Soil-structure interaction effects on the excitation and response of a low-rise RC building subjected to near- and far-fault earthquakes

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SUMMARY:
Earthquake induced acceleration data has been systematically collected in the Town Hall of Selfoss, Iceland, over a ten year period. The objective is to analyse and compare the seismic behaviour of the Town Hall during two large earthquake events that occurred in the vicinity of the building on June 21, 2000 and May 29, 2008, with magnitude of 6.5 and 6.3 respectively. The ground underneath the building is expected to consist of at least three different soil and rock layers. The behaviour of these different layers needs to be understood in order to be able to explain the different response characteristics observed from the two earthquakes. The material properties of the soil layers will change in terms of shear strength and damping as a function of the intensity of the excitation. However, for exploratory purposes the soil/rock layers were modelled with a linear model for each event. The procedure provides an indication of how the soil-structure interaction affects the earthquake response characteristic and the structural behaviour.

Keywords: Strong motion, acceleration data, soil-structure interaction, earthquake response

1. INTRODUCTION

The study revolves around a three story office building (see Fig. 1.1) located at Selfoss, a rural town in south-Iceland. The town is located within the South-Iceland-Seismic-Zone (SISZ) and has been subjected to many small earthquakes since built in the 1940's. The building was instrumented in 1999 and earthquake induced acceleration data has been systematically collected there since. The focus of this study is on two strong earthquakes, in June 2000 and in May 2008, with magnitudes of 6.5 and 6.3 respectively (Sigbjornsson et al). The accelerations induced by these earthquakes were recorded both at the basement level and at the third floor. Unexplained dissimilarities were observed in the structural response characteristics for these two events. It is suspected that the differences can be explained by soil-structure-interaction effects.

1.1. The structure

The Town Hall at Selfoss (see Fig. 1.1) 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 and a shear core around the stairway and elevator shaft. In addition there are two rows of interior concrete columns and interconnecting floor beams that carry the slabs (see Fig. 1.3). The orientation of the building is such that the length of the building approximately aligned along an ESE-WNW axis. As the building was built in wartime conditions structural drawings are only partially available and do not indicate exactly the location or the amount of reinforcement. The building was retrofitted and renovated during the period 1997 to 2001. The EQ retrofitting included additional concrete shear wall units strategically placed to improve the structural behaviour and enhance and improve the balance of the overall resistance of the building.

Two steel cross-braces were also inserted into the window frames at ground level on the north side. The layout of these additional structural elements is partly shown in Fig. 1.1 and 1.3. Fortunately this work was finished before the occurrence of the large earthquakes in 2000. After the earthquakes in June 2000, the top level was renovated. A 2.5 cm concrete levelling layer was removed and a 5 cm layer of reinforced concrete was added to strengthen the floor. Then the top floor was partitioned with double gips walls. This work has added significant mass to the top floor in addition to providing an increased diaphragm stiffness of the floor. Therefore, in many ways the available database for this building is composed of subsets for at least 3 somewhat different structural conditions depending on observation period.

Figure 1.1 The Town Hall at Selfoss, view from Northeast.

During the earthquake in 2008 the Town Hall suffered some minor structural damage and cracks could be observed in the concrete walls. Two examples of these cracks are shown in Fig. 1.2. In Fig. 1.2a one can see the crack has propagated through the wall along the stairwell. Fig. 1.2b shows a crack in the outside wall.

(a) (b)

Figure 1.2. a) Observed cracks in the building walls after the earthquake on May 29th in 2008: (a) In the stairwell wall; (b) in the outside wall.

1.2. Instrumentation and recordings.

The building was instrumented in January 1999. Figure 1.3 shows the arrangement of the instrumentation as well as the structural elements of the ground floor. The instrumentation is located at two levels (see Fig. 1.3): the basement, and top storey (the 3rd floor if the ground floor is no. 1). A Kinematrics-K2 digital recorder with an internal 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. The red boxes in Fig. 1.3 represent the location of the recording instruments in the third floor. The arrows indicate the positive direction. The blue triangle represents the accelerometer in the basement. The uni-axial sensors (channels) are connected to the K2 digital recorder in the basement. The sampling rate is 200 Hz. The data acquisition starts automatically when the acceleration exceeds a specific trigger level, which can be defined for each channel (0.05-0.075%g in the basement, 0.20-0.25%g on the top floor).

