Earthquake Design Practice of Traditional Norwegian Buildings According to Eurocode 8

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SUMMARY:
The Eurocode 8 design requirement was launched in Norway early this millennium. Seismic actions are since 2010 completely covered by the Eurocode 8, including relevant national values. Traditionally, dominating horizontal forces originate from wind induced loads. This design requirement has over the years shaped the common practice in design of traditional Norwegian building and foundation solutions, rather than seismic actions which may render larger design forces. Special attention is paid to earthquake design based on the new design regulations in Norway. The consequences of the new design code are exemplified with three buildings from regular Norwegian practice. The influence of soil-structure interaction into response is discussed and exemplified by typical soft clay amplification, common in the Oslo area, for recorded and artificial time history and response spectra, with the importance of having accelerograms that represent the Norwegian hazard levels. Earthquake design influences on best practice with new considerations are also drafted.

Keywords: Low seismic area, Response spectra EC 8, Soil-structure interaction, accelerograms EC 8

1. INTRODUCTION

Continental Norway is situated in a low seismicity region without any late significant earthquake history recordings of severe damages to buildings and structures. The first considerations related to a seismic structural design goes back to the sixties and are linked to an initial feasibility study and planning of a nuclear power plant construction in the Oslo region. Studies related to this undertaking revealed noteworthy historical seismic activities in this region, even though records of only small earthquakes surfaced.

One of the earliest historical chronicle from a significant earthquake in the region dates back to the year 1647. The epicenter of this earthquake has been assessed to the outer Oslofjord and the magnitude estimated close to 5 on the Richter scale. Earthquakes are reported from time to time during the following centuries but apparently no seismically significant event is mentioned until 1904. Then a moderate to strong sized earthquake struck the Oslo region and was widely felt all along the Oslofjord and caused damage to several buildings. The epicenter was located near the mouth of the Oslofjord about 100 km south of city of Oslo. The magnitude has been estimated to MS = 5.4, see Molina and Lindholm (2005). Based on seismo-tectonic considerations, Bungum et al. (2005) have assessed that the zone can potentially produce a magnitude 6+ earthquake.

A seismic risk scenario was conducted in Molina & Lindholm (2005), which was based on the capacity-spectrum method applying two earthquake sources, one very close to the city and one near the 1904 epicenter. Both scenarios exhibit strong dependencies on the soft clays underlying large parts of Oslo, where the deep clay deposits under the city contributed to the damages. The risk scenario for the city of Oslo indicates that as much as 45% of the building mass will be damaged if a magnitude 6 earthquake occurs on the eastern rift boundary fault near Oslo. These numbers do not include secondary damages such as associated fires, liquefaction etc.

2. CONSEQUENCES FOR TRADITIONAL NORWEGIAN BUILDINGS; CASE STUDY

To exemplify the effects of the newly introduced seismic design actions to the common practice in Norway, three buildings are investigated. All three buildings are constructed with a vertical bracing system designed and distributed according to the transvers forces acting in the floor diaphragm from wind and vertical misalignments; a building configuration not uncommon in its design to the Norwegian design practice. However, all three buildings clearly deviate more or less from good seismic action design following basic principles such as robustness, continuity, simplicity and symmetry. The designing features for two of the three cases presented below are still expected to be wind and vertical misalignment assuming the foundation is on bedrock, the same cannot be said for the general case with other forms of soil conditions. For the last case the seismic actions clearly will be the dominating designing features and illustrates the need for early introduction of seismic design consideration into the building process.

To evaluate the seismic actions and illustrate the new challenges for design of structures as well as possible shortcomings of existing structures both response spectra and acceleration time histories are used to estimate seismic reactions. For comparison are all seismic actions, response spectra and accelerograms component scaled according to the site peak ground acceleration (PGA). It is then assumed a design situation employing the, for Norway, new design code Eurocode 8 with PGA value corresponding to a return period of 475 years assuming a seismic importance factor of II. It is chosen to introduce three natural recorded time histories as well as one artificial time history simulation. The artificial follows from the response spectra on bedrock. It is here used the guidelines of the Eurocode 8 when there are no other national guidelines regarding simulations, such as the use of a stationary part of 10s, with approximately the energy levels of the Norwegian elastic response spectra. This will create a variety which all designing engineers must face in their practice when dealing with the new seismic actions.

2.1. Traditional Norwegian Buildings, Oslo

The building stock of the Oslo area can shortly be summarized as consisting of a predominance of wood and concrete structures with an older agglomeration of masonry buildings and modern steel structures. There are a large number of light wooden frame structures, between 1-2 stories, mainly in the outskirts of the Oslo area. More modern structures are constructed with concrete shear walls, between 4-7 stories or steel braced frame structures, then either as low-rise buildings with 1-3 stories or as mid-rise buildings with 4-7 stories. The basis for new design is quite commonly steel frame structures and concrete shear walls, mainly between 4-7 stories. Historically unreinforced masonry bearing walls structures were quite common both as low-rise buildings, 1-2 stories and as mid-rise buildings with 3-4 stories. Due to several reasons, the latter building type, even before the introduction of seismic actions, is less common today.

