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ANALYSING AND MODELLING RECORDED EARTHQUAKE INDUCED STRUCTURAL RESPONSE

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SUMMARY

Earthquake induced acceleration data has been systematically collected in two reinforced cast-in-place concrete buildings over periods of 14 and 5 years respectively and the database from the two buildings presently contains 870 records from 107 events ranging from magnitude 2½ to 6½ with acceleration amplitudes of up to 34% g. This data is used to examine structural behaviour and system parameters and their dependence on excitation conditions. The system identification based on the sampled data serves both as a baseline for damage identification as well as calibration for further structural modelling of the buildings. Visible damage is slight, but the system parameters reflect structural changes. There are clear indications that the system parameters are amplitude dependent, even for relatively small response amplitudes. Inelastic dynamic analyses were also carried out on one of the structures to investigate the reliability of the analytical models and to estimate the capacity of the building to sustain future earthquake events. The results show that the model does represent the structure and although it came through the recent earthquakes with no damage the structure has limited capacity to withstand any event that would be larger than those experienced to date.

INTRODUCTION

The paper evolves around two reinforced cast-in-place concrete buildings. One is a 14-story office building and the other is a 3-story Town Hall building. Earthquake induced acceleration data has been systematically collected in the buildings over periods of 14 and 5 years respectively and the database from the two buildings presently contains 870 records from 107 events ranging from magnitude 2½ to 6½ with acceleration amplitudes of up to 34% g. The purpose of the paper is to utilise this data to examine structural behaviour and system parameters, their variability and the dependence on excitation conditions. The instrumentation of the buildings is described and the recorded structural response data presented. System identification analyses of the buildings are carried out applying a previously verified parametric method to the recorded data [9]. The work also includes inelastic dynamic analyses of the building, using the Ruaumoko software [2].

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THE BUILDINGS, INSTRUMENTATION AND DATA ACQUISITION

The Commerce Building in Reykjavik

The structure

The Commerce Building in Reykjavik is 14 stories high (45 m) office building. It is a reinforced cast-in-place concrete structure, basically composed of shear walls and slabs. The geometry of the building is rather complex, as the floor plans vary, changing vertically, as shown in figure 1 and 2. The 14^{th} floor plan is about 8 m wide and 18 m long. The alignment of the building is such that the translational modes of vibrations are approximately in the ESE-WNW and NNE-SSW directions. Some earlier investigations of the building and its response to wind and earthquake excitation have been reported [8,10].

Figure 1. The Commerce Building in Reykjavik, view from Northwest.

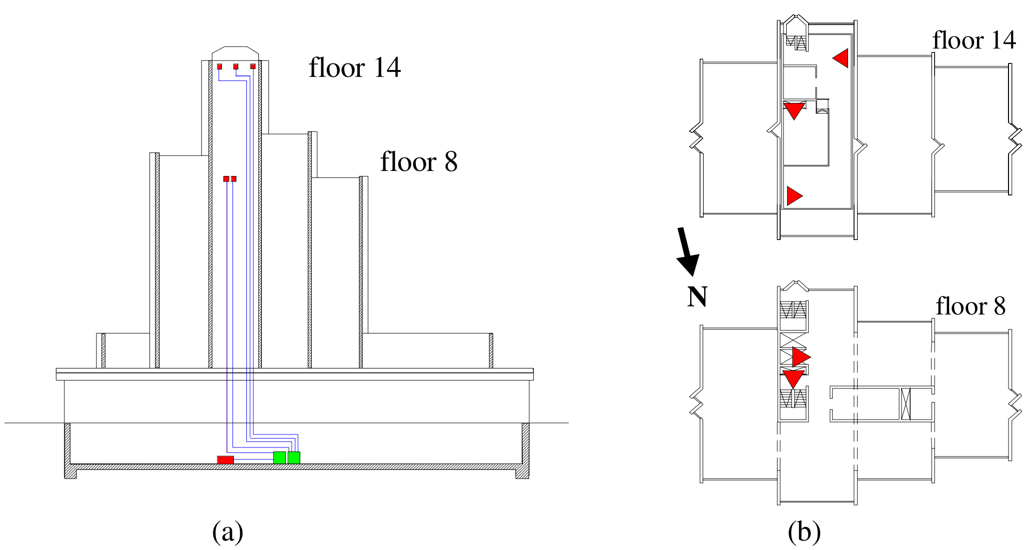


Figure 2. The instrumentation arrangement: (a) Vertical section, and (b) Floor plans of the building, showing the location of instrumentation (▲ uni-axial accelerometer, ■ tri-axial accelerometer and ■ data acquisition).