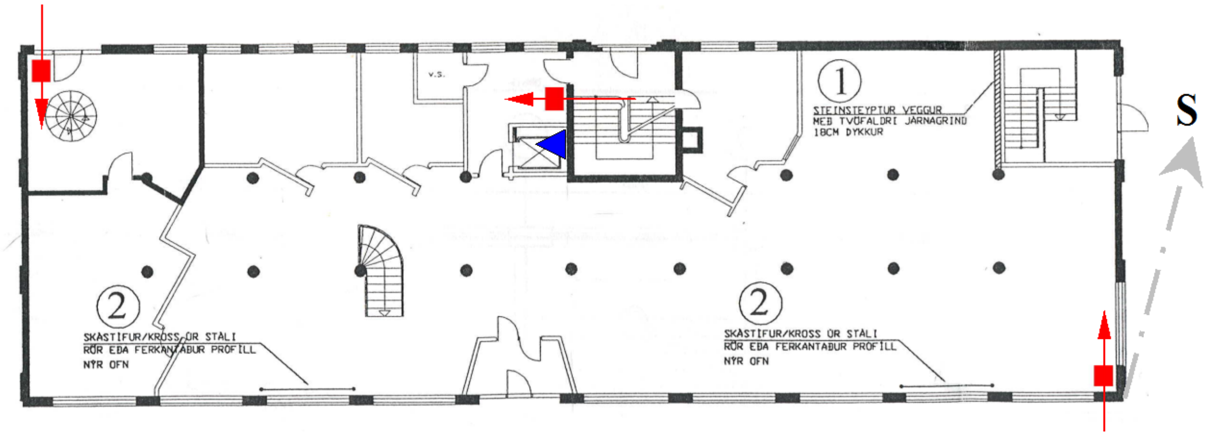


Figure 1.3. The Town Hall at Selfoss, plan view of the ground floor. The location of uni-axial (squares) and tri-axial (triangle) accelerometers within the building plan as well as their positive direction is shown. Note the location of the steel cross-braces (2) and reinforced concrete wall (1) installed in spring 2000 for retrofitting.

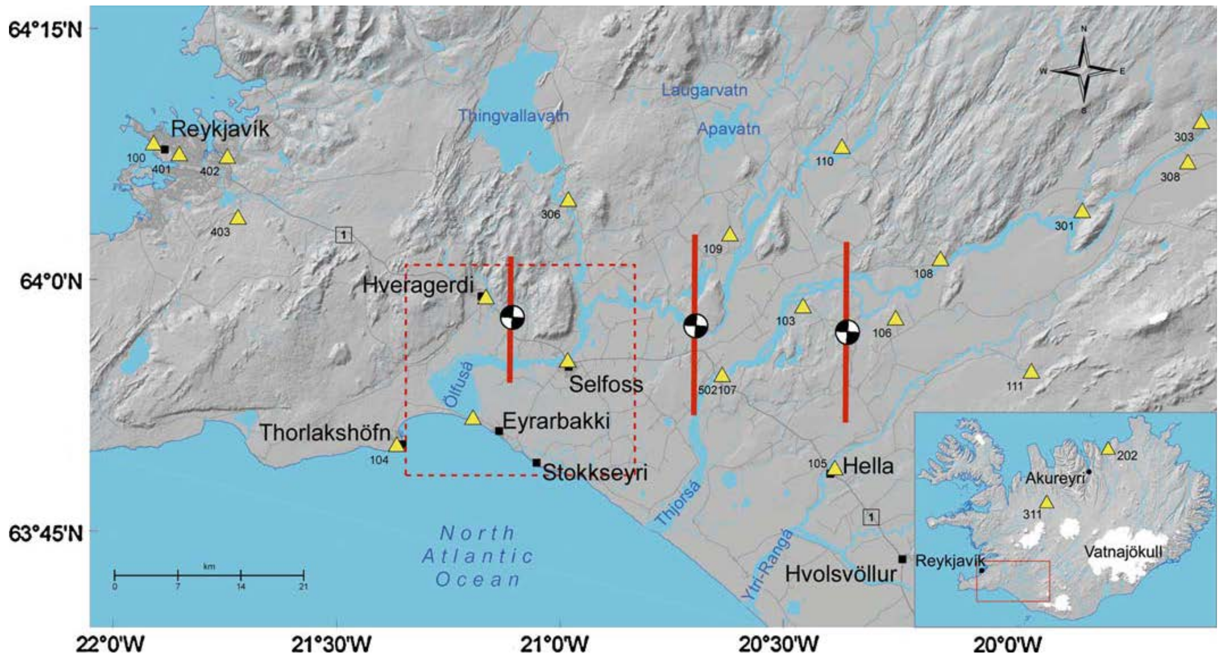


Figure 1.4. A map, showing the South-Iceland-Seismic-Zone, the epicentres of the Earthquakes in 2000 and 2008 and the location of the town of Selfoss (Sigbjornsson et al. 2009). The triangles show the locations of the recording stations of the Icelandic Strong-motion Network.