2.1.1. Building I and II

These buildings are initially the same building but with different bracing systems rendering significant differences in the structural behaviour when exposed to seismic actions, see Fig. 1. This type of building and its configuration is quite common in Norway and normally consists of one or two stair cases constructed as reinforced concrete shafts. Complementary bracing is then often constructed by either concrete shear walls or as in this case with steel frame bracing, which then may introduce a structural stiffness variation in bracing components, enhancing the loss of symmetry for the structural system. The hollow core slabs, which are precast slabs of pre-stressed concrete, are commonly carried just by simply supported beams, not necessarily designed with continuity. Finally, these structures are frequently constructed with a foundation system consisting of one or several basement floors including parking. A deep foundation solution on sites with layers of soft soils to the bedrock or stronger layer of subsoil below are often based on different types of impact driven pile solutions.

2.1.2. Building III

This is a low rise structure which covers a large extending floor area frequently used as school structures or as assembly halls, shopping malls or other cultural institutions, see Fig. 2. The common practices are to base these structures on hollow core pre-stressed slabs and simply supported beams while the columns continue through the height of the building. The bracing system is here based on three concrete shear walls in the longitudinal direction, which may constitute parts of the staircases in the building. This is then complemented with steel frame bracings in the transversal direction. For this type of structure with large extensions and heavy floors is the bracing system symmetry even more important to consider due to seismic actions. Otherwise, the structure may suffer considerably under great stiffness variations between concrete shafts and the much softer steel bracings. However, and perhaps unfortunately, this is commonly in Norway a process more or less fully controlled by the architect with just minor influence of the designing engineer. The foundation of this type of structure is often a shallow type of foundation. This is normally constructed as a slab-on-grade foundation where the weight of the building is transferred to the soil through a concrete slab placed directly on the ground surface.

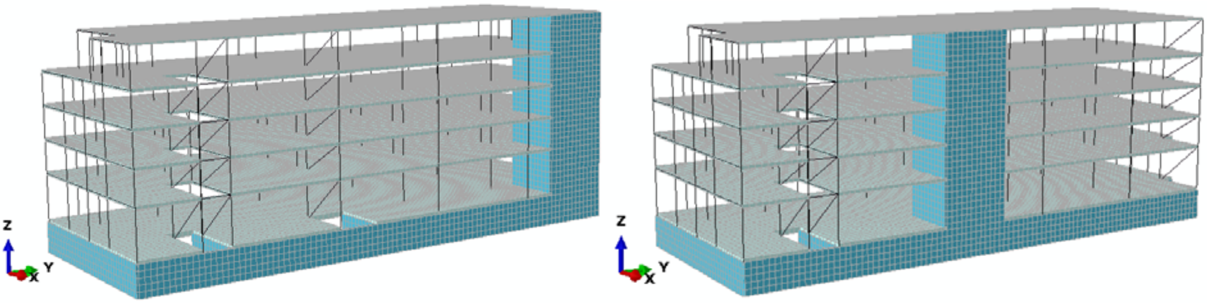

Figure 1. Building I and II, a conventional building system in Norway; left, building I and right, building II.

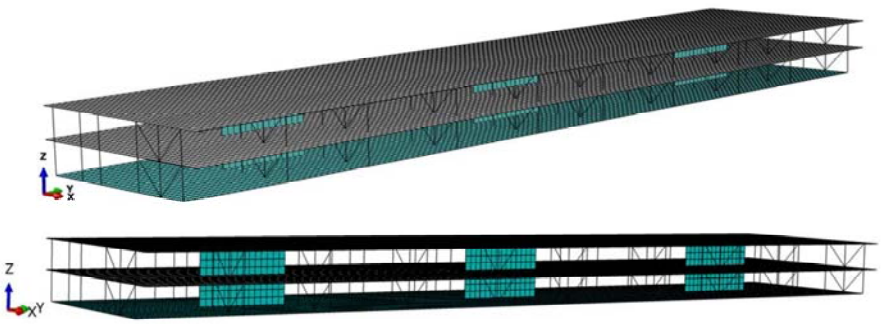

Figure 2. Building III A conventional building system in Norway; common school or other cultural institution building.

Building I will clearly have a larger twisting motion due to the great asymmetry, especially from loads in the x-direction. Large horizontal interstory displacements may also occur for the columns located opposite to the concrete shaft. This is illustrated by the first three modes which all are combined modes with respect to the basic degrees of freedom, see Fig. 3. The first mode is as expected a displacement and rotational mode with its centre of rotation close to the eccentric concrete shaft. The second mode is actually a pure displacement mode. However, the displacement pattern is diagonal across the structure in a x- and y-direction as shown in Fig 3. Finally the third mode is a torsion mode with its centre of rotation close to the mid-section of the building. The corresponding natural frequencies for mode 1 to 3 are 1.45 Hz, 2.16 Hz and 3.45 Hz, respectively. Thus, columns designed to carry only vertical loads must be controlled for horizontal displacement such that they do not lose their vertical capacity due to buckling or P-Δ effects. This building configuration clearly deviates from good design practice following the concepts of simplicity, symmetry, continuity and robustness.

