Earthquake engineering design practice in Norway: Implementation of Eurocode 8

A. Rønnquist
Dep. of Structural Engineering, NTNU, R. Birkelandsvei 1a, 7491 Trondheim, Norway

S. Remseth
Dep. of Structural Engineering, NTNU, R. Birkelandsvei 1a, 7491 Trondheim, Norway

C. Lindholm
NORSAR, Gunnar Randersvei 15, 2007 Kjeller, Norway

SUMMARY:
Earthquake records of Norway show only one damaging earthquake, 1904 Oslofjord, where buildings suffered minor damage (M=5.4). Larger earthquakes have occurred in Scandinavia over the past couple of hundred years, but with long recurrence intervals and they are geographically disperse. Earthquakes exceeding M=6.0 are expected, also in this stable tectonic region. Earthquake design was introduced in Norway with new design requirements launched early this millennium, requiring control of earthquake load effects. Traditionally, Norwegian designs are based on horizontal forces from wind and misalignment loads, which have shaped the common practice in design of traditional Norwegian buildings. When Eurocode is applied , for linear elastic design, forces several times higher than wind may be reached creating challenges for new and/or retrofitting of buildings, e.g. additional floors. It is here evaluated the consequences for Norwegian construction community and best practice. The new conditions are examined including the use of accelerograms and soil conditions.

Keywords: Earthquake design, Low seismic areas, Traditional Norwegian buildings, Eurocode 8

1. INTRODUCTION

Put into a global scale, Norway can be seen as a low to intermediate seismicity area. An analysis of historical data can indicate that earthquakes with a magnitude of 5 or larger on the Richter scale be expected to have a return period of 10 years according to NORSAR. Earthquakes with a magnitude of 6 or larger will have a return period of 100 years. The new design code is based on a magnitude of 6.5 on the Richter scale, which will have a 10 % probability of exceedance over 50 years, i.e. a return period of 475 years. Historical documentation of the largest earthquakes in and around Norway, including the offshore areas shows that the largest records of an earthquake was in the Rana region in 1819 with a magnitude of 5.8. The one in the Vøring Basin in 1866 was estimated to a magnitude of 5.7 on the Richter scale. Finally, the earthquake in the outer Oslofjord area in 1904 has been estimated to a magnitude of 5.4 on the Richter scale.

In the current design provisions for land based structures from 2004 there are limited guidelines on what type of seismic excitation in the form of accelerometric time series should be applied. However, there are in general several possibilities one of which is outlined in Seismic zonation for Norway Bungum et al., (1998) by suggesting three earthquake records, respectively, from Italy, USA and Canada. The published seismic zoning maps for Norway (NS-EN 1998-1:2004+NA:2008). These maps display the horizontal spectral acceleration for undamped natural frequency equal to 40 Hz (T = 0.025 s) and critical damping ratio equal to 5%, see also Fig. 1.

It shown that the response spectrum in the zonation study is significantly lower at low frequencies most likely due to the assumption that the Eurocode 8 spectrum is derived on the basis of data from seismically high active areas of Europe (southern Europe, the Mediterranean). It is stated that it is expected for large earthquakes to have relatively greater low frequent energy because of corresponding differences between source spectra for larger and smaller earthquakes.

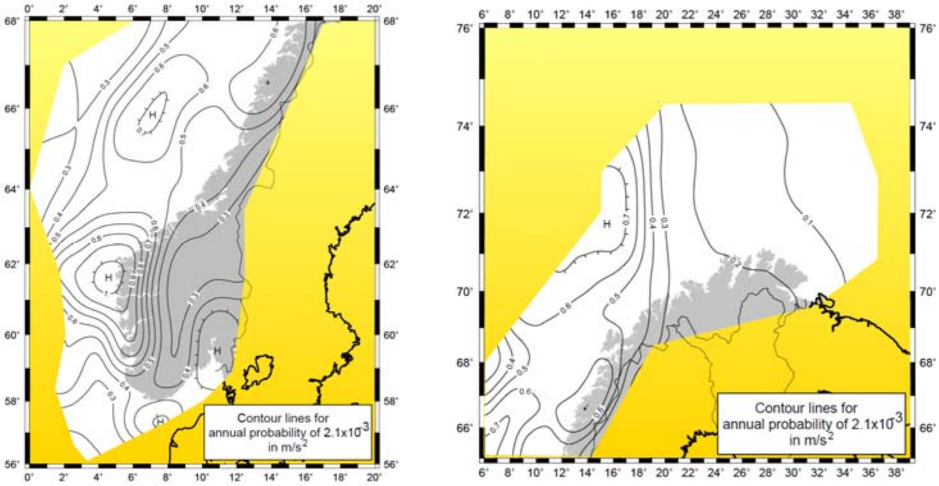

Figure 1. Norwegian Seismic zoning maps, contour lines for annual probability of $2.1 \cdot 10^{-3}$, courtesy of NORSAR