1.3. The earthquake events studied

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. The earthquake had a surface wave magnitude of 6.6 and a moment magnitude of 6.5 (Global CMT Catalog 2007). It was followed by major seismic activity throughout the entire South Iceland Seismic Zone, including the Hengill area, northwest of the town of Hveragerdi, and the Reykjanes Peninsula. 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 (Global CMT Catalog 2007) (Sigbjörnsson and Ólafsson, 2004; 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. The moment magnitude of the earthquake was 6.3 according to the Global Centroid Moment Tensor (CMT) database, as well as the United States Geological Survey (USGS) and the Instituto Nazionale di Geofisica e Vulcanologia (INGV). The earthquake can be characterised as a shallow crustal earthquake on a north-south trending right-lateral strike-slip 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). Fig. 1.4 displays the location of the epicentres and approximate surface traces of the causative faults along with an overview on the location of strong-motion stations in the near-fault area (Sigbjörnsson and Ólafsson 2004). The peak accelerations recorded during these three main events are listed in Table 1.1.

Table 1.1. Peak accelerations recorded at the Town Hall in 2000 and 2008.

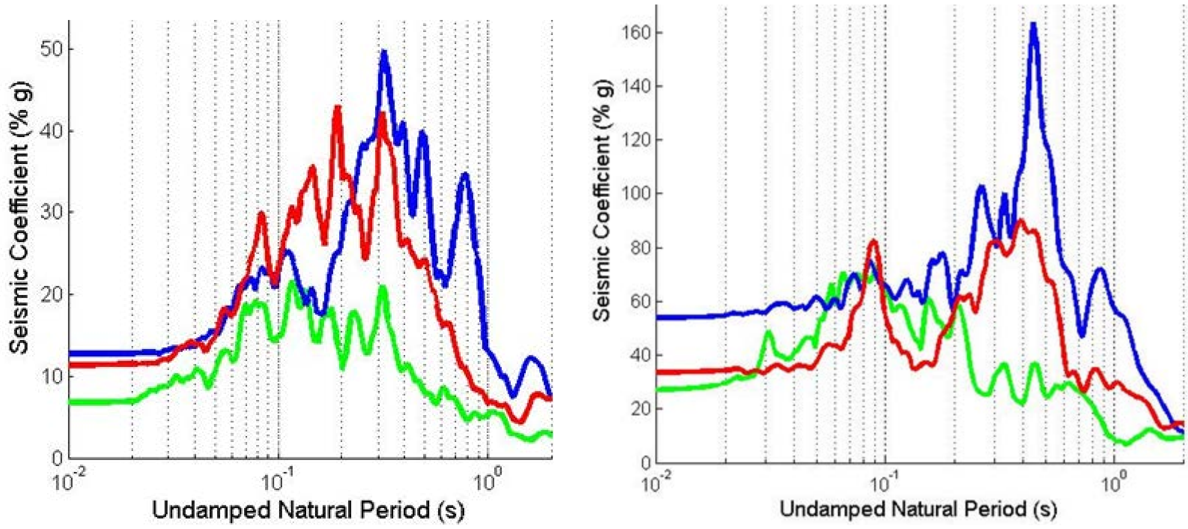
Date of event	Magnitude	Distance from site (km)	Peak ground acceleration (%g)			Peak response acceleration (%g)		
			Vert	N-S	E-W	W:N-S	C:W-E	E:S-N
June 17, 2000	6.5	32	2.9	7.6	5.5	14.6	12.13	15.84
June 21, 2000	6.5	15	5.0	12.7	11.2	30.2	21.4	29.2
May 29, 2008	6.3	8	26.6	31.6	26.7	74.6	47.3	68.2

2. THE STRUCTURAL RESPONSE DURING THE TWO MAIN EVENTS

When comparing the recorded surface and response accelerations from the two main earthquakes, i.e the one in June 2000 and May 2008, a clear difference in the frequency content and thereby the characteristics of the response is noticed. This will be demonstrated further in the following.