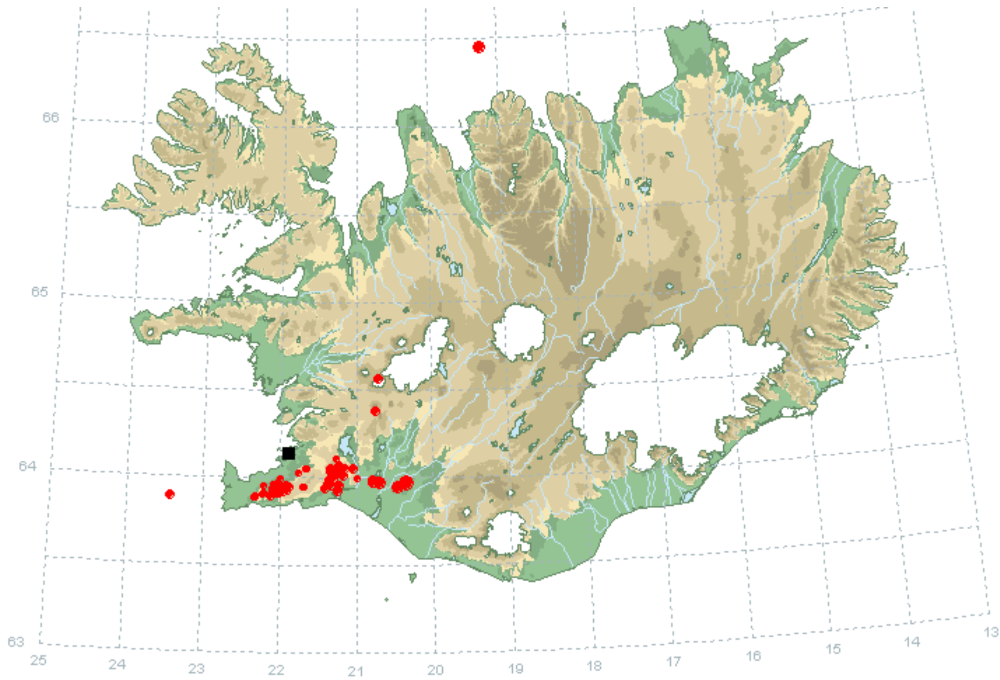


Figure 3. A map showing Iceland and the location of the Commerce building (the black square) and the epicentre of the recorded EQ events (red dots).

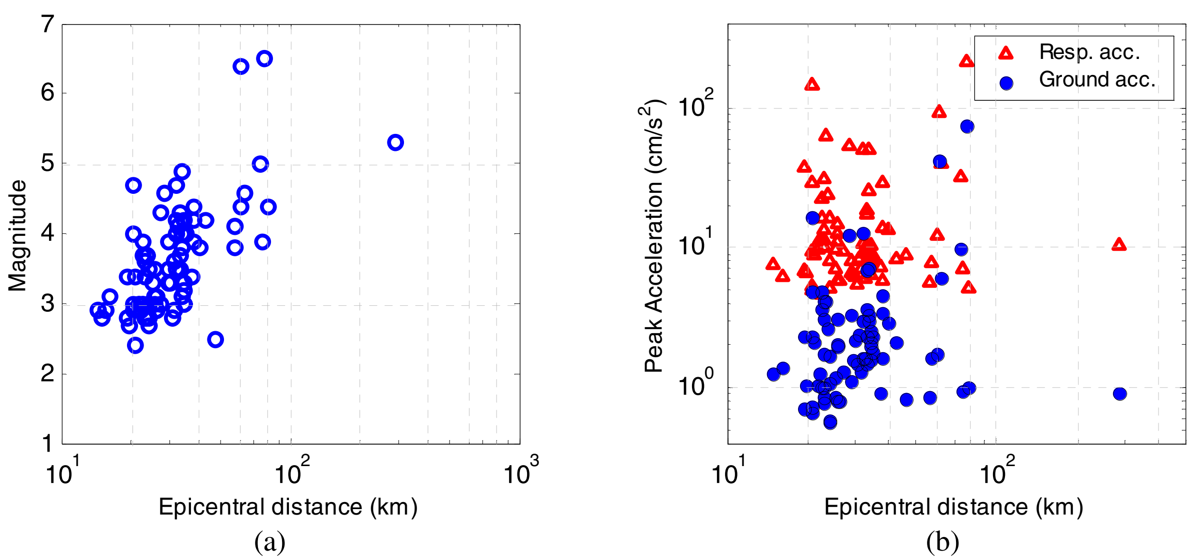


Figure 4. (a) Magnitude as a function of epicentral distance to the Commerce building and (b) the corresponding PGA values and peak response acceleration as a function of epicentral distance.