Strong ground motion in the vicinity of an earthquake source is characterized by strong velocity pulses with high energy concentration. The velocity pulses are more prominent in the forward direction, i.e., at stations towards which the fault rupture propagates. The concentration of seismic energy in one or a few cycles of strong pulses causes severe lateral displacement demands on engineering structures. On the other hand, the high frequency energy is relatively lower. These features suggest that a closer look on the seismic design of buildings subjected to near fault effects is also needed.

It has been noted that several cases of design in Norway now is seemingly dominated by the seismic loads rather than the more traditional wind and misalignment loads.

2. THE FENNOSCANDIAN SHIELD AND THE SEISMIC-TECTONIC BACKGROUND OF NORWAY

Norway is geologically part of the Fennoscandian Shield, which also includes Sweden, Finland and the northwestern part of Russia. Archean rocks, the oldest rocks of the Fennoscandian Shield, may be found on the Kola Peninsula, Karelia and northeastern Finland. Magmatic rocks of the Permian Oslo Graben, roughly 250 million years ago (Ma), formed in a failed rift system which continues into the Skagerrak and the North Sea. The Scandinavian Caledonides covers most parts of Norway but also include parts of Sweden. These were formed when North America and Greenland collided with Scandinavia during the Caledonian orogeny, roughly 400 Ma ago, and created the large thrust sheets penetrating some 100 km eastwards over the edge of the Fennoscandian Shield. Areas of the Caledonian deformation including also the Precambrian gneisses of western Norway are cross-hatched on the map provided by courtesy of Åke Johansson, The Swedish Museum of Natural History, Stockholm, Fig. 2.

Oslo, the capital of Norway is located within an aborted Permian rift structure. Granites and magmatic rocks constitute the Oslo rift zone and the southeastern part is tilted about 3 km vertically and filled with sediments, see e.g. Ramberg and Spjeldnes (1978) and Sundvoll and Larsen (1994). The return period for damaging earthquakes is presumably very large although similar rift structures are generally known as the sites of large earthquakes. Also, seismic actions and common building practices as well as soil conditions make the city vulnerable to future large earthquakes. Thus, to consider earthquake damage scenarios for Oslo and Norway is important.

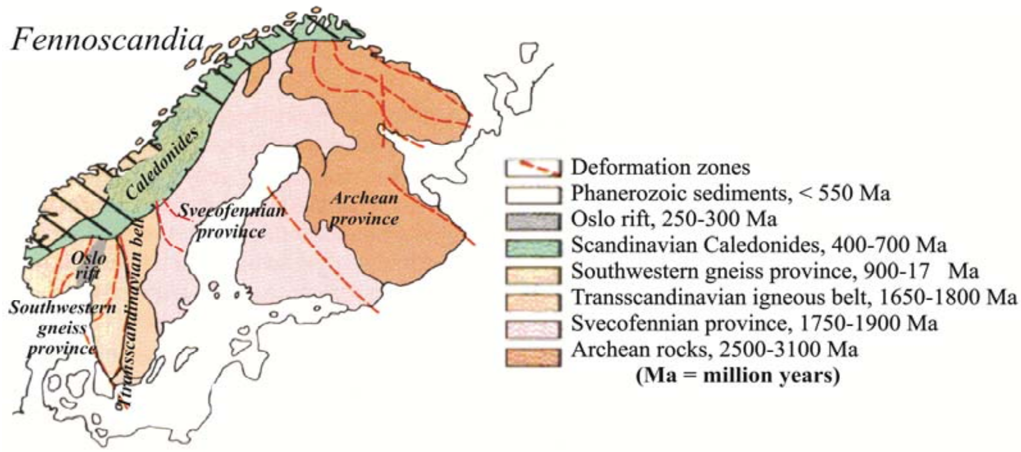

Figure 2. Geological terms part of the Fennoscandian Shield, courtesy of Åke Johansson, The Swedish Museum of Natural History, Stockholm