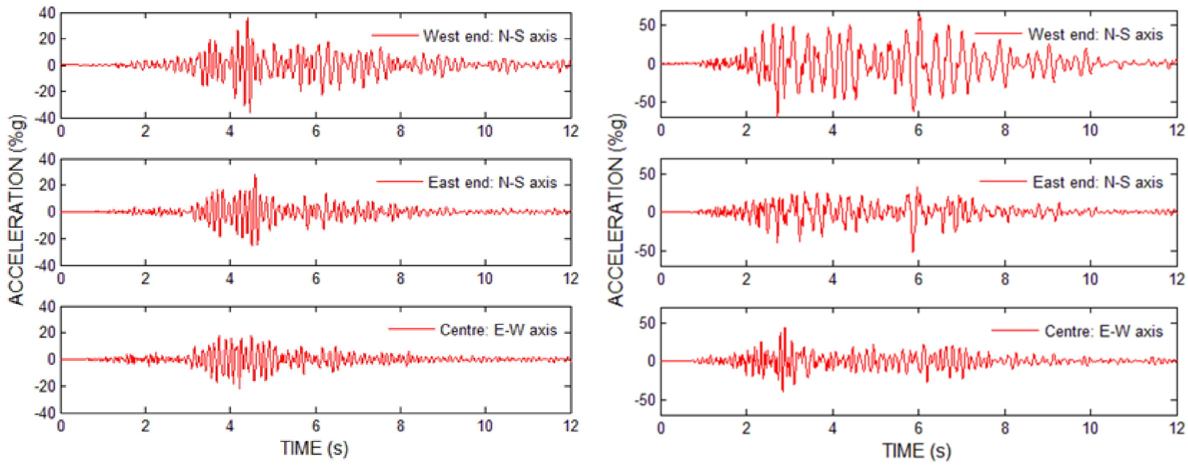
The response spectra are computed from the recorded accelerations in the basement and shown in Fig. 2.1. The horizontal components show a significant response from about 0.1 s up to 0.8 s, for both events. However, for the 2008 event a strong response peak is observed at 0.4-0.5 s. This response peak is likely to be 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.

Figure 2.2 shows the relative acceleration at the third floor. The relative accelerations are calculated by subtracting the ground acceleration from the recorded absolute acceleration at third floor. It can be observed that the acceleration is in both cases higher in the N-S direction than in the E-W direction, which is to be expected considering the geometry of the building. It is also seen that the accelerations are always larger at the west gable than the east gable, indicating considerable torsional effects in the response. This is to be expected as the east side of the building has considerably more stiffness in the N-S direction than the west-side of the building. It was one of the objectives of the rehabilitation project to reduce this difference, but it is clear that there is still considerable eccentricity in the N-S stiffness of the structure.



(a) (b)

Figure 2.1. 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.



(a) (b)

Figure 2.2. Time-series of acceleration response on the third floor. (a) The event on June 21. 2000; (b) The event on May 29. 2008.

Figure 2.3 reveals the frequency content of the relative accelerations at the third floor during these two main events. Comparing the E-W components, shows more or less the same spectral peaks but they are shifted about 1 Hz towards lower frequency values for the 2008 event. Comparing the N-S components, 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 soil effect dominates the response from the earthquake in 2008. The difference in the response between the east and west gable is also further revealed in the spectral analysis.

A view of the identified modes of vibration is given in Table 2.1. The 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. As can be seen the shift in frequency for comparable mode shapes is real, and several modes are found in the 2008 event that are not seen in the 2000 event. The development of natural frequencies and critical damping ratios for the building in recorded earthquakes between 1999 and 2003 were studied in Snæbjörnsson et al. (2004).