Instrumentation

The building was instrumented in January 1989. The instrumentation is located at three levels (see figure 2): the basement, the 8th floor and the 14th floor. A tri-axial accelerometer (Kinemetrics FBA-23) is located in the basement, measuring the three components of base (ground) acceleration. On the 8th floor two uni-axial accelerometers (Kinemetrics FBA-11) are located measuring the two horizontal components of the response. On the 14th floor (the top floor) three uni-axial accelerometers are located, one measuring motion in the N-S direction and two measuring in opposite corners (i.e. N-E and S-W) measuring motion in the E-W direction. This makes it possible to detect torsional effects on the 14th floor (see figure 2). The eight sensors (channels) are connected to two interconnected data acquisition units (Kinemetrics SSA-1). The sampling rate is 200 Hz. The data acquisition starts automatically when the acceleration on the 14th floor exceeds a specific trigger level, which is at present 0.5% g.

Earthquake data and response

The system has recorded 78 earthquakes at all eight channels. The recorded earthquakes range in magnitude between 2½ and 6½. Figure 3 and 4 gives an overview on the recorded earthquakes. The geographic location of the EQ epicentre is shown on figure 3 in relation to the location of the building. Figure 4 shows the earthquake magnitude and maximum horizontal ground and response acceleration as a function of epicentral distance. The acceleration values shown are based on time-series, which have been corrected and filtered to account for the characteristics of the sensors, noise, zero offset and drift. It should be noted, that earthquakes often occur in sequence, in which case several events are recorded within a time frame of few days or less.

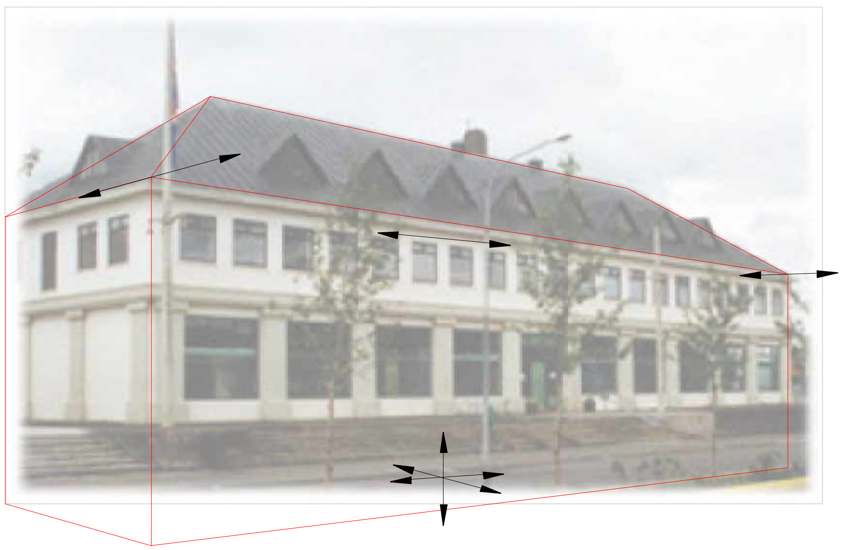


Figure 6. The Town Hall at Selfoss, view from Northeast. The black arrows indicate location and directionality of the accelerometers.

The Town Hall in Selfoss

The structure

The Town Hall at Selfoss (see figure 6) is an office 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. Its rectangular plan is about 38 m long and 10 m wide. It is a reinforced cast-in-place concrete structure, composed of outer shear walls and a shear core around the stairway and elevator shaft, but with two rows of interior concrete columns and interconnecting floor beams to carry the slabs (see figure 7). The orientation of the building is such that the length of the building approximately aligned along an ESE-WNW axis. This building was built in the 1940s in wartime conditions and the available drawings 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 the strength of the building. Also two steel cross-braces were in inserted into the window frames at ground level on the north side.

Instrumentation
The building was instrumented in January 1999. The instrumentation is located at two levels (see figure 5 and 6): the basement, and top storey (the 3rd floor if the ground floor is no. 1). A Kinemetrics-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 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).