The first more reliable historical report of an earthquake in the Oslo region is from May 4, 1647, where an earthquake with an estimated magnitude of 4.7 occurred in the outer Oslofjord. The next 250 years shows sporadic earthquake reports, but with generally imprecise descriptions which indicate that these reflect smaller earthquakes. The last major earthquake occurred about 100 km south of Oslo at the mouth of the Oslofjord on October 23, 1904, with an estimated magnitude of 5.4. It was severely felt from the epicenter northwards all along the Oslofjord and caused damage to several buildings. This resulted from an oblique reverse faulting mechanism (Bungum et al., 2009) where the southern part of the Oslofjord rift zone is bounded to the east by a major fault zone. The geometry of this fault zone is unknown to depth, but it is reasonable to assume that the 1904 earthquake reactivated a deeper part of this fault zone

The historical earthquake record shows some concentration of seismic activity along the rift. However, there have not been recorded any severe damaging earthquakes after 1904. The rift structures are part of sites with infrequent large earthquakes which are not easily predicted from the background seismicity, see Gangopadhyay and Talwani (2003) and Zoback and Richardson (1996). Earthquakes of moderate size, M 3-4, are felt in the Oslo region every 3-4 years, and a scenario with a major earthquake on the deeper parts of the boundary fault zone and to the north of the 1904 and 1647 earthquakes is viable. Based on seismo-tectonic considerations, Bungum et al. (2005) have assessed that the zone can tectonically be capable of generating a magnitude 6+ earthquake.

Earthquakes of magnitude 6 or more can lead to significant structural damage, especially in areas where seismic design codes have not been implemented and consequently without buildings and structures being designed against earthquake excitation. Structures and installations on the Norwegian Continental Shelf related to gas and oil exploration and production have been designed for earthquake action already from the early seventies. Land based structures have been subject to earthquake provisions from December 2004 according to NS 3491-12. The present seismic design provisions are dated April 2010 based on Eurocode 8 with an additional national application document (NAD) accounting for the local geological and seismo-tectonic features, and to a certain extent also Norwegian building traditions.

3. HOW AND WHY NORWEGIAN BUILDING TRADITIONS, BEST PRACTICE, MUST CHANGE TO ACCOUNT FOR EARTHQUAKES AND WHAT THIS MAY BRING ALONG OF MORE ROBUST STRUCTURES

One of the reasons for introducing the new series of standards in 2004 was to update the reliability based design of structures. This included new load factors as well as updated load actions. New to

Norway compared to previous sets of design codes was the introduction of a seismic action part, which also in low seismic areas such as Norway should be a part of the structural design guides. This is a load action which has a low probability of occurrence and thus can be include as design criteria that allows for significant levels of structural damage. This type of design can be seen as comparable with or complementary to robustness analysis in structural design. Both of these design criterions are events where it is expected very rare events and which are thus hard to enumerate. The main objective in the new Norwegian codes is to make sure that the new structures survive such events. The seismic actions introduced in Norway should thus ensure that new structures can survive earthquakes up to at least a magnitude of 6.5 on the Richter scale

In Norway, the new seismic actions will only apply to new buildings and for major retrofitting of existing structures. Thus, there are no forthcoming plans to evaluate the existing buildings or building traditions. However, there have been raised concerns regarding buildings of great national importance such as the parliament, and if such buildings should be within the scope of the new codes. There have also been raised questions regarding how suitable the modern Norwegian building traditions are when it comes to their robustness and reliability to withstand seismic actions.

In the new design of buildings it is of great importance to incorporate the necessary changes of the structural system design. Thus, this topic must be addressed early in the design process compared to common practice, increasing the collaboration between architects and structural engineers at an also during the concept phase. Common practice must ensure that the stress distribution to adjacent structural members must be sufficiently secured, especially to achieve high ductility and redundancy. By the later, the energy dissipation can be distributed as widely as possible throughout the structure.

Thus, some established basics in seismic design must to a greater extent be incorporated into the Norwegian best practice; as secure diaphragmatic design in the horizontal direction. These changes also need to include robust boundary conditions to the vertical load carrying structural elements, i.e. walls or moment stiff frames, to tie the building. This has historically not been as demanding for large heavy structures when only assessing the transfer of horizontal forces from wind or misalignment of the vertical bracing, as it is under seismic loading.