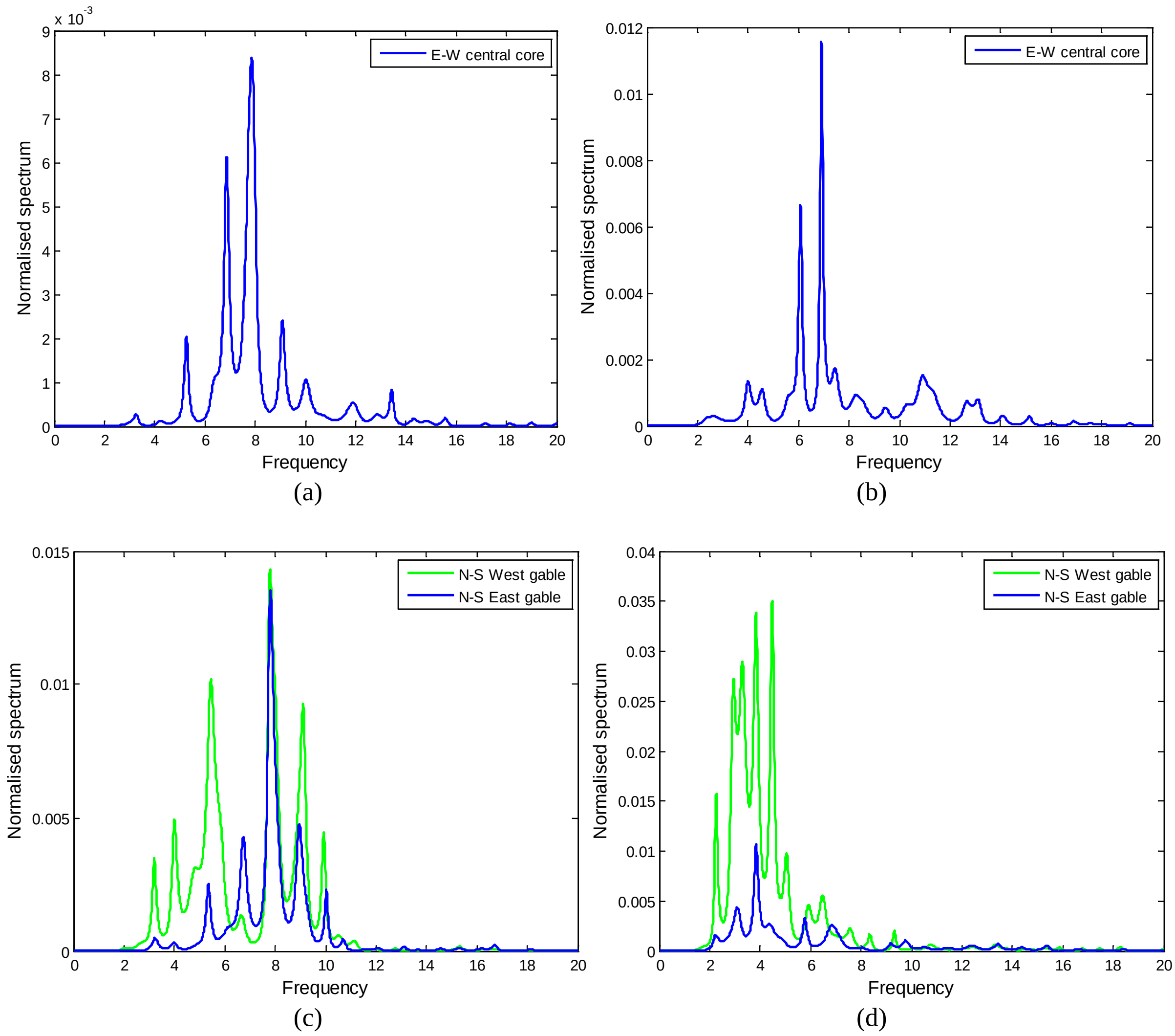


Figure 2.3. Power spectral densities of relative acceleration response on the third floor. (a) The E-W component from the event on June 21. 2000; (b) The E-W component from the event on May 29. 2008; (c) The N-S components from the event on June 21. 2000; (d) The N-S components from the event on May 29. 2008. The blue line in (c) and (d) represents the motion of the west gable but the green line the motion of the east gable.

Table 2.1. Peak accelerations recorded at the Town Hall in 2000 and 2008.

Mode no.	2000 Description	2000 Natural frequency (Hz)	2000 Critical damping ratio	2008 Description	2008 Natural frequency (Hz)	2008 Critical damping ratio
1				N-S	2.2	5.5
2	N-S & rotation	5.4	1.3	N-S & rotation	3.8	2.9
3	E-W & rotation	7.6	1.1	E-W & rotation	5.5	2.4
4	E-W & rotation	8.1	0.6	E-W & rotation	6.9	0.5
5	E-W & rotation	10.7	0.7	E-W & rotation	8.2	0.9
6	N-S	15.2	1.3	N-S	10.3	0.7
7	N-S	16.8	0.4	N-S	11.8	0.7
8				N-S	15.2	0.9
				E-W & rotation	17.9	3.6

Figure 2.4 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 were 512 m/s and 960 m/s in 2000, but 128 m/s and 256 m/s in 2008, assuming a 16 m thick rock-soil layer underneath the building (see Fig. 3.1).