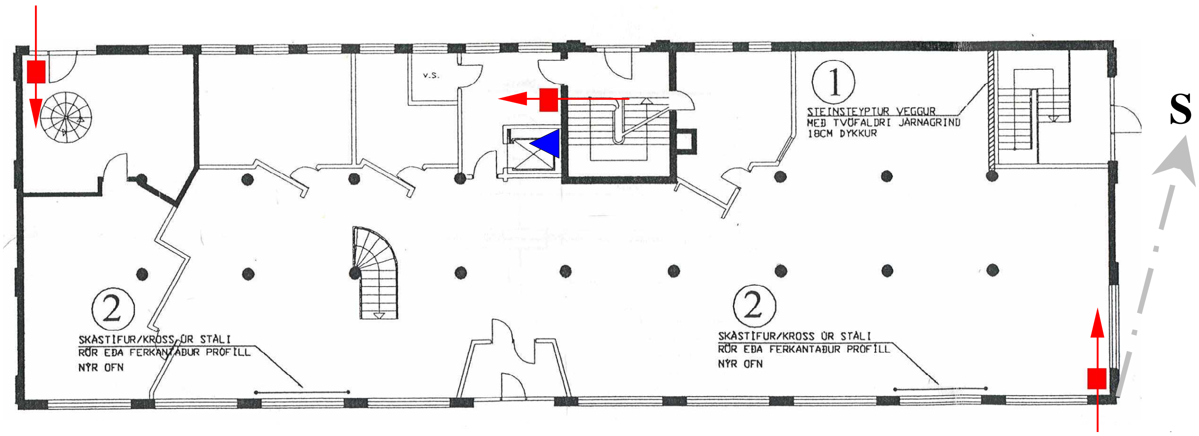


Figure 7. The Town Hall at Selfoss, plan view of the ground floor. The location of uni-axial (squares) and tri-axial (triangle) accelerometers within the building plan as well as their positive direction is shown. The location of steel cross-braces (2) and reinforced concrete wall (1) installed in spring 2000 for retrofitting purposes can also be seen.

Earthquake data and response
The system has recorded 40 earthquakes at all six channels. The recorded earthquakes range in magnitude between 2 and 6½. Figure 8 and 9 give an overview on the recorded earthquakes. The geographic location of the EQ epicentre is shown on figure8 in relation to the location of the building. Figure 9 shows the earthquake magnitude and maximum horizontal ground and response acceleration as a function of epicentral distance.

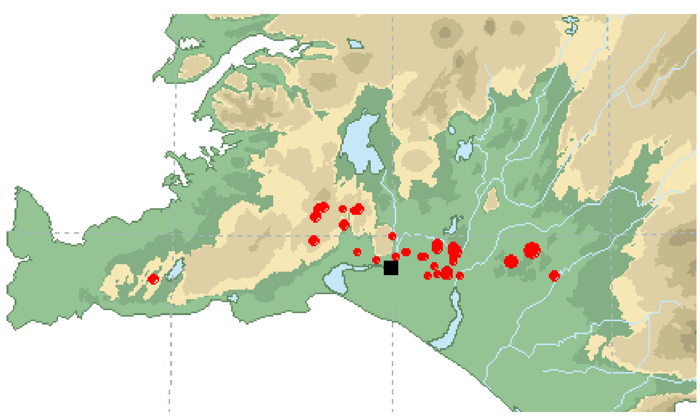
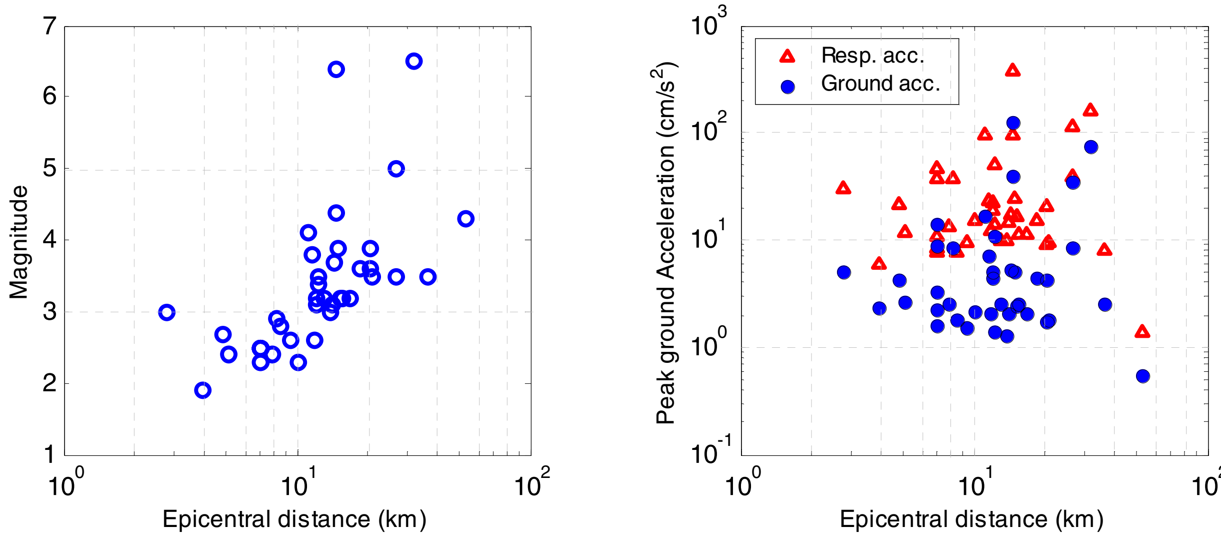


Figure 8. A map showing Iceland and the location of the Selfoss Town Hall (the black square) and the epicentre of recorded EQ events (red dots).