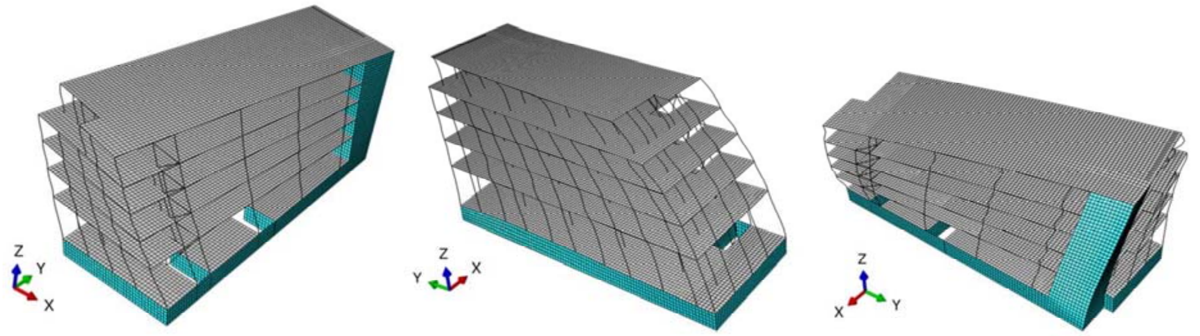

Figure 3. Building I; 1st Mode 1.45Hz (left), 2nd Mode 2.16Hz (mid), 3rd Mode 3.45Hz (right)

Building II improves on these conditions by a more centrically placed concrete shaft. This is also a common placement in that it is still kept to the outer wall of the building. Thus, it is still asymmetric in the x-direction leading to coupled motions around a rotation point close to the shaft. This can clearly be seen in the natural modes of the structure as displayed in Fig 4. Here the first mode is a rotational mode, again with its rotational centre at the concrete shaft, now situated in the y-direction. The second mode is a displacement mode which now clearly is just along the x-direction with a minimum of coupling motion. The third mode is again a rotational mode but this time the rotation centre is situated on the opposite side of the centre to mode 1, rendering the somewhat special coupling properties as the first. The corresponding natural frequencies for building II shows similar values as for building I, now with natural frequency 1 to 3 given as 1.44 Hz, 2.17 Hz and 2.82 Hz, respectively.

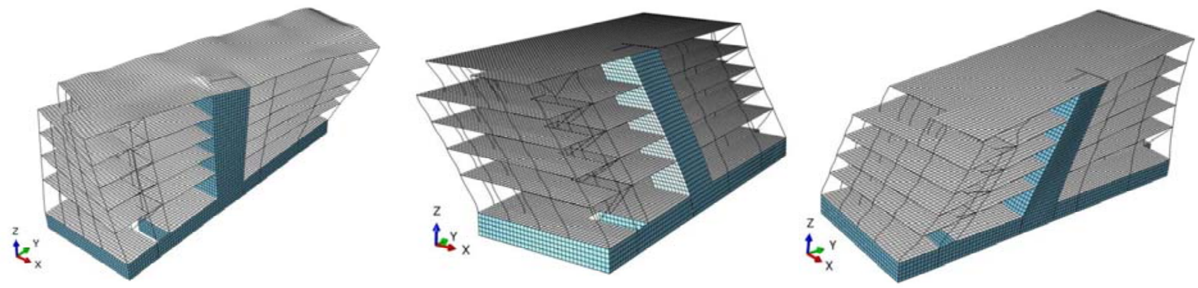

Figure 4. Building II; 1st Mode 1.44Hz (left), 2nd Mode 2.17Hz (mid), 3rd Mode 2.82Hz (right)

Finally, for building III where the geometric symmetry in any direction is not an issue, as was the case for the previous two configurations. However, the material and member dimensional differences in the two directions may constitute a potential problem. The bracing system is here chosen to cover some of the greater forces from the deck masses. Normally it would be expected that the shear walls also could be were facing the opposite direction to control the wind forces, whereas in the present layout they will be facing misalignment forces, in a common design sense. For this building it will clearly be great differences between seismic actions and loads from wind and misalignment. Of course, the final influence of earthquake actions on the vertical bracing system will be dependent on the severity of the seismic forces, again due to site specific parameters such as peak ground accelerations (PGA) and soil conditions, which will be demonstrated.

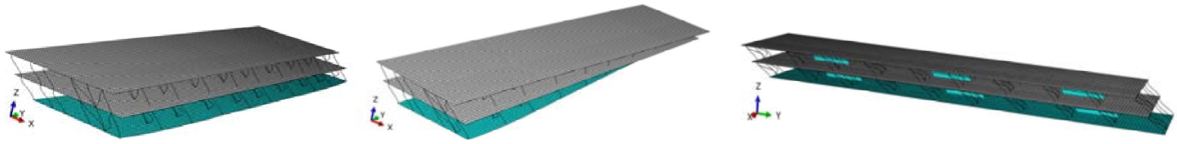

Figure 5. Building III; 1st Mode 3.15Hz (left), 2nd Mode 3.19Hz (mid), 3rd Mode 4.56Hz (right)

2.2. Seismic Actions According to Eurocode 8

Basis parameters for seismic actions according to Eurocode 8 including national box values; PGA = 0.55 m/s^2, ground type A and D, importance class II thus importance factor γI = 1.0, 5% structural damping, low ductility class thus behaviour factor q = 1.5. Importance class II is used for ordinary buildings, while Importance class III is defined for buildings where seismic resistance is of importance regarding collapse such as schools. In accordance with table Eurocode 8 NA.4(902) schools may under certain circumstances be in class II, even though commonly in class III. Thus, building III, which is defined as a school building, will for simplicity and comparison together with the other two buildings be assumed as class II, importance factor γI = 1.0.

