Effects of soil-structure interaction on the excitation and response of RC buildings subjected to strong-motion

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ABSTRACT
The objective of the paper is to investigate if and how site effects caused by a complex combination of rock and sediment layers affect the response characteristics of reinforced concrete buildings. The study focuses around two buildings in the town of Selfoss in South Iceland. The buildings were monitored during and/or after major earthquake events. In an effort to recreate the observed variation in the structural behaviour, a finite element model has been constructed of the structures including the foundation effects through simple spring elements. The models have been applied to match the numerical response to the recorded response. Analyses of recorded seismic response indicates considerable influence from the foundation on the structural behaviour and the dynamic response of the reinforced concrete structures. It is also deduced, that when subjected to an earthquake, the dynamic properties of the soil/rock foundation will change in terms of shear strength and damping as a function of the intensity of the excitation. This is probably especially true for the underlying sedimentary layer as the amount of strains in the sedimentary layer to a large degree controls the frequency response. The response spectra obtained indicate that the large damping and the excitation frequency shift created by the soil layers in the foundation generally reduces the higher frequency amplitudes of the accelerations at basement level within the buildings but increases the lower frequency acceleration amplitudes. For tall and medium-rise buildings this may increase the overall acceleration levels of the building compared to a rock based foundation, whereas for low-rise buildings this may reduce the story drift and thereby the overall structural strains.

Keywords: Earthquake, Site effect, Acceleration, Building, Response.

1 INTRODUCTION

Earthquake induced strong-ground-motion in Iceland has been monitored over the last three decades (Sigbjörnsson et al., 2014). In earthquake engineering practice in Iceland, site effects are generally not considered in earthquake resistant design, mainly because before construction starts, the relatively thin topsoil is in most cases removed from the uppermost competent rock (e.g., lava rock, hyaloclastite, dolerite, etc.). However, for lava rock the presence of pronounced site effects was reported after the earthquake in June 21, 2000 (Bessason and Kaynia, 2002). The study presented herein, revolves around two specific buildings located in Selfoss, a rural town in south-Iceland, which is located within the South-Iceland-Seismic-Zone (SISZ). Earthquake induced acceleration and ambient seismometer data is available for both buildings, in addition to structural analysis and finite element modelling.

Recorded data and observations made during three strong earthquakes within the South-Icelandic seismic zone, two in June 2000 and one in May 2008, with magnitudes of 6.5, 6.4 and 6.3 respectively (Sigbjörnsson et al, 2007 and 2009), revealed an interesting and somewhat unexpected phenomenon influencing structural response. The paper will discuss some of the key findings regarding the response characteristics observed and the relevance of the geological settings for earthquake resistance of similar buildings.

2 SURFACE GEOLOGY

The surface geology in South Iceland was mostly formed during and after the last ice age. The costal topography is of low elevation, approximately flat and largely covered with postglacial basaltic lavas, as well as tuff layers, intercalated with Quaternary sediments of mainly fluvial, glacial and glaciofluvial origin (Atakan et al., 1997).
The youngest lavas are from the Holocene (not more than 200 years old), while the oldest formations are up to 3.3 Myr old.
During glacial periods, Iceland was covered with a plateau glacier. During warmer interglacial periods, the ice melted and the glaciers retreated, which resulted in sea level changes of up to 200 m. The South Iceland Lowland was then, a seabed, accumulating marine sediments. During warm periods and towards the end of the Pleistocene, when the glaciers were retreating and the land was undergoing isostatic rebound, glacial streams formed thick sediment layers, composed chiefly of sand and fine-grained gravel.
In the postglacial period, some of these sediments were covered by lava, which adds to the complexity of the geological structure of the surface. The lava layers may be as thick as 10 m while the sediment layers can be up to 20 m thick or even more.
This has resulted in geological profiles consisting of recurring layers of basaltic lavas, as well as tuff layers, often with intermediate layers of sediments or alluvium.
It will be demonstrated herein, that this geological structure has an effect on overall structural excitation and behaviour in earthquakes.