New designs by Eurocode 8 design spectrum with national values for Norway are controlled by the behaviour factor defining the ductility properties of the structures. The behaviour factor, is as known, used to approximate the reduction of the imposed seismic actions on the structure by an elastic analysis with assumed ductile behaviour according to current ductility levels. In Norway it can only be used low or medium ductility levels, high is according to the national annex not to be used. The tradition for controlling ductility and overstrength to get redundant structures are not common practice and therefore it is too many structures that are designed by introducing a $q \leq 1.5$, according to DCL. This will render either no seismic loading or seismic actions which are considered purely by elastic forces in the vertical bracings of the structure. Thus, it appears as the approach in Norwegian design practice, so far, seemingly is to build non-dissipative structures rather than to exploit the possibility of dissipative structures in the medium ductility class.

It is important to introduce the necessary ductility and redundancy through the introduction of parallel load carrying paths to transmit the imposed motions from and back to the foundations. By proper member and connection design desired ductility and continuity can be established.

Since no structure will ever be better than its weakest link, it is important to improve robustness in the design practice account for alternative load paths together with enough ductility to secure these paths. At the same time the load paths may be designed such that they allow for sectioning of the structure. This will ensure that a failure event cuts the path to limit the extension of the failure region. Limiting the progressive failure of the structure will naturally affect locations of damage and confine damage development within the building and thus also limit the total costs of structural damage.

4. DESCRIPTION OF THE DESIGN SPECTER INCLUDING MOLINA & LINDHOLM, BASIS OF THE NORWEGIAN ANNEX IN EC8

The Eurocode 8 as well as the Molina-Lindholm spectral forms, Molina & Lindholm (2005) represents a uniform hazard spectrum. The uniform hazard spectra are derived using an earthquake catalogue covering the study region and a suitable ground-motion prediction model (attenuation equation), GMPE. The catalogue is either a collection of historic earthquake data, simulated synthetic earthquake catalogue or, as in most cases, a combination of these. The GMPEs are derived using regression models and available strong-motion data.

The majority of available strong-motion data worldwide can be characterized as far-source data, which results in great weight on the far-source spectral behavior. The result represents a tendency towards higher spectral values for stiff structure than near-source records of short duration would give. The duration of a near-source record is close to the source duration which is typically only approximately 4 to 6 seconds for a magnitude 6 event. The duration, say, for a source to site distance equal to 50 km might, on the other hand, be as great as 15 s, which clearly amplifies the normalized spectral amplitudes in the high frequency range. In the near-source region the seismic energy of the wave motion is shifted towards longer periods governed by the pulse period, which clearly reduces the energy content in the high frequency range.

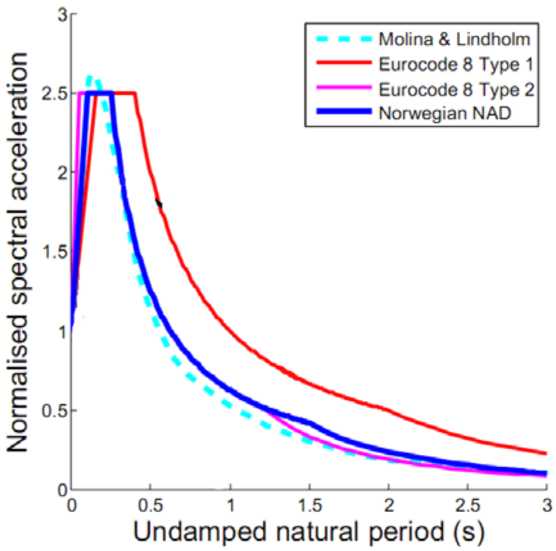

Figure 3. Comparison of response spectra for earthquakes in the magnitude range 5.5 to 6.0: The red curve represent Eurocode type 1 response spectrum; the blue curve represent Eurocode type 2 response spectrum; the bold magenta curve is the Norwegian NAD spectrum; and the cyan curve is uniform hazard spectrum given by Molina and Lindholm (2005). The spectra are normalized using the PGA value.

Scaling of the response spectrum is an important feature when comparing different spectral shapes. Peak ground acceleration, PGA, is a widely accepted scaling factor, where the PGA values are simply picked from strong-motion records. An alternative approach is to use a selected response spectral value as a scaling parameter. In the Norwegian seismic provisions this value has been selected as 40 Hz (T = 0.025 s) spectral acceleration ordinate, which can be compared to the spectral value corresponding to undamped natural period T = 0 s, i.e. the PGA. This results in the Norwegian case of additional scaling factor of 0.8 to bring the 40 Hz results down to the values corresponding to an infinitely stiff structure, i.e. the values given on the hazard maps in the NAD, i.e. NA.3(901 and 902), must be multiplied by 0.8 according to NA.3.2.1 to produce values comparable with the main stream Eurocode 8 representation. This has been taken into consideration in Fig. 3.