Ductility classes available in Norway are in accordance with the national annex restrained up to the medium ductility class (i.e. DCL and DCM), i.e. cannot use behaviour factors, q, other than specified for the ductility class DCM, Eurocode 8 NA.6.1.2. In Norway it is not historically common to employ ductility properties of structural components and connections and thus, so far, the most used ductility class of practicing engineers is DCL. Therefore, to address the situation in Norway, these evaluations are restricted to the same behaviour option rendering q = 1.5.

The Oslo area is used as site. Variation in depth of the clay deposits is expected from foundation on bedrock to foundation on top of several tens of meters of clay. This is because large portions of the Oslo area were submerged during the last glaciation and the later sea level change from the land uplift left areas with deep clay deposits. Clay deposits in Oslo will fall in category of soft soils with a maximum average shear wave velocity of 180 m/s (ground type D) to stiff soils, then 180 – 360 m/s (ground type C) as reported in Molina & Lindholm (2005). In the present investigation it is chosen to use bedrock foundation (ground type A), and a common soft clay deposit, here set to ground type D. Incorporating the change of softer soil structure response in the present investigation is done by transfer of the bedrock accelerations through a site specified soil structure before exposed to the structure. This response analysis is used to account for possible amplifications of ground accelerations through a, for Oslo, typical layer of approximately 40m of clay above bedrock. A one-dimensional wave propagation problem is solved by an iterative solution through a horizontally layered visco-elastic soil profile using secant soil stiffness.

2.3. Numerical Evaluation of Structural Response to the Seismic Actions

An alternative approach to safe earthquake design of the three building in Figs. 3to 5 compared to the response spectra analysis (RS) is to employ the use of accelerometric time series. Three recommended recorded time series from Friuli (F), Imperial Valley (I) and Nahanni (N) have been applied; see Bungum et al (1998). Furthermore, a time series is simulated according to the elastic response spectra to bedrock following the somewhat crude guidelines of the EC8, for use when there are no other national guidelines available. This creates an acceleration time series in line with the response spectra, which is expected to give conservative results compared to the recorded time series. The artificial time series should include a stationary part of 10s, which together with rise time and decay must be seen as over-conservative when applied for Norwegian conditions. Furthermore, artificial records often exhibit larger number of cycles, greater than natural recorded time histories. Thus, they may lead to an over-conservative structural demand for elastic systems. The report by Bungum et al (1998) indicates a total duration between 5-15 seconds should be expected for Norway.

Peak base reactions in x- and y-directions are important when evaluating structural design by the lateral force method and must in general uphold symmetry. From the results in Table 1 it is clear that building configuration I has a particularly unfavourable distribution of bracing elements in the x-direction rendering large torsional response. This can easily be identified with unreasonably low values of peak base reaction forces in the x-direction as well as the reaction forces in the y-direction due to the same seismic action being of equal size. For building II this has improved significantly but still shows the coupling between the two directions, where the vector sum of the two is approximately the same. It is noted that for ground condition of bedrock, both will have wind/ misalignment as designing forces. However, for the clay conditions in Oslo seismic actions estimated by RS will be the designing forces. Furthermore, it is noted that the natural time histories renders results similar or even lower than wind action, where the artificial time series do not. This shows that the use of natural time series can be helpful in representing realistic seismic levels to be used in the design process.

For building III the results are quite different, showing the beneficial symmetry properties of the bracing system. However, this type of structure is expected to have seismic action as major design criterion for the bracing system. This is clearly shown in Table 1. A factor well over 3 for the XX-response and an even greater factor for the clay soil conditions, close to 8, can be found.

Table 1. Peak base shear in x- and y-direction for all configurations [kN]. A, bedrock- and D, clay conditions. XY = response in X-dir. due to seismic action in Y-dir., W/S-wind/misalignment, F-Friuli, I-Imperial Valley, N-Nahanni, EC8simulated, RS-response spectra.