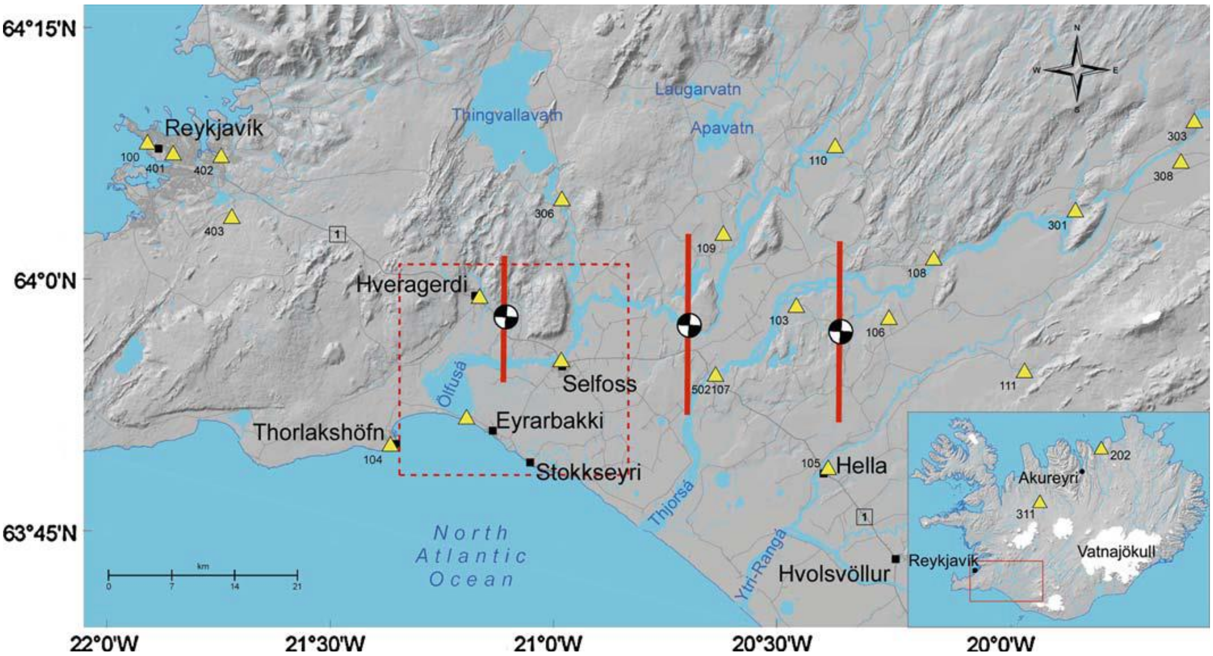


Figure 1 A map, showing the South-Iceland-Seismic-Zone, the epicentres of the Earthquakes in 2000 and 2008 and the location of the town of Selfoss (Sigbjornsson et al. 2009). The triangles show the locations of the recording stations of the Icelandic Strong-motion Network.

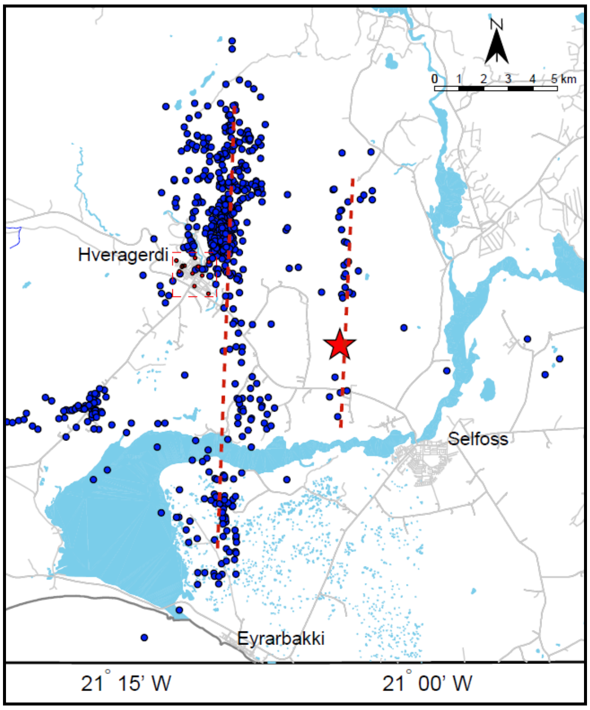


Figure 2 The N-S trending alignments of the seismicity distribution of aftershocks for the 15:45 UTC 29 May 2008 Ölfus earthquake (blue circles) indicate the location of the causative faults (dashed lines). The red pentagram shows the epicenter of main-shock.

3 THE EARTHQUAKE EVENTS

A damaging South Iceland earthquake sequence began on 17 June 2000 at 15:41, with an earthquake that had an epicentre just north of the rural village of Hella (see Figure 1). The earthquake had a surface wave magnitude of 6.6 and a moment magnitude of 6.5. It was followed by major seismic activity throughout the entire South Iceland Seismic Zone.