5. ACCELERATION TIME HISTORY REPRESENTATION OF THE SEISMIC ACTION

According to Eurocode 8 (3.2.3.1) acceleration time series can be represented by recorded time histories, physical simulation time series of source and travel path mechanisms or artificially generated time series. This also includes representation of the soil conditions deemed appropriate to the site in question, covering possible soil amplification. In artificial accelerograms Eurocode 8 (3.2.3.1.2), when site-specific data is not available the minimum duration of the stationary part of the artificial acceleration time series should be set to 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 zonation report for Norway Bungum et al (1998) indicates that a total duration between 5 and 15 seconds should be expected for Norwegian conditions. Also, the time history should be generated to match the elastic response spectra for 5 % damping. The duration of the accelerograms must, according to Eurocode 8, be consistent with the magnitude and other relevant features of underlying seismic events which is the basis for establishing peak ground acceleration. Suggested time series for Norway can be found in the seismic zonation report, which gives three relevant recorded earthquakes also to be used as basis for the present evaluation in addition to the artificial time history.

Three time series are suggested to represent the design spectrum for Norway, with the recording site given within following brackets. These are; Nahanni, Canada (site 3), Imperial Valley, USA (Superstition Mountain) and Friuli, Italy (Tarcento). All three time series are selected from magnitude, distance and peak ground accelerations which correspond to a low exceedance probability earthquake, Bungum et al (1998). The time series in Fig. 4 are all scaled to the corresponding site Peak ground acceleration, here for the Oslo area. Since this investigation concerns common practice in Norway and possible changes in design practice the time series are scaled to be in accordance with the corresponding design spectra for Norway with a structural behaviour factor for low ductility, i.e. q = 1.5.

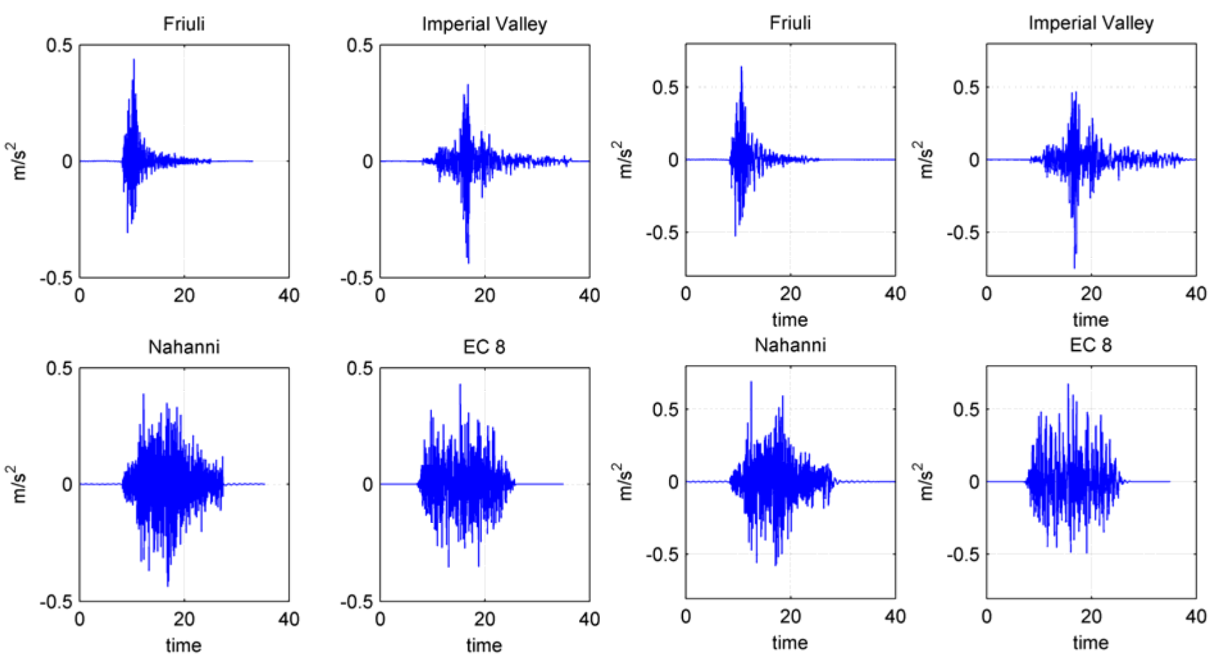

Figure 4. Acceleration time series of Friuli, Imperial Valley, Nahanni and EC8; bedrock (left) and clay (right)

6. CONCERNS OF SOFT SOIL REPRESENTATION IN THE SOIL-STRUCTURAL SYSTEM

It is well known how important soil effects are on the intensity and frequency content of ground motions. Usually these effects are in civil engineering accounted for by wave propagation analysis where either equivalent-linear or nonlinear soil behavior may be included. In EC8 is the problem of soft soil amplification treated by the introduction of various ground type response spectra, which are empirical and may be an oversimplification, see also Ziotopoulou A. and Gazetas G, (2010).