	RS;A	RS;D	W/S	Max(F,I,N;A)	Max(F,I,N;D)	EC8;A	EC8;D
Building I, XX –dir.	588	1593	1871	722 (N)	1727 (F)	853	2070
Building I, YX –dir.	584	1636	0	527 (F)	1077 (F)	566	1135
Building I, YY –dir.	769	2197	840	852 (N)	1698 (F)	1026	2159
Building I, XY –dir.	584	1636	0	527 (F)	1077 (F)	566	1135
Building II, XX –dir.	907	2612	1871	1165 (F)	2375 (F)	1276	2456
Building II, YX –dir.	198	537	0	284 (F)	571 (F)	298	595
Building II, YY –dir.	706	1706	841	1066 (I)	2447 (I)	1076	2858
Building II, XY –dir.	198	537	0	284 (F)	571 (F)	282	595
Building III, XX –dir.	5495	12670	4032	5760 (N)	17520 (N)	7187	19810
Building III, YX –dir.	0	0	0	0	0	0	0
Building III, YY –dir.	5268	10790	634	7232 (I)	13150 (I)	9401	13800
Building III, XY –dir.	0	0	0	0	0	0	0

The results in Table 2 show the resulting forces in the bracing system with difference between the global base shear results and the local elements. It is clear that the response spectra results for clay conditions are greater than wind actions. However, if the same reactions are considered for the time series calculation then these are considerably lower for natural time series. The results in Table 2 also show from the response spectra (RS) similarities between the EC8 simulated bedrock time series and the corresponding site specific time series, indicating good agreement between the soil analysis and the soil assumptions in EC8. Thus, the simulation does not give any extra information regarding the seismic response. This indicates that it may be useful to apply time domain analysis to obtain better knowledge of the dynamic structural behaviour and thus possibly reduce the influence of seismic loads on a final design. Even if time series analysis would not render forces lower than those from wind it will assist in the documentation required by EC8 for ductility and redundancy within the structure. This can be used to establish the behaviour factor needed for low seismic forces, still well within the medium ductility class. For building II, it is obvious that the improved symmetry gives an improved force distribution both between bracing elements as well as between floors This is also displayed in the reduced difference between response spectral analysis and time series analysis. Finally, as can be seen for building III the response spectra as well as the time history analyses will of course give seismic action higher than those from wind but with a clear reduction when employing time series, which together with DCM ductility should balance the reactions forces.

Table 2. – Bracing force response in X-dir. due to seismic action in X-dir [kN]. A, bedrock- and D clay, W/S-wind/misalignment, F-Friuli, I-Imperial Valley, N-Nahanni, EC8simulated, RS-response spectra.

	RS;A	RS;D	W/S	Max(F,I,N;A)	Max(F,I,N;D)	EC8;A	EC8;D
Building I, Floor 1	132	374	315	75 (F)	276 (F)	141	378
Building I, Floor 4	60	162	150	48 (I)	129 (F)	89	200
Building II, Floor 1	55	156	152	63 (F)	126 (F)	65	125
Building II, Floor 4	36	103	87	47 (F)	92 (F)	48	83
Building III, Floor 1	225	552	133	238 (N)	724 (N)	271	767
Building III, Floor 2	101	251	34	109 (F)	297 (N)	153	377

Finally, in Table 3 the results in the stiffer parts of the bracing system are presented. These are the structural components one wishes should attract most of the reaction forces. Due to their high stiffness compared to the steel frame bracing, the distribution between the components will be well correlated to the displacements of the structure. This do not suggest that a stiffer structure is beneficial, far from it, but once they are there one want to use them to relieve other parts of the structure, such as the columns, reducing the inter-story drift which is a possible damage indicator to be kept under control. For all three buildings the total drift and the inter-story drift are minor and thus not addressed in the present investigation. The results in table 3 for the XX-direction compared to the results of the bracing forces for the first floor as given in Table 2, clearly show the distribution difference of seismic and wind reactions.

Table 3. – Base shear reaction for concrete shear walls (i.e. staircase for building I and II, outer wall for building III) in x- and y-direction [kN]. A, bedrock- and D clay conditions. XY = response in X-dir. due to seismic action in Y-dir., W/S-wind/misalignment, F-Friuli, I-Imperial Valley, N-Nahanni, EC8simulated, RS-response spectra.

	RSA	RSD	W/S	Max(F,I,N;A)	Max(F,I,N;D)	EC8;A	EC8;D
Building I, XX –dir.	33	94	322	152 (F)	222 (F)	195	257
Building I, YX –dir.	31	90	0	69 (F)	118 (F)	78	160
Building I, YY –dir.	122	348	73	216 (N)	423 (N)	329	500
Building I, XY –dir.	181	516	0	127 (F)	364 (F)	193	573
Building II, XX –dir.	747	2150	330	916 (F)	1877 (F)	922	1850
Building II, YX –dir.	174	456	10	247 (F)	597 (F)	278	644
Building II, YY –dir.	844	2096	149	1097 (I)	2806 (F)	1187	3228
Building II, XY –dir.	200	532	109	225 (F)	479 (F)	244	477
Building III, XX –dir.	1931	4521	1407	1994 (N)	6153 (N)	2494	6940
Building III, YX –dir.	55	163	309	54 (N)	239 (F)	66	244
Building III, YY –dir.	2086	3599	446	2407 (I)	4380 (I)	3063	4529
Building III, XY –dir.	158	543	0	162 (I)	320 (N)	196	384

To complement the results in Tables 1 to 3 are also the spectral densities for base shear reactions from the present investigation presented. These are all from the base shear time series collected for all four time histories and for all three buildings. The results shown are from seismic action in the X-direction with the corresponding base shear also in the X-direction for all buildings, Figs.6 and 8-9, and as well for the YY-response of building I, Fig 7. This response analysis is thus used to emphasize the frequency dependent amplifications structural response due to ground accelerations in a typical Oslo soil condition as bedrock or with clay layer thickness of approximately 40m above the bedrock.