The second earthquake in the sequence occurred on 21 June 2000 at 00:52. It had a surface wave magnitude of 6.6 and a moment magnitude of 6.4. The epicentre was approximately 17km west of the epicentre of the first event (see Figure 1) (Sigbjörnsson et al. 2007).

A Third damaging earthquake occurred in South Iceland on Thursday 29 May 2008 at 15:45 UTC. The epicentre was in the Olfus District about 8 km north-west of the town of Selfoss (see Figure 2). The magnitude of the earthquake was 6.

The earthquake can be characterized as a shallow crustal earthquake. It occurred on two parallel, near vertical north-south trending right-lateral strike slip faults approximately 4.5 km apart. While the epicenter was located on the eastern fault, around 5 km N-W of Selfoss, the majority of the aftershocks occurred on the western fault that ruptured ~2 seconds after the eastern fault. The basic properties of this event are found to be similar to the characteristics of the South Iceland earthquakes in June 2000 (Sigbjornsson et al. 2009).

4 THE BUILDINGS

Both buildings studied herein, are located in the town Selfoss (see Figure 1 and 2). The riverbed of Ölfusá (e. Ölfus river) runs through the centre of Selfoss. On the west bank there is relatively solid bedrock from the last ice age both lava and tuff, ca 1.500.000 years old. On the east bank, on the other hand, there is the large post glacial Thjórsárhraun lava field ca 7.800 years old. The Thjórsárhraun lava is the biggest scoria lava field in Iceland exceeding 900 km^2 and may have flowed over distances of 120 km. The lava thickness may be up to twenty or thirty meters. The river has been running in its current riverbed from the time of Þjórsárhraun. As the old bedrock on the west side of the river is much more solid than the Þjórsárhraun lava the river has mainly been eroding the softer bedrock of the east bank.

4.1 The Town Hall

The Town Hall at Selfoss (see Fig. 3) is a cast-in-place reinforced concrete building with 3 stories and a basement. It is about 11 m high from ground level to the rooftop, and about the same distance from the basement floor to the top floor, with the basement reaching 4 meters below ground level. It is rectangular in plan, about 41 m long (east-west direction) and 13 meters wide (north-south direction). The structural system is composed of outer shear walls a shear core and two rows of interior concrete columns and interconnecting floor beams that carry the slabs. The orientation of the building is such that the length of the building approximately aligns along an ESE-WNW axis. The building was renovated and retrofitted in the period of 1997 to 2001.

Figure 3 The Town Hall at Selfoss, view from NE.

The building was instrumented in 1999 and earthquake induced acceleration data has been systematically collected there since. The instrumentation is located at two levels, the basement and the top storey (the 3rd floor if the ground floor is no. 1). A tri-axial accelerometer is located in the elevator shaft in the basement, measuring the three components of base (ground) acceleration. On the top floor three uni-axial accelerometers are located, one measuring motion in the E-W direction and two measuring in opposite corners (i.e. N-W and S-E) measuring motion in the N-S direction. This makes it possible to detect torsional effects in the building response. The sampling rate is 200 Hz.

The accelerations induced by the earthquakes of June 2000 and May 29, 2008 were recorded in the building, both at the basement level and on the third floor. The events in June 2000 had an epicentral distance of 32 km and 15 km from the building and the ground acceleration at the building site was less than 15% g (see Table 1). The structural capacity of the building was therefore not severely tested at that time.
The epicentre of event in May 2008, was less than 8 km away, and the peak ground acceleration (PGA) at the building site was 54% g. It is therefore not surprising that the Town Hall suffered some minor structural damage in this event such as visible cracks in the concrete walls. The damage was however not sufficient to change the structural characteristics of the building which continued to serve its function.
Unexplained dissimilarities are observed in the structural response characteristics for these two events. It is suspected that site effects and/or soil-structure-interaction effects are causing the differences observed. The peak accelerations recorded during these three main events are listed in Table 1.

Table 1 Peak accelerations recorded at the Town Hall in 2000 and 2008.

Date of event	Magnitude	Distance from site (km)	Peak ground acceleration (%g)			Peak response acceleration (%g)		
			Vert	N-S	E-W	W: N-S	C: W-E	E: S-N
June 17, 2000	6.5	32	2.9	7.6	5.5	14.6	12.1	15.8
June 21, 2000	6.5	15	6.8	12.7	11.2	30.2	21.4	29.2
May 29, 2008	6.3	8	26.6	53.8	33.4	74.6	47.3	68.2

4.2 The Bell Tower

The bell tower of the Church of Selfoss is a square 4 m by 4 m cross section, see figure cast-in-place, reinforced concrete shear wall building (see Figure 4). It is approximately 23 m high, making it one of the tallest building in Selfoss. The tower is connected to two small adjacent buildings, of an approximately 7 m in hight on the North and the East side.
The roof is made of timber girders and stiffeners of timber and a cooper cladding.