Incorporating the change of softer soil structure response in the present investigation is therefore treated by transfer of the bedrock accelerations through a site specified soil structure before being exposed to the structure. The site response analysis is traditionally performed using the equivalent linear method solving the wave propagation problem through a horizontally layered visco-elastic soil profile. The problem is solved in the frequency domain by complex response method. The non-linear soil response is approximated by an iterative procedure using secant soil stiffness in the iterations. The key parameters for site response analysis are the small-strain shear modulus and mass density of the soil. In addition, to account for the nonlinear soil response during the earthquake is the modulus reduction curve and damping as functions of shear strain in the soil needed. Thus, the analysis structure of the code is kept while better representing the Oslo clay soil structure amplification. This is used to improve on the otherwise wide flat response spectra basis for code generated time series.

A discrepancy can clearly be seen between the design spectra and simulated soil structure, where the code uses a rather crude characteristic for softer soil types (i.e. low stiffness and/or large thickness), which renders a flatter design spectrum. If this were the reality, ignoring SSI for a structure on soft ground would have led to conservative results, as shown in Ziotopoulou A. and Gazetas G, (2010). The simulated spectral distribution shows clear peaks around the site fundamental frequencies for the soil response, Fig. 5, thus showing the opposite trend.

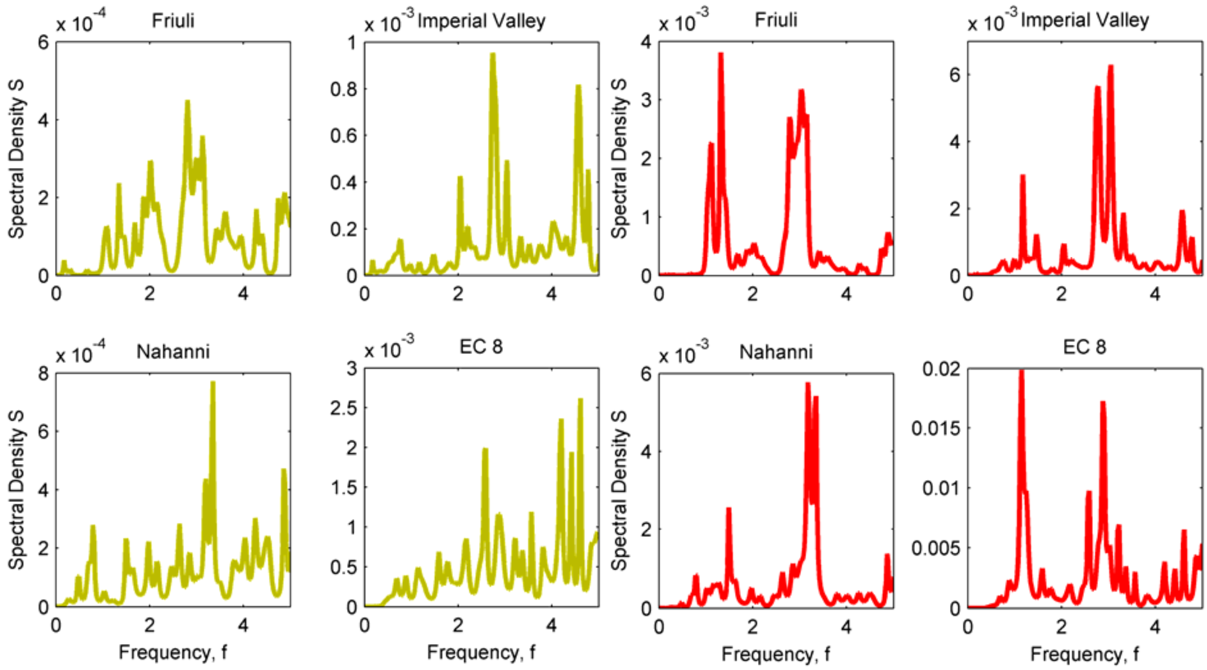

Figure 5. Spectral densities of acceleration time series of Friuli, Imperial Valley, Nahanni, EC 8; Bedrock (left) and Clay (right)

For example may local site characteristics in the Oslo area be such that the structural response may differ substantially from the designated response spectra according to the code. It may therefore be necessary to develop site specific response spectra. This also means that several Norwegian building sites will experience strong seismic motions, even for otherwise low seismic action areas. Care should be taken with these models since they may experience strong amplification at predominant ground natural frequencies for local soil parameters rendering a localized increase in the response spectra as clearly can be seen in Fig. 6.