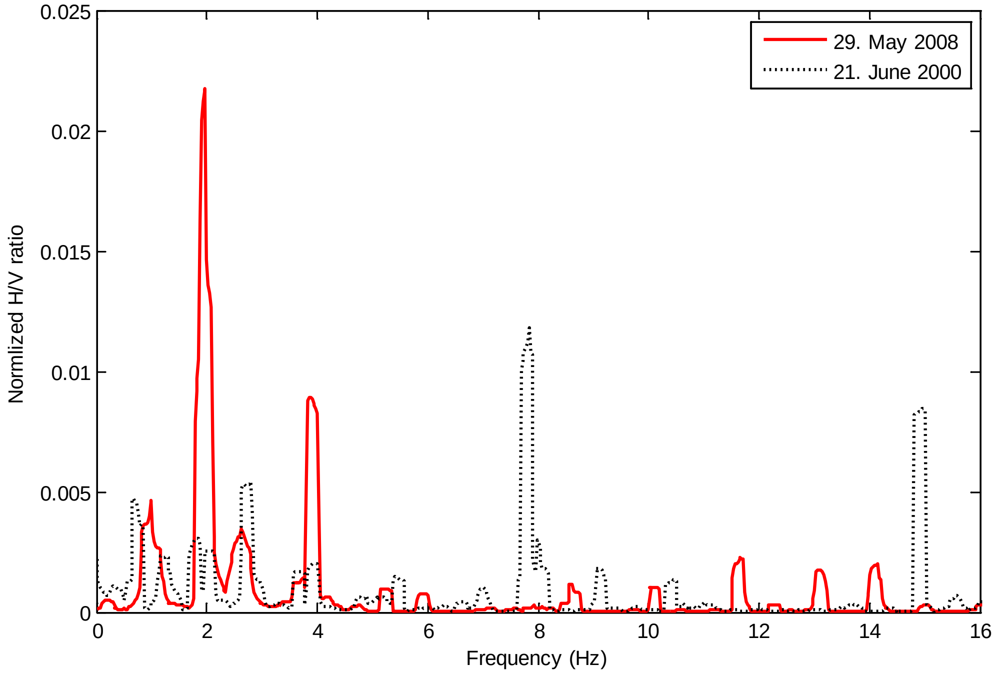


Figure 2.4. The normalised H/V-ratio as a function of frequency for the two earthquakes in 2000 and 2008.

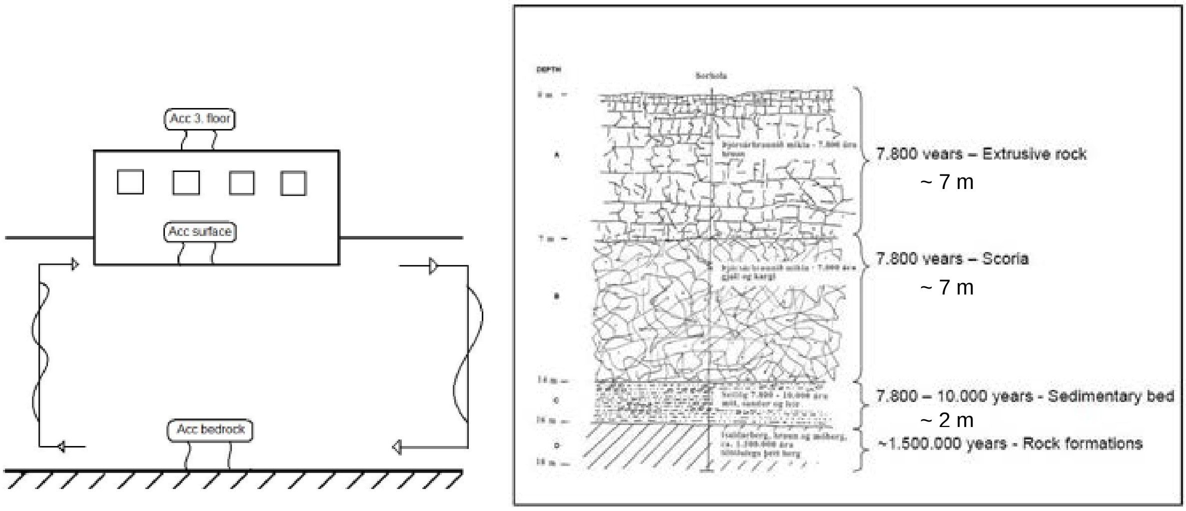
3. THE SOIL-STRUCTURE INTERACTION

The Town Hall of Selfoss is located on top of several soil and rock layers. A hypothetical rock-soil profile from the building site is shown on Fig. 3.1. The rock-soil profile is assumed to be constructed of at least three different layers of different material, although the H/V-ratio may indicate a more complex composition. The two upper rock layers were made of volcanic rock and are relatively stiff, while the bottom layer is a soft sedimentary layer consisting of humus, sand and clay. This layer originates from when the last glacial period when this layer was the seabed bottom. After the ice age the land rose and volcanic activity produced several layers of lava on top of this layer of soft soil, which is believed to have a significant effect on the dynamic behavior of the structure.

It is proposed to construct a finite element based model that can recreate the dynamic response of the structure and the soil. The model will include both the properties of the ground and the structure. Fig. 3.1 shows a schematic drawing of the soil-structure interaction system. To be able to recreate the dynamic response, the bedrock acceleration is desired as an excitation for the dynamic system. The goal is to obtain a dynamic response similar to the recorded one, for each major event. The objective is to gain insight into the observed difference in structural behaviour between the two main events, which is believed to be greatly influenced by the local site effects.

3.1. Site response and finite element analyses

The ground stiffness is very much dependent on the shear strength of the soil material, which can be calculated based on the shear wave velocity squared multiplied with density. Therefore, any changes in the shear wave velocity, causes large change in the shear strength of the material (Bessason and Kaynia 2002). Fig. 3.2 shows the proposed shear wave velocity profile for the site.