Three church-bells, 75 cm, 67 cm and 56 cm in diameter and weighing 240 kg, 170 kg and 100 kg, respectively, are at the top of the concrete structure. Each level of the tower is approximately 3 m in height having concrete staircase in between each level.
During the earthquake a fracture opened through the entire perimeter of the tower, see Figure 5. The fracture opened both on the outside of the tower as well as on the inside. The location of the crack is at the height of the adjacent buildings, i.e. at about 7 m from the ground. This location is vulnerable due to abrupt changes in the stiffness of the tower at

this height, but also there was also a construction joint at this level.

Figure 4 The bell tower of Selfoss church.

Figure 5 The crack in the Bell tower.

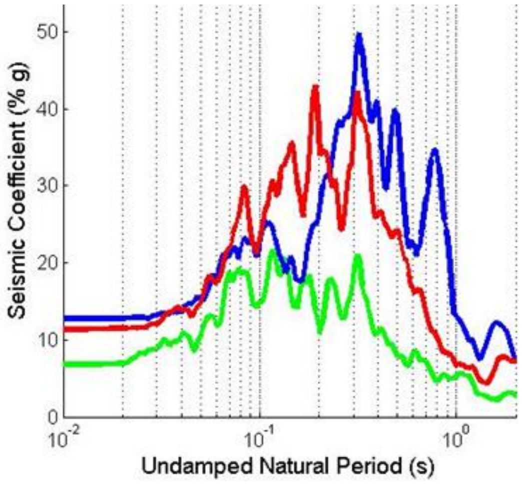
No instrumentation was in the bell tower when the three largest earthquakes occurred. The nearest instrument was located in the Town Hall of Selfoss about 350 m east of the tower.
After the earthquake on May 29, 2008, and the consequential damage to the tower, two strong-motion accelerograph units were installed on the top-floor of the tower in the south-east and north-west corner of the tower. The locations were selected to obtain the torsional motion of the structure. These two units monitored response to aftershocks and ambient vibration of the tower.

5 STRUCTURAL ANALYSIS

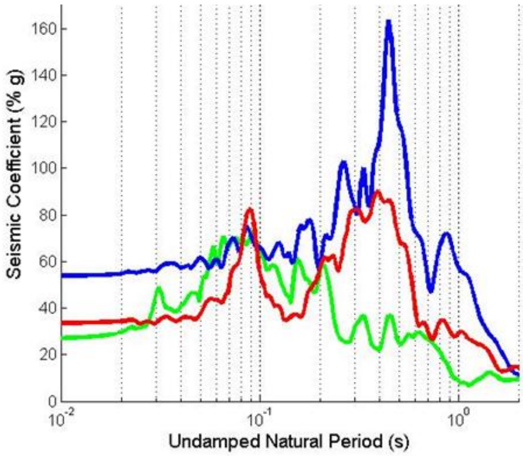
The behaviour and response of both buildings have been analysed on the bases of FE-modelling and recorded acceleration. In both instances it is observed that the recorded response cannot be described by a structural model of the buildings alone. This will be demonstrated further in the following.

5.1 The Town Hall

The instrumentation system in the Town hall recorded all three main events as well as many smaller earthquake events over the last 17 years. Herein the focus will be on two earthquakes, i.e. the M6.4 on June 21, 2000 and the M6.3 on May 29, 2008,
When comparing the recorded surface and response accelerations from these two events a clear difference in the frequency content and thereby the characteristics of the response is noticed. The response spectra are computed from the recorded accelerations in the basement and shown in Figure 6. The horizontal components show a significant response from about 0.1 s up to 0.8 s, for both events. However, for the 2008 event a strong response peak is observed at 0.4-0.5 s. This response peak is likely caused by local site effects that control the frequency content of the surface motion. The vertical acceleration response seems less affected by possible site-effects but the response spectra for the May 2008 event is slightly shifted towards the low-period range. This is most likely caused by the near-field characteristics of the motion, enhancing the contribution of the low period pressure wave.
Figures 7 and 8 reveal the frequency content of the relative accelerations at the third floor during these two main events. Comparing the E-W components (see Figure 7), shows more or less the same spectral peaks but they are shifted about 1 Hz towards lower frequency values for the 2008 event.