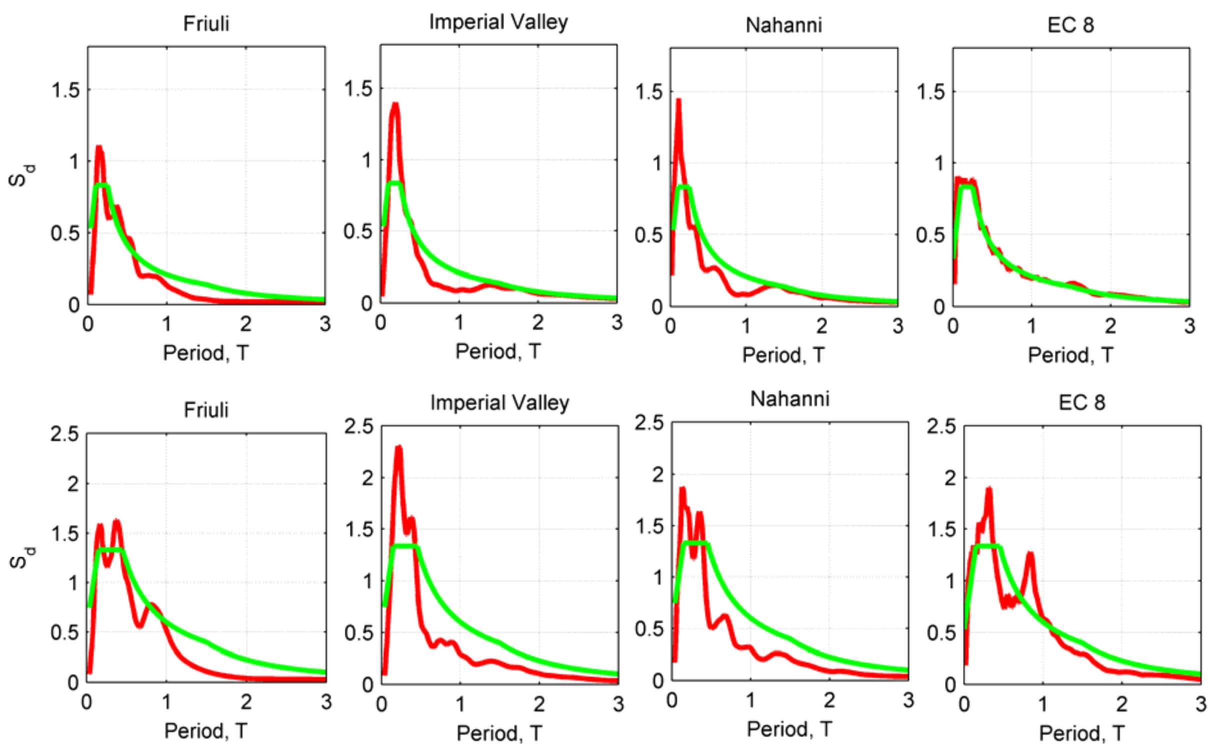

Figure 6. Response spectra of Friuli, Imperial Valley, Nahanni, EC 8; Bedrock (top) and Clay (bottom)

One principle which is especially important for structures on softer soils is to keep the foundation capacity high enough to be able to transfer any overstrength forces. These are the forces transferred from the ductile structural behaviour of yielding in the structural elements rendering the necessary structural behaviour factor corresponding to the, for Norway available, medium ductility class. Thus, it is for typical Oslo clay site areas to design the foundation to always remain linearly elastic, avoiding any unpredicted behaviour while at the same time incorporate the structural soil interaction and reducing stress concentrations under the foundation. This may bring forward longer periods of the structure as fundamental rocking motions, see also Bachmann (2002). This will decrease the sensitivity of the main frame structural components to have high stresses in the most exposed components.