Since recordings were started in the building a data sample of 40 events has been gathered. Five events are from 1999, after renovation and retrofitting of the basement and the first floor but before retrofitting of the ground floor. The ground floor was retrofitted in the spring of 2000 by adding a reinforced concrete shear wall at the west end of the building and diagonal braces were added to the windows in the North face (see figure 7). After this work was done, 30 events were recorded in the period of June and July 2000, of which two events were of magnitude 6.5. Finally, 5 events were recorded in the period 2001-2004, one in February 2001 while work was still on going on the top floor of the building, and 4 after all work on the building was completed. The top floor was strengthened with a 5 cm reinforced concrete layer on top of the old floor after removing an old 2.5 cm concrete levelling layer. The top floor was then partitioned with double gips walls. This work has added significant mass to the top floor in addition to the stiffening effect on the floor. Therefore, in many ways the database is composed of subsets for at least 3 somewhat different structural conditions of the same building. This fact characterises the data in many ways as will be demonstrated in the following.



(a) (b)

Figure 9. (a) Magnitude as a function of epicentral distance to the Selfoss Town Hall and (b) the corresponding PGA values and peak response acceleration as a function of epicentral distance.

SYSTEM IDENTIFICATION

Methodology

The recorded acceleration data was used for system identification of the building. The aim was, to estimate the natural frequencies and critical damping ratios for the main modes of vibration. Furthermore to examine the system parameters, their variability and dependence on excitation conditions. Finally, to check if any changes in structural behaviour could be observed throughout the observation period, which included two 6.5 magnitude events that induced acceleration levels of 22 and 39% in the respective buildings in addition to the renovations of Selfoss Town Hall.

For this purpose a parametric method was applied, using a state-space model identification (SMI) toolbox [5]. After proper pre-processing of the data, only two parameters are required for an output-error model identification problem: the upper bound on the expected order of the system and the true order of the system to be extracted from the data. The upper bound on the expected system order can be determined trough the Akaike criterion of an AR-SISO model of increasing order [12]. A SISO approach, where a single input (ground acceleration series) and a single output (response acceleration series) are modelled, was found to give more consistent results than any multi input, multi output (MIMO) combination. After establishing the natural frequencies of the structure a sharp band-pass FIR filter was applied around each natural frequency band to improve the damping estimation. A more general discussion on various system identification techniques may be found in [1].

The model used, is a linear, time-invariant system, which is not strictly appropriate for strong motion records which are non-stationary and the vibrating structures are likely to show time-variant and even non-linear characteristics [6]. However, non-linear identification models are difficult to use because of complexity and identifiability problems. On the other hand, it is possible to evaluate the dynamic characteristics of non-linear systems as equivalent linear, time-invariant systems for relatively short-time segments of each record. This has been referred to as time-variant linear models [13]. Since strongly non-linear behaviour is not to be expected for the two buildings studied this was considered an acceptable methodology. The outlined procedure was therefore applied to the acceleration data at hand and the results are presented in the following.

Results for the Commerce building

The records from the Commerce building were analysed in a sequentially, in order to establish a temporally organised database of system parameters. Figure 10 gives an overview on the modal frequencies. There are three modes strongly excited during vibration in the ESE-WNW direction, just below 2 Hz, just above 4 Hz and at 8 Hz. For vibration in the NNE-SSW direction, two modes dominate the response, at just above 2 Hz and at 8 Hz. The irregular shape of the building along with the asymmetric location of the shear-core around elevators and staircase in the centre tower makes all modes influenced by torsion.