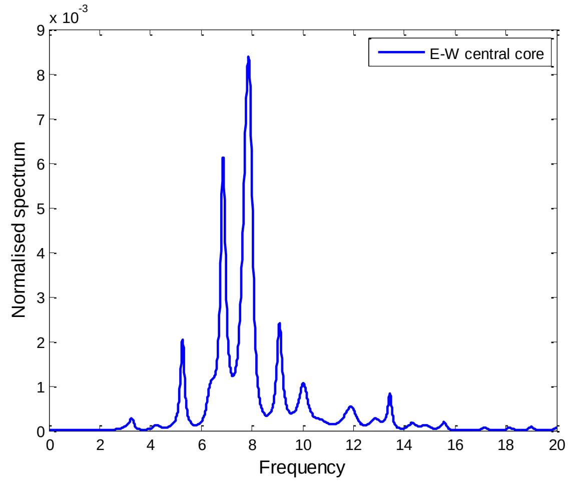
(a)



(b)

Figure 6 Response spectra evaluated from the basement records. (a) The event on June 21. 2000. (b) The event on May 29. 2008. The blue line is the N-S component, red line is the E-W component and the green line is the vertical direction. The damping is 5% of critical damping.

Comparing the N-S components (see Figure 8), a much more dramatic difference is observed. Again the frequency peaks are shifted about 1 Hz towards lower frequency values for the 2008 event, but it is also seen that the energy is distributed between 2 and 5 Hz in the 2008 event whereas it was mainly distributed between 5 and 10 Hz in the 2000 event. This dramatic shift in frequency content may indicate that the site-effects dominate the response from the earthquake in 2008. The difference in the response between the east and west gable demonstrates that the east side of the building has considerably more stiffness in the N-S direction than the west side of the building.



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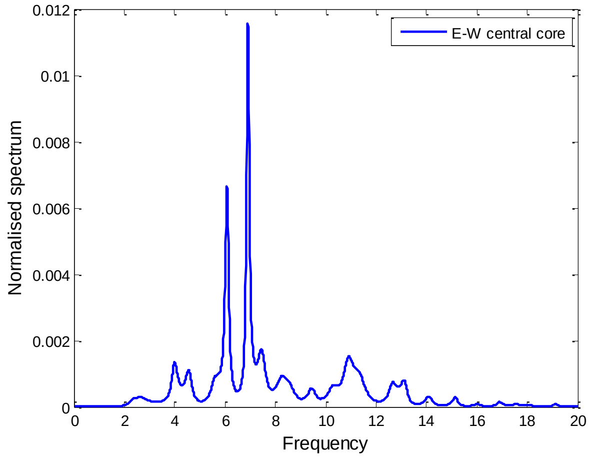
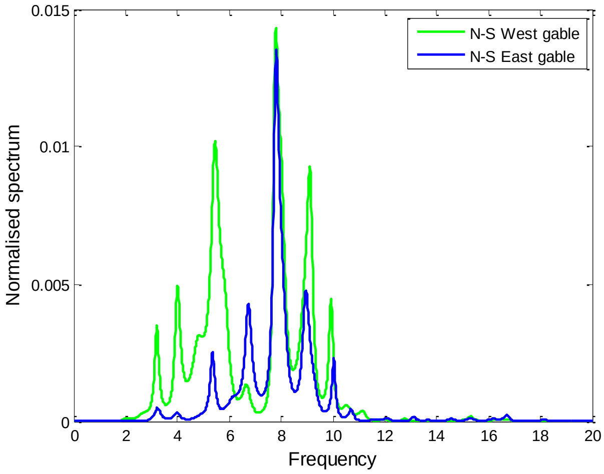


(b)

Figure 7 Power spectral densities of relative acceleration response on the third floor. (a) The E-W comp. from the event on June 21. 2000; (b) The E-W comp. from the event on May 29. 2008.

A system identification was done using a combined subspace algorithm provided by the MACEC toolbox (Reynders et al. 2011). The basement records were used as an input in an MIMO analysis. The shift in frequency for comparable mode shapes was confirmed and several modes are found in the 2008 event that are not seen in the 2000 event. Figure 8 shows the H/V-ratio (Nakamura 2008) evaluated based on the recorded basement motion in June 2000 and May 2008. As can be seen the H/V-ratio further establishes the difference in the soil-rock response between the two events studied. In 2000 the main soil-response is seen to be at 8 and 15 Hz, whereas in 2008 the main soil-response is seen at 2 and 4 Hz. This indicates that the shear wave velocities in the rock-soil layers underneath the building were considerably lower in 2008 than in 2000.