(a) (b)

Figure 3.1. (a) A schematic drawing of the structure-soil interaction system. (b) A section showing a hypothetical rock-soil profile from the building site (Stray 2010).

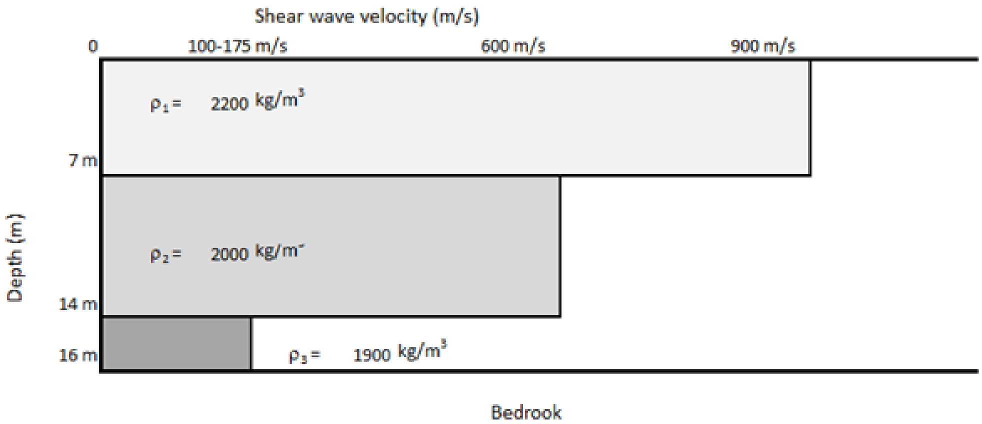


Figure 3.2. (a) A schematic drawing of a section showing a hypothetical shear wave velocity profile from the building site (Stray 2010).

A computer program called EERA (Equivalent-linear Earthquake site Response Analyses) has been developed at the University of Southern California (Bardet et al. 2000). This program makes it possible to estimate the effect of the local site conditions.

The idea is to use the site response analyses (e.g., strain dependent shear modulus and damping curves) to predict the effects of the proposed soil profile on the site response during an earthquake to simulate the bedrock acceleration based on the recorded acceleration in the basement. The approximated bedrock motion can then be used as an input excitation for the model to reproduce the response at the basement and the third floor of the building. Using the shear-wave velocity profile shown in Fig. 3.2 and the respective basement acceleration time-series as input in the EERA program resulted in the bedrock acceleration time-series displayed in Fig. 3.3 (a) and (b). The corresponding Fourier amplitude spectra are shown in Fig. 3.3 (c) and (d).

The bedrock acceleration time-series were applied to a Ruaumoko 3D-finite element model (Carr 2009) (see Fig. 3.4) and response acceleration time-series evaluated for the third floor. The calculated time series were not a perfect match to the recorded ones, but they showed similar differences in characteristics for the two events studied as the recorded time-series. Further development and improvement of the model and the simulation procedure is on-going.