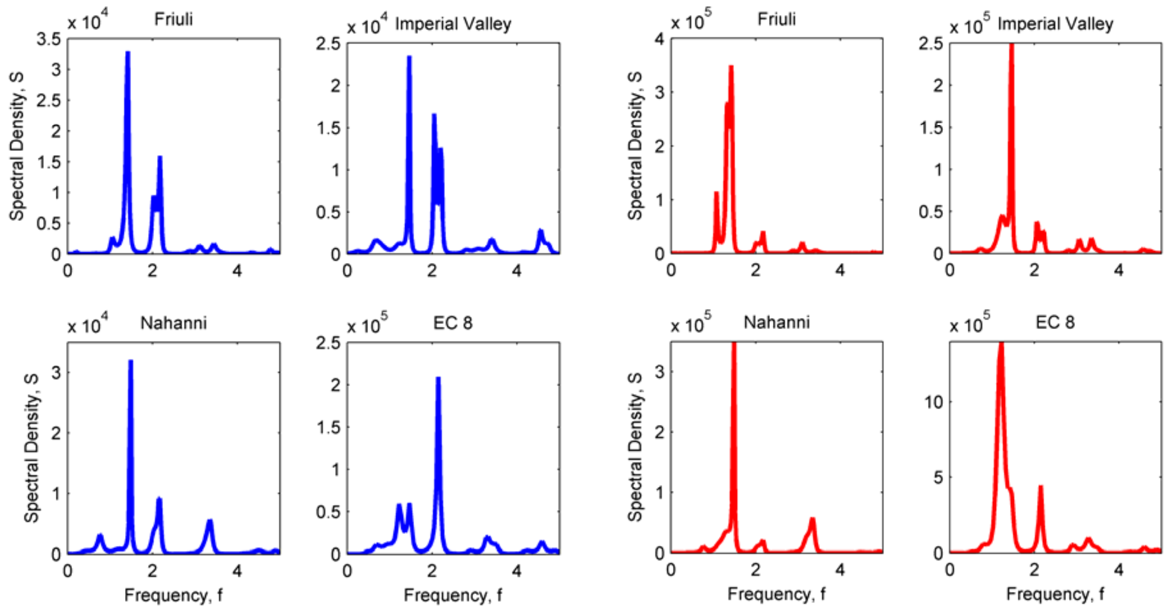

Figure 6. Spectral densities of base shear time series of building I (XX-response) from Friuli, Imperial Valley, Nahanni, EC 8; Bedrock (left) and Clay (right)

From results retrieved by spectral analysis of the time series resulting from the mentioned soil analysis the clay soil system natural frequencies were identified. The three first modes of the soil system are 1.15 Hz, 2.9 Hz and 5.19 Hz. With these results and the results previously presented for the natural frequencies and modes of the three buildings the dominating factors and modes can be identified. It is quite clear that the spatial motion of the modes can be seen. However, it is the diagonal 2nd mode of building I that shows the largest contribution to this coupling. This is especially clear for the Friuli and Imperial Valley earthquakes, but also for Nahanni and Eurocode 8, where reaction forces indicate this for bedrock conditions (left diagrams). The XX-response show large contributions for the two first time histories at this frequency in addition to the expected first mode contribution, Fig 6, while the same time series do not show the same coupling tendencies for the YY-response shown in Fig 7, bedrock conditions. Now, the same cannot directly be said for the resulting spectral densities of clay conditions. This is due to the amplification of frequencies within the soil system which have a natural frequency close to that of the 1st mode of building I, clearly activating the coupling which also is present in all modes. This becomes evident if the diagram for the Eurocode 8 simulated time series with clay conditions is scrutinized closer, Fig 6. Here the response is actually totally dominated by the response peak of the 1st frequency to the soil system. The same peak can also be seen in the Friuli clay condition response diagrams.