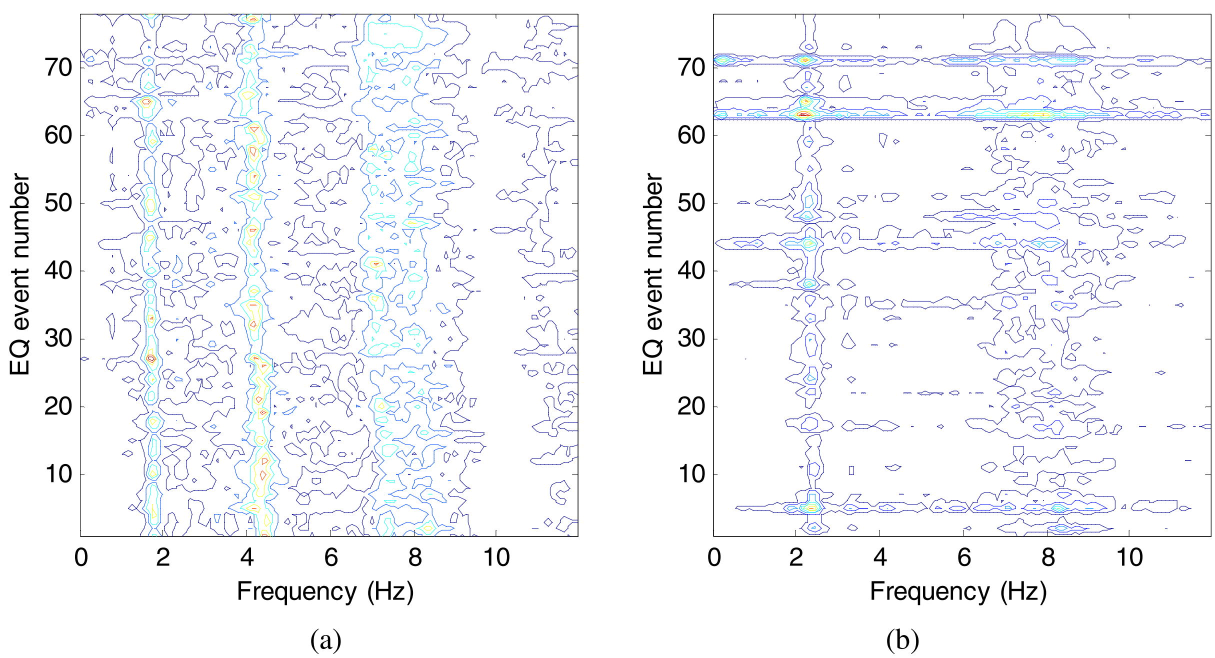


Figure 10. Contour graphs of the Fourier spectrum for the 78 events analysed. (a) For vibration in the ESE-WNW direction. (b) For vibration in the NNE-SSW direction.

Figure 11 gives an example of the evolution and variability in the system parameters for the second ESE-WNW mode in the 78 events recorded during a 14 years observation period. The 'time-series' shown are a smoothed version of the estimated parameters to give a visually clearer picture. The damping is seen to vary from 0.5% for low amplitude data up to 1.5% and higher for the high amplitude data such as the two 6.5 magnitude events. The frequency is also seen to vary considerably, or from 4.4 Hz down to 4 Hz. Also, there are conceivably indications of a long-term reduction in the natural frequency. Looking at the large events, marked by the vertical grid lines, it is seen that although the natural frequency seems to be re-established at 4.2-4.3 Hz, it takes the structure a considerable time to reach that point. Especially considering that 8 smaller events are recorded during the 4 days between 17 and 21 of June. Another 6 events are recorded in the period after that, in July and November 2000 and then two in 2001. This indicates that the 'rebound' of a structure undergoing excitation, seemingly within the linear range of response, is not instantaneous but a process that may take few days, weeks or longer.