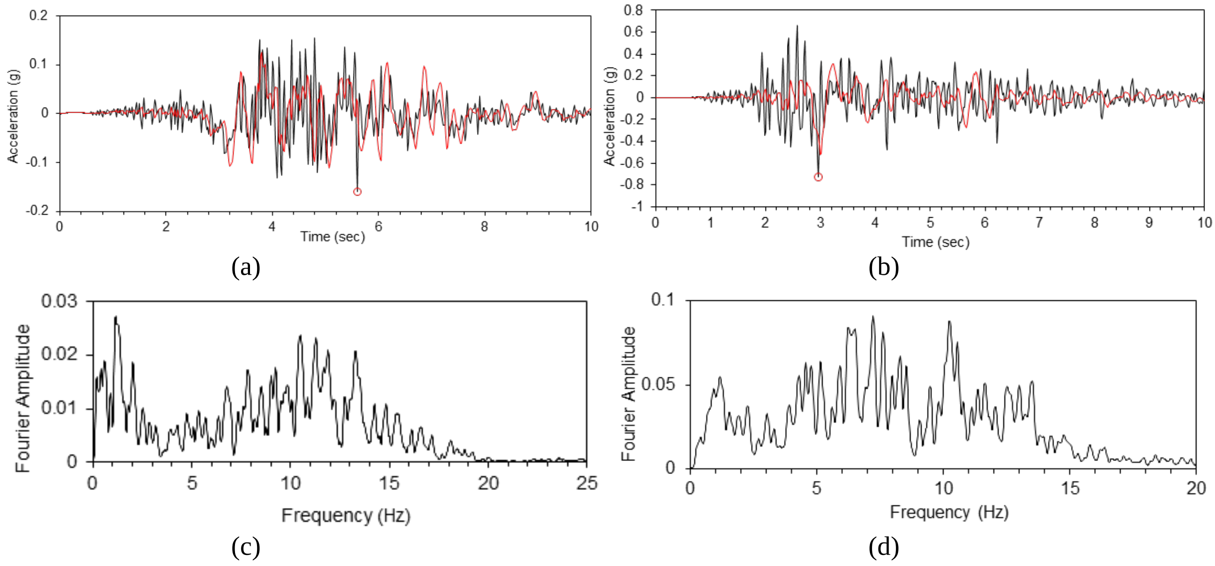


Figure 3.3. N-S acceleration component, the red line is the recorded accelerations from the basement and the black line is calculated accelerations on the bedrock. Fourier spectra of the calculated N-S bedrock component.

Inelastic analyses were also performed in order to estimate the capacity of the structural system. Inelastic analysis indicated that 39 members experienced post-yield behaviour during the earthquake in 2008. The highest ductility demand occurs in the first floor columns, with a ductility ratio above 6. A structure of this type and age would be likely to suffer considerable damage at ductility levels above 3 to 4. There are however, no visible indications of such damages. Thus, the inelastic model seems to overestimate the inelastic behaviour. Nevertheless, the model gives valuable indication of where to expect yielding.

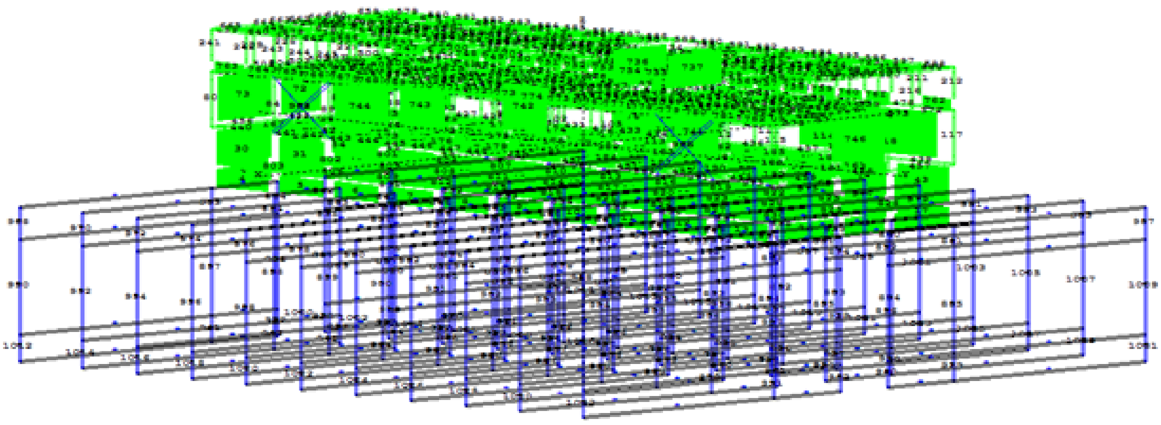


Figure 3.4. The Ruaumoko 3D -finite element model. The blue squares indicate the soil springs. The concrete beam- and shells elements representing the building are green and the steel beam x-bracing elements in the two frontal windows are blue.

4. DISCUSSION AND FINAL REMARKS

To explain the response from the two recorded earthquakes in 2000 and 2008, is seems clear that the behaviour of the different soil-rock layers underneath the building needs to be understood and included in the structural modelling. Especially, the soft sedimentary layer that likely 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 mapping of such behaviour is therefore difficult to model with a quasi-linearly elastic model. However, this has been handled, by “manually” incorporating the effect from the varying strain level in the soil-rock layers. The resulting simulation does not give a perfect match to the recorded response, but the observed differences in the earthquake response characteristics between the two events are similar. Therefore the analysis confirms the influence of the soil-rock layers beneath the building on the building response.

The model used, has recreated a similar response to the response recorded in the Town Hall of Selfoss. It shows that the amount of strains in the sedimentary layer to a large degree controls the frequencies of response. There are also indications that the large damping effects created by the soft layer reduce the amplitude of the ground accelerations, as well as lowering the frequency content of the excitation.

This soil-structure interaction has most likely been beneficial for the earthquake response of the building, during the earthquake in May 2008, and perhaps limited the risk of yielding predicted by the inelastic analysis.

However, it is likely that a considerable part of the structures limited plastic capacity was utilized during the earthquake in 2008. Therefore, another earthquake of the same magnitude is likely to cause more extensive damage to the structure.

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 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 soil-structure interaction.

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