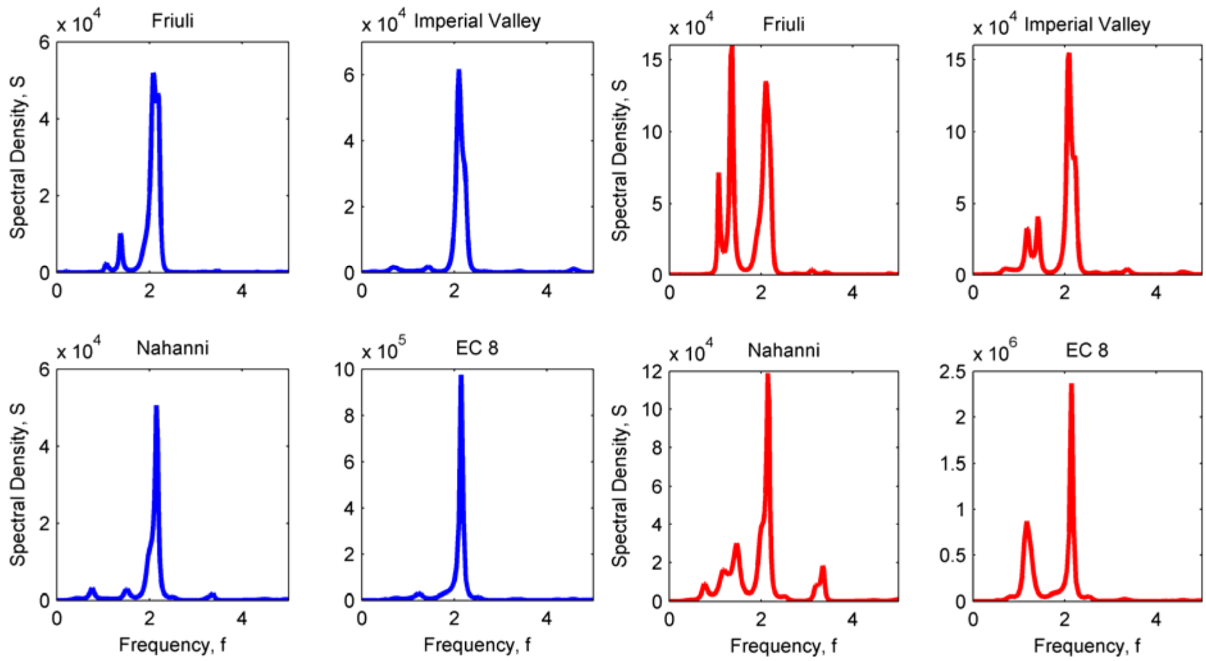

Figure 7. Spectral densities of base shear time series of building I (YY-response) from Friuli, Imperial Valley, Nahanni, EC 8; Bedrock (left) and Clay (right)

As mentioned, changing from the building I configuration of the bracing system to that of building II also changed the unfavourable asymmetric influence. This can also be seen in Fig. 8 showing the spectral densities for the XX-response of building II where the bedrock conditions are dominated by mainly one frequency. It is especially the 2nd mode that dominates all three natural recorded time series as well as for the artificial Eurocode 8 time series. This is expected since it is the pure displacement mode in the x-direction with no to minimal coupling to the x-direction. It is only some contributions in the clay soil system which shows frequency components of the soil. Since the 1st soil frequency is close to that of the structure is this frequency component also activated, thus it is the soil system that introduces the coupling between modes, which especially can be seen for the Friuli clay spectral density plot in Fig. 8.

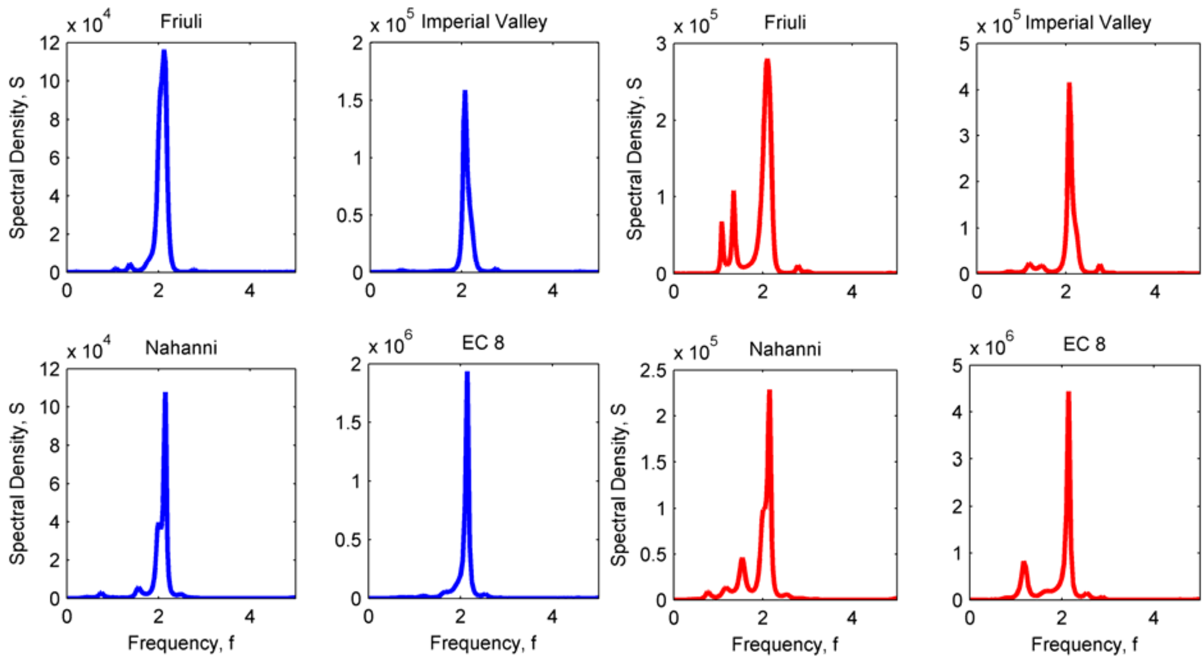

Figure 8. Spectral densities of base shear time series of building II (XX-response) from Friuli, Imperial Valley, Nahanni, EC 8; Bedrock (left) and Clay (right)

Finally, for completeness is also the XX-response for building III included in Fig. 9. This response is not coupled by the modes them self, which also is evident from the mode plots in Fig. 5. It is worth noticing that there are some additional peaks in the spectral densities which originate from the seismic motion in the bedrock figure, not to be confused with coupling of motions in the two directions or the amplification from the soil system. This comes from the stronger quasi static response which is expected in the much stiffer structure that building III represents.