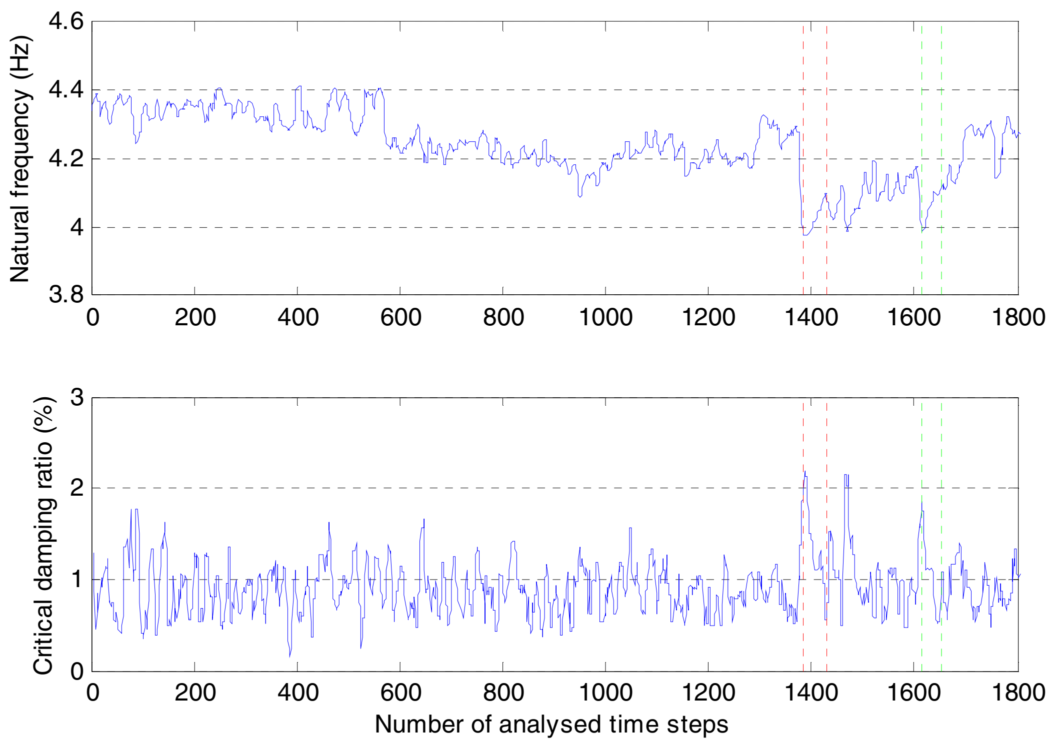


Figure 11. Natural frequency and critical damping ratio as a function the number of analysed time steps. The 6.5 magnitude events of June 17 and June 21 in 2000 are indicated within the dotted stripes, red and green, respectively.

Individual series were also studied, in particular the 6.5 magnitude earthquake from June 17 in 2000 [14]. The objective was to establish, if and how the system parameters change within each event. What makes the June 17 event special is that the recordings combine three different earthquakes. The first one was of magnitude 6.5 and located 77 km East of the building in the South Icelandic Seismic Zone (SISZ). The second earthquake, M_w~5, occurred 26 s after the SISZ event, about 23 km SSE of the building. The third earthquake occurred 4 s later, 12 km to the west or about 22 km South of the building, near the eastern shore of Lake Kleifarvatn. Timing suggests that these earthquakes were triggered dynamically by shear waves from the SISZ event travelling at a velocity of 2.5 km/s. This phenomena, is displayed by the time-series in figure 12, which shows the observed E-W response of the NE corner on the 14th floor, recorded during the earthquake. The E-W direction lies across the weaker axis of the structural system, therefore the building response (acceleration) is always strongest for that direction. The frequency content of the acceleration changes throughout the earthquake and the response has three separate bursts of peak acceleration with centres at around 13 s, 22 s and 27 s. The high frequency and high amplitude acceleration burst at 22 s can be traced to the second event with epicentre 23 km SSE of the building. Few seconds later there is a lower frequency acceleration burst that can be traced to the epicentre 22 km South of the building.

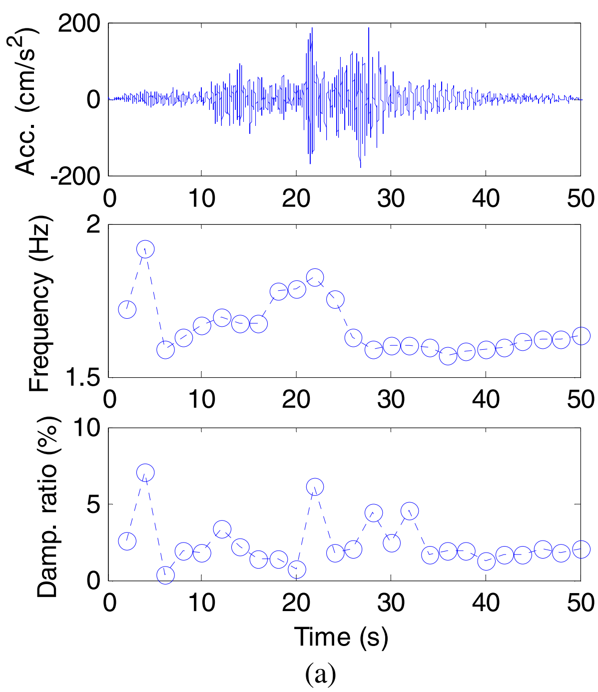


Figure 12. The acceleration time series from the June 17 main event along with the frequency- and damping sequence from the same event for (a) the first and (b) the second mode of E-W vibration.