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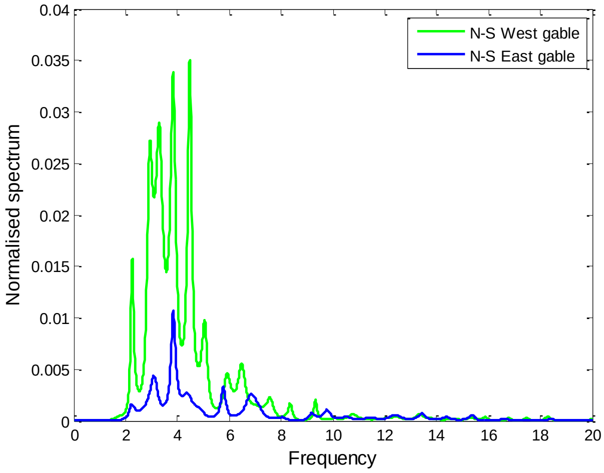


(b)

Figure 7 Power spectral densities of relative acceleration response on the third floor. (a) The N-S components from the event on June 21, 2000; (b) The N-S components from the event on May 29, 2008.

Two different finite element models have been made of the Town Hall, using Ruaumoko-3D (Stray, 2010) and SAP2000. Both models verify, along with system identification of data, that the fixed-base building has natural frequencies above 6 Hz, as seen in figure 7(a) and 8(a).

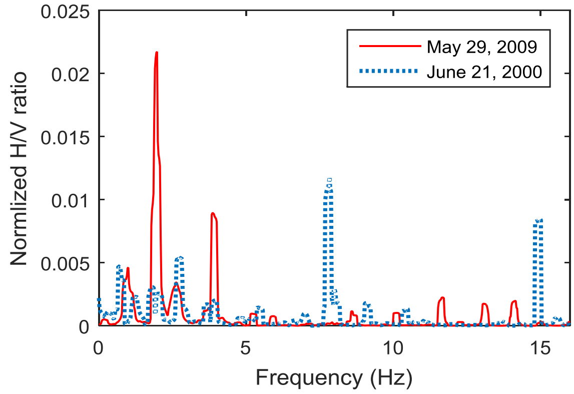
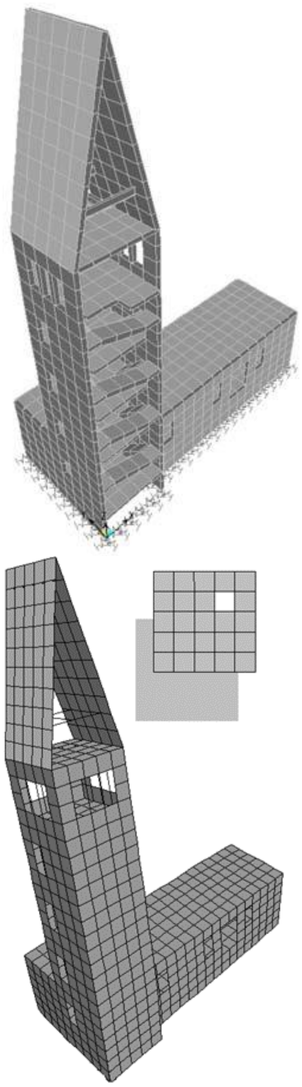


Figure 8 Normalised H/V-ratio as a function of frequency for the earthquakes in 2000 and 2008.

5.2 *The Bell Tower*

A three-dimensional linear finite element model of the tower was created using the finite element computer programs SAP2000 and ETABS (Bardarson, 2009). The model was based on the design drawings of the tower as well as an on-site survey and measurement of the dynamic modulus of elasticity. The tower was modelled using shell and frame elements. The FE model (see Figure 9) was designed to represent the structural geometry as accurately as possible and the weight of the roof structure and the church bells was included to insure a correct mass distribution.



(a) (b)

Figure 9 The Finite element model with spring supports (a) and its first mode of vibration (b). The first mode has a frequency of 5 Hz.

