 1,25 to 6,0. For low-rise structures of limited ductility (SLD) a structural ductility factor between 1,25 and 3,0 is normally used (Paulay & Priestly, 1992). This requires less complexity in the structural analysis and less demanding detailing requirements than the higher ductility classes, simplifying both the design and construction process. The SLD ductility class allows the use of a single curtain layer of reinforcement in walls.

ASCE/SEI 7: In the US standard ASCE/SEI 7 (ASCE, 2005) the building must be designed in a very prescriptive way using a specific Seismic Design Category, SDC. The selection of SDC is based on the following paramenters:

- Response acceleration at a short period, S_{DS} = 0,2 s
- Response at the one second period S_{D1} = 1,0 s.
- The Occupancy Category OC
- The horizontal Peak Ground Acceleration PGA.
- Possibility of near fault effects with $S_1 \geq 0,75$.

The buildings in this study are in Occupancy Category OC = II and the site has a PGA = 0,5 g and near fault effects with $S_1 \geq 0,75$. Therefore they both should be designed for a Seismic Design Category, SDC = E. SDC E requires that the structure must be regular in plan. The buildings must be designed using special reinforced shear walls according to ACI 318 (ACI, 2008) with relatively demanding detailing requirements and a response modification factor R = 5,0. The R factor is very high compared to the other standards, i.e. EC8 and NZ117.5 and no reduction of the R factor is offered based on the response periods or aspect ratios of walls as do the other standards. ASCE 7 provides a redundancy factor which was selected ρ = 1,3. The standard allows use of one curtain of reinforcement in walls, if the shear stress in the wall is less than 0,85 MPa for concrete strength f_c= 25 MPa.

The amount of reinforcing steel in walls required by the three design codes was evaluated for the two Cases studied. The results are shown in tables 1 and 2 and discussed in the following.

CASE no. 1

The EC8 large lightly reinforced walls provide the minimum steel area with two curtains of rebars S10 c400 in walls. The reinforcing steel for ACI 318 Special Structural Walls is the minimum steel area with two curtains of rebars S10 c310 in walls. With a response modification factor R = 5,0 and redundancy factor ρ = 1,3 the shear stress is much lower than for EC8 DCM walls (this corresponds to EC8 q = 5/1,3 = 3,85 approximately). For NZS 3101 Structural walls of structures of limited ductility, the reinforcing steel is the minimum steel area with two curtains of rebars S10 c310 in walls. With inelastic spectrum scaling factor k_μ = 1,4 and structural performance factor S_p = 0,7, the shear stress is similar to EC8 DCM walls (this corresponds to EC8 q = 1,4/0,7 = 2,0 approximately).

Table 1: Comparison of combined horizontal and vertical reinforcement, Case study 1.

EC8 DCM walls	EC8 Large lightly reinforced walls	ACI318 Special structural walls	NZS3101 Structural walls of structures of limited ductility
mm^2/m^2	mm^2/m^2	mm^2/m^2	mm^2/m^2
835	792	1000	1048

CASE no. 2:

The reinforcing steel for ACI 318 Special Structural Walls is the minimum steel area with double curtain reinforcement of S10 c350 in walls. With response modification factor R = 5,0 and redundancy factor ρ = 1,3, the shear stress is much lower than for EC8 DCM walls.

For NZS 3101 Structural walls of structures of limited ductility, the reinforcing steel area is similar to EC8 DCM walls. With inelastic spectrum scaling factor k_μ = 1,38 and structural performance factor S_p = 0,7, then the shear stress is similar to EC8 DCM walls (this corresponds to EC8 q = 1,38/0,7 = 1,97 approximately). However sliding shear strength in construction joints governs the design of vertical reinforcement in nearly all cases. The reason is that the earthquake vertical acceleration response spectrum is not reduced with ductility factor, μ = 1,0. Structural walls of structures of limited ductility may have one curtain of rebars in walls.

As expected, the EC8 large lightly reinforced walls require less reinforcing steel area than the EC8 DCM walls.

Table 2: Comparison of combined horizontal and vertical reinforcement, Case study 2.

EC8 DCM walls	EC8 Large lightly reinforced walls	ACI318 Special structural walls	NZS3101 Structural walls of structures of limited ductility
mm^2/m^2	mm^2/m^2	mm^2/m^2	mm^2/m^2
1335	1063	907	1187

DISCUSSION

In this study two typical low-rise in-situ cast RC wall buildings have been designed according to three prominent seismic design standards: Eurocode8, ASCE 7 and NZ 1170.5.

For Case no. 1 the minimum reinforcement requirement governs the design of the walls in Eurocode 8 DCM and New Zealand Standard SLD, but sliding shear in construction joints governs the design of vertical reinforcement in many construction joints, because of the combined horizontal and vertical acceleration action applied in the analysis and design. The minimum reinforcement requirement governs the design of the walls using the LLRW option in EC8, also for vertical reinforcement. The minimum reinforcement requirement similarly governs the design of the walls according to ASCE7 US Standard and vertical reinforcement in all construction joints, which can mainly be contributed to the high response modification factor R used by the standard. The required amount of reinforcement in walls is similar using EC8-DCM and EC8-LLRW when the minimum reinforcement requirement governs the design.

In Case no. 2 the minimum reinforcement requirement governs the design of the walls in the basement, the 1st storey and the 2nd storey using EC8-DCM, EC8-LLRW and NZ1170.5-SLD, but sliding shear in construction joints governs the design of vertical reinforcement in most construction joints, because of the combined horizontal and vertical acceleration action applied in the analysis and design. The minimum reinforcement requirement governs the design of the walls in most cases in the basement, the 1st storey and 2nd storey using ASCE7 and also the vertical reinforcement in nearly all construction joints, which can mainly be contributed to the high response modification factor R used by the standard. The amount of reinforcement in walls is similar in EC8-LLRW and NZ1170.5-SLD, but EC8-DCM gives higher values as show in table 2.

It can be concluded from the cases studied that it is important to use horizontal and vertical acceleration response spectrum, when designing in high seismic risk areas according to the EC8 and NZ1170.5 and even the ASCE standard. This is because sliding shear will in many cases govern the design of vertical reinforcement in concrete joints in walls and the vertical acceleration will also generally increase the demand for vertical reinforcement in walls.

The buildings are both well within the allowable limits of drift set by the standards, with about 6% - 8% of allowable drift limits depending on the code applied, as may be expected for low-rise wall buildings.

The future development of guidelines for design of earthquake resistant structures will aim towards performance and displacement based design methods with the emphasis on examining strain and displacement. Future codes should include guidance regarding the inclusion of structure-soil interaction (SSI) in the design process. The European and New Zealand standards may in the future use ground acceleration contour maps anchored to the short and long period range of the design response spectra to give a more uniform hazard. Another option is to have a separate response spectra for local and distant events. The standards should also include more information on design for near fault effects. Preferably a map for displacement based design response spectra should be included as well as a rocking spectra for lift and tilting of foundations.

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