The numerical results for recorded and artificial time histories according to Eurocode 8 above are based on the Norwegian conditions. Thus, the underlying basic parameters for the described seismic actions all in accordance with the Eurocode 8 including all necessary national box values renders; peak ground acceleration, PGA = 0.55 m/s^2; two chosen ground type A (bedrock) and D (soft clay); importance class II is assumed, thus importance factor γI = 1.0; the structural damping is set to 5 %; finally, low ductility class is assumes, thus behaviour factor q = 1.5.

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, 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.

7. A PEDAGOGICAL CHALLENGE FOR THE INDUSTRY; WHAT MEDIA RESPONSE HAS THE NEW DESIGN CODE RAISED FROM THE CONSTRUCTION COMMUNITY?

In [Aftenposten, in Norwegian] 13.10.05, the year after these new Norwegian design code was introduced, were concerns raised regarding Norwegian high rise buildings like the Oslo Plaza and old block of flats, where the latter to a large extent are older traditional masonry buildings. Especially in areas with higher seismic action and with somewhat troublesome marine sediments conditions of soft clays. It is also mentioned that not even the airport control tower at the new Gardermoen airport is designed for seismic actions. Furthermore, there are also concerns raised that the new design action will not have a retroactive effect for existing structures, not even for the Stortinget (the Norwegian Parliament building) [Teknisk Ukeblad 16.10.09, in Norwegian] that was built and taken into use from 1865, thus has survived the 1904 Oslo earthquake.

One of the problems with the introduction of a new load action to be designed for and which will demand changes to the design philosophy is that the latest larger earthquake in Norway was over 100 years ago, in 1904. The Oslo earthquake was measured to 5.4 on the Richter scale. The largest known earthquake in Norway happened in the Lurøy area outside Rana in 1819 and is estimated to 5.8 on the Richter scale, which also is the strongest earthquake in north-western Europe. This, which is compared to the new code demands of a seismic action of up to 6.5 on the Richter scale, will be used.

In an article from 2006 were concerns raised for the increase of construction costs due to new design of structures from seismic actions [Byggeindustrien 22.05.06, in Norwegian]. Among other reasons, these concerns originate from the lack of knowledge in the consulting community regarding design due to seismic actions. They state that this may lead to a large increase of the total costs, as they at the same time not necessarily can see that this will lead to safer or more robust structures, as also stated in [Aftenbladet 28.01.05, in Norwegian]. They can even foresee a large increase in costs when considering a seismic action, which may be 30 times larger than the 1904 Oslo earthquake, with costs that may reach 30 million Euro on an annual basis.

One of the concerns raised are the large increase in construction cost as well as more comprehensive geotechnical and structural analyses due to the new seismic demands. As late as in the present year of 2012 have concerns been raised regarding the increase cost of new buildings due to changing demands of building practice, including seismic actions, [Byggeindustrien: 19.02.12, 22.02.12, in Norwegian]. It states that several new projects have had noticeable increase in structural analysis costs. As an example, they referred to the cost of a new two story building to be used for a kindergarten, which have had an estimated rise of 30 % to the foundation cost due to seismic design. Another concern is also that Norway may have a shortage of competent geotechnical and structural consultants, this during a period where Norway is in great need to increase its building production.

On the other hand, there are also voices which state that the changes shouldn’t be great for Norwegian structures and that the changes in design philosophy will not noteworthy raise structural cost levels [GEMINI 11.12.98, in Norwegian].

CONCLUSION

The Eurocode 8 design requirement was launched in Norway early this millennium. Design for 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. By applying the recommended response spectrum analysis given in Eurocode 8, may render horizontal design forces several times that of designing wind forces. This will become a challenge for new buildings but more so for retrofit of existing buildings, e.g. to extend a building with an additional floors.

Introduction of design against seismic action in Norway represent challenges to the profession. For combinations of earthquake hazards and type of structure that require earthquake design, it will be advantageous to apply modelling and methods of analysis that to a sufficient degree demonstrate the properties of the structure to withstand the earthquake excitation. In order to then be able to dimension for earthquake resistance in a safe an economical way the structural system must be suitable this type of load action. The new design code may set new organizational demands on the entire building process. This then go towards engineers working within conceptual design, i.e. among other considerations focus on symmetry, continuity and robustness. This clearly must also include early understanding and collaboration between consulting engineers as well as across all fields with all participants of the project, such as architect, civil engineer and the building owner. Thus, this will most likely be a great pedagogical challenge for the industry to rethink the traditional forced based design, incorporate continuity and robustness and not the least to accept that earthquake can and will happen, also in low-seismic areas such as Norway.
However, since in Norway there are no plans to introduce any retroactive regulation for existing structures, besides major retrofitting, this will have no influence on the existing buildings.

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