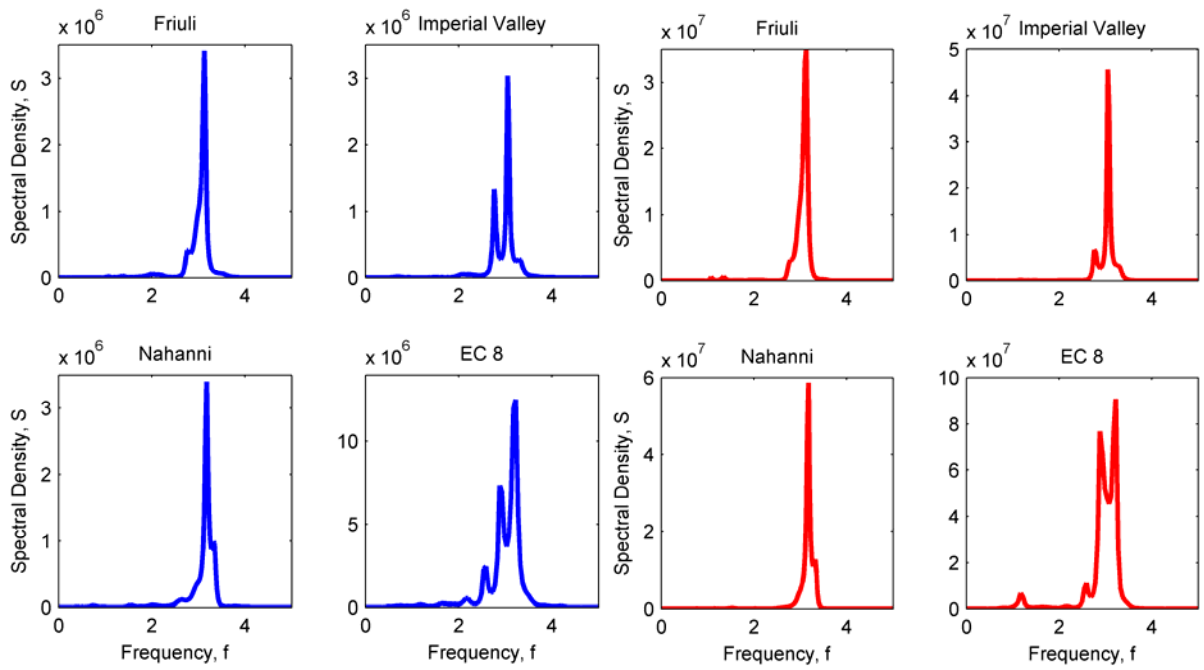

Figure 9. Spectral densities of base shear time series of building III (XX-response) from Friuli, Imperial Valley, Nahanni, EC 8; Bedrock (left) and Clay (right)

CONCLUSION

Introduction of seismic action design in Norway by the new design code should introduce new organizational requirements on the entire building process. It must move towards engineers working more during the conceptual design phase, i.e. among other considerations focus on symmetry, continuity and robustness of the structure. This clearly must also include early understanding and collaboration among consulting engineers as well as across all fields with all participants of the project, such as architect, civil engineer and building project owner. Apart from other seismic protection design by controlled behaviour of structural components and overall global performance by the engineers, earthquake protection should be incorporated within the architectural design process. It is well known that last minute changes always will be costly and will also include possible changes within the developed architecture, which is never welcomed by any of the parties involved.

The intention of the present investigation was to highlight the structural challenges and the need for small but necessary changes in the design of structures in Norway to include safe earthquake design. It is shown that some traditional bracing configurations designed for wind and misalignment loads clearly do not hold the necessary considerations needed for seismic design. By introducing conceptual design with focus on a few basic principles such as symmetry, simplicity, robustness and a better understanding of the global behaviour of the structure for large displacements ensuring ductile behaviour, safe earthquake design of buildings, when required, should not introduce much extra cost

The earthquake risk, which is the product of the hazard and the exposed agglomeration of buildings, will be a slowly increasing value as long as the number of buildings prone to be sensitive to seismic actions is limited. This may be because new designs have not in a correct way been taking into account and the need of proper seismic consideration based on the principles of Eurocode 8. This may be because of unawareness, convenience or even just simple ignorance; for similar findings see also Bachmann (2002). In Norway there are no plans to introduce any retroactive regulation for existing structures. This wouldn't be a good investment as the cost would be high compared with the cost of rebuilding the structures that might be heavily damaged, contrary to the very small cost of investments associated with safe earthquake design for new buildings.

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