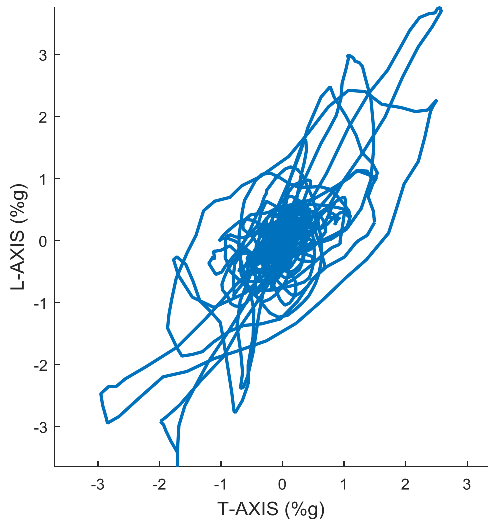


Figure 10 A plot of the two horizontal acceleration components recorded in an aftershock.

During test runs the model was not found to represent the dynamic behaviour observed in the aftershock and ambient excitation data recorded in the tower. The model was considerably stiffer, with a first natural frequency of 7.5 Hz, whereas the recorded data showed response at 5 Hz (0.2 s). After some consideration, it was determined necessary to consider foundation-structure interaction in the modelling to reproduce the observed behaviour. The foundation-structure interaction was modelled by placing springs at each foundation joint of the structure. According to Wilson (2000), the use of appropriate site-dependent free-field earthquake motions and selection of realistic massless springs at the base of the structure is the only modelling assumptions required to include site and foundation properties in earthquake analysis of most structural systems.

Ambient vibrations have been successfully applied to calibrate computational models in dynamic analysis of civil engineering structures (Jaishi and Ren, 2005). After the introduction of properly tuned springs, the model was found to correspond well with the recorded data. Figure 10, shows a plot of the two horizontal acceleration components for an aftershock record. The main vibrations follow a diagonal pattern, consistent with the first mode of the structure as calculated by the FE-model (see Figure 9b).

In 2002 test holes were drilled at four locations, along the Ölfusá river. One of them at the riverbank close to the church. Based that hole the geological profile shown in Figure 11 was put forward for the church site by Imsland (2002).

The tower and the layered soil profile characterized by velocity reversals was modelled as a classically damped 5 degree-of-freedom linear oscillator. Where each model component, i.e. the tower, the lava, scoria and sediment layers, have a mass and stiffness. The mass and stiffness of the tower is taken from the FE-model whereas the mass and stiffness of the soil layers are based on reasonable estimates of density (2200, 2000 and 1800 kg/m^3) and shear velocity (1800, 1200 and 200 m/s). The effective area of the tower foundation is used to define the masses. Eigen frequency analysis gave the results shown in Figure 12. It can be noticed that the first mode has a frequency of 5.2 Hz, which corresponds well with the acceleration recordings and the updated FE-model. Excluding the tower from the model and considering only a soil-rock column of unit area gave the same frequency for the first mode, i.e. 5.2 Hz. It should be noted that modes 2 and 3 have a much higher natural frequency of 40 and 72 Hz.

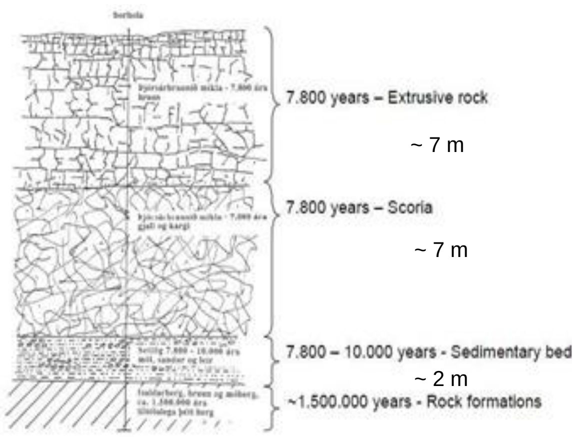


Figure 11 A hypothetical rock-soil profile

5.3 Evaluation of HSVR

Spectral analysis of microseismic (ambient) vibrations via the HVSR method are used to estimate the site response in urban environment (D'Amico, 2008). Microseismic data is easily obtained and can provide additional constraints on site characterization using the HVSR method.

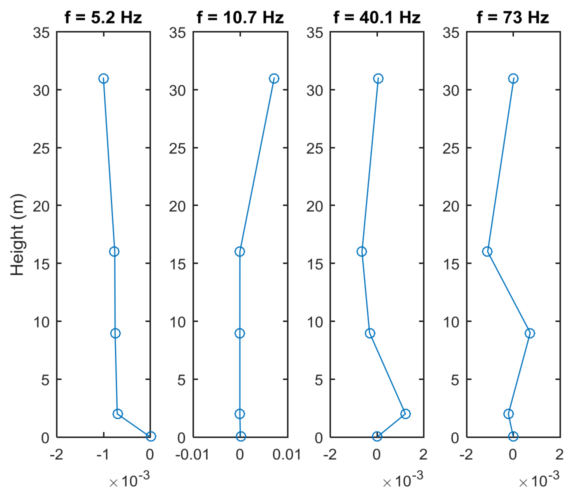
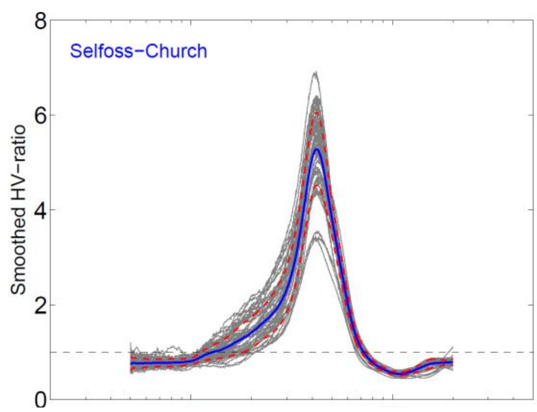


Figure 12 The modes of vibration of a simple soil-structure model.

Continuous ambient noise recordings of minimum one-hour duration were performed at ISMN sites in Selfoss, as well as in the church, using REF TEK 130-01 Broadband Seismic Recorders and Lennartz LE-3D/5s Seismometers. Raw data from these sites have been analysed using Nakamura's method, Horizontal to Vertical Spectral Ratio (HVSR) (see Nakamura, 2012). The HVSR method uses three-component single station ambient noise records, and involves the ratio of the combined horizontal frequency spectrum, H, to vertical, V, frequency spectrum at the studied site. The final mean HVSR for each site was determined by calculating the geometric mean of the HVSRs from all the individual time windows analysed. The results are shown in Figure 13. The HVSRs figures show magnification at frequency between 4 and 5 Hz, which is observed very strongly at the church site, but less so at the Town-Hall. Then there is some response at 7 Hz in the plot for the Town-Hall.

It is well know that the shear stiffness of the soil layers is very dependent on the state of shear stress in the soil, decreasing rapidly with increased strain levels (Kramer, 1996). Therefore, ambient measurements of this type are not conclusive regarding site effects in strong near-fault earthquakes, but it is interesting to see that the frequency of the HSVR response at the church sites more or less coincides with previous evaluation based on monitoring and structural modelling.



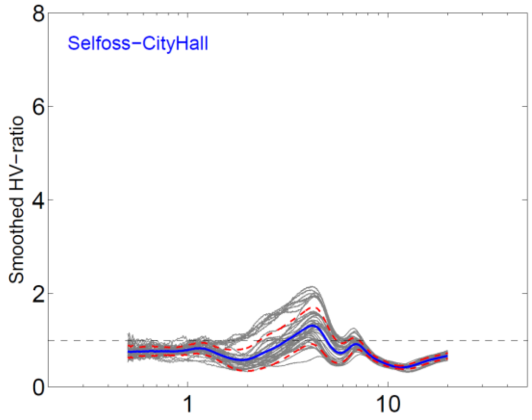


FIGURE 13 Mean HVSR +/- one standard deviation from ambient noise measurements. The grey lines show 20-min ambient noise windows used to derive the mean HVSR (blue line) and standard deviation (red dashed line).

6 DISCUSSION AND FINAL REMARKS

The analysis confirm the influence of the soil-rock layers beneath the buildings on the building response. It seems clear that the behaviour of the different soil-rock layers underneath the building should be recognized and included in the structural modelling. Especially, the soft sedimentary layer that exists underneath the younger lava layers. When subjected to earthquake motion of different intensity, the material properties will change in a non-linear fashion in terms of shear strength and damping. The amount of strains in the sedimentary layer will, to a large degree control the frequencies of response. There are also indications that the damping effects created by the soft layer could reduce the amplitude of the ground accelerations, as well as lowering the frequency content of the excitation.
The site effects/soil-structure interaction observed in the earthquake in May 29, 2008, has most likely been beneficial for the earthquake response of the buildings.
In the wake of the latest South-Iceland earthquake sequence that started in 2000, the importance of the foundation on the overall earthquake behaviour of buildings has become clear. A good understanding and modelling methods that include soil-structure interaction and or site-effects are therefore essential, both to insure a reliable design of new structures as well as for estimating the risk of damage for existing buildings. The case studied herein, is intended to serve as a contribution to increased awareness of the importance of site-effects and soil-structure interaction.

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