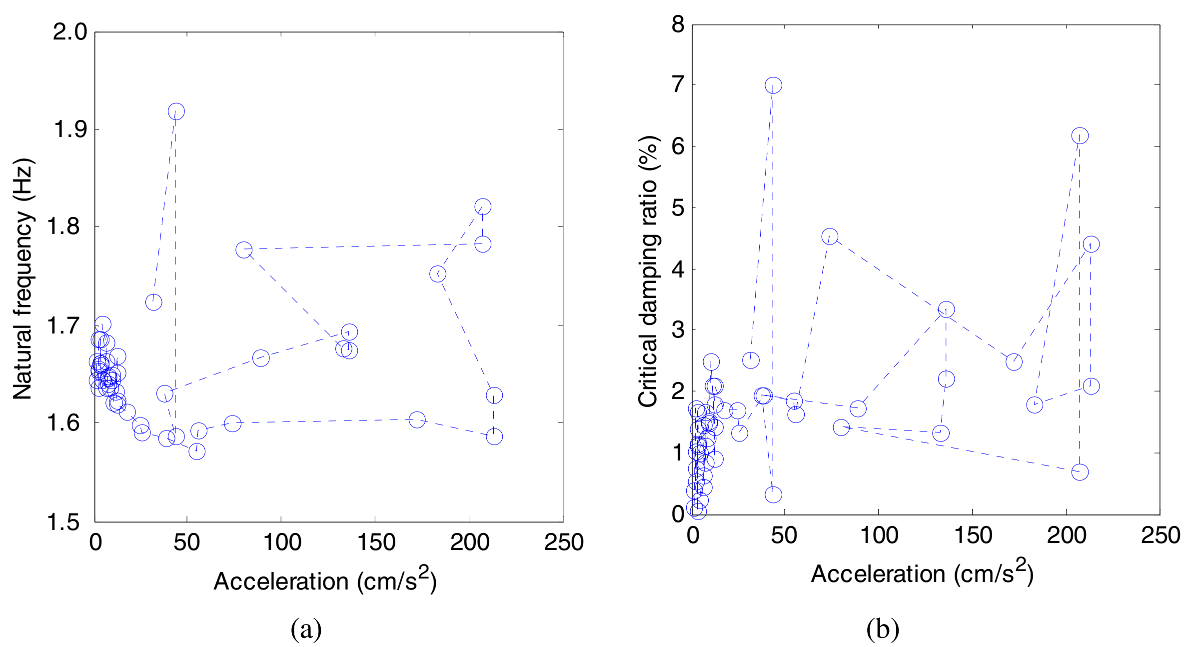


Figure 13. Natural frequency (a) and critical damping ratio (b) for the first mode of E-W vibration as a function of acceleration amplitude during the June 17 main event.

The time-series of natural frequency show partly how the frequency changes with amplitude but also how that the participation of the modes varies throughout the earthquake. The second mode is clearly increasingly active throughout the peak of the earthquake and then its participation reduces and the frequency increases towards it starting value. On the other hand, the first mode starts out strong, then its participation reduces while the first arriving waves of higher frequency content have passed, at which time its participation increases again and dominates the decay of the motion. The estimated damping is bit chaotic, but is in general seen to increase as the amplitude increases. The variation is seen to be greater for the first mode, where the damping repeatedly exceeds 5% of critical, while the damping of the second mode is more stable around 2%. This is not unreasonable as damping is generally related to velocity or displacement controlled processes [11], and both velocity and displacement are considerably larger for the first mode than the second.

Plotting natural frequency and critical damping ratio versus the peak acceleration amplitude for each time-step within the event, gives another view on how the SI-parameters are changing during the earthquake. This is done in figure 13 for the first E-W mode of vibration. The result is a trace that in some ways resembles a displacement trace during a non-linear response. This underlines the fact that it is not just the overall amplitude of motion that affects the SI-parameters, but rather the relative contribution of the modes of vibration at each time step.

Results for Selfoss Town Hall

The records from the Selfoss Town Hall building were analysed in a similar fashion as the Commerce Building, but the two buildings represent two different cases. Figure 14 gives an overview on the primary modal frequencies. It can be seen that it is difficult to identify clearly the modes of vibration in the NNE-SSW direction. Something is going on at about 6 Hz and then again between 8 and 10 Hz. For vibration in the ESE-WNW direction there is basically one mode at about 8 Hz that dominates the response. The asymmetric location of the shear-core around the staircase along with irregular distribution of shear walls along the NNE-SSW axis and large windows on the North side at the ground floor level makes all the modes linked with torsion. In fact frequency response is detected at similar frequencies for perpendicular acceleration signals and it is difficult to distinguish between or decouple the translational motion and the torsional motion. Another observation, when looking at figure 14, is that the central frequencies of the active frequency ranges representing the modes of N-S vibration are not constant but slightly increasing throughout the observation period. This is demonstrated with the double dotted lines drawn across the contour plots.

After some investigation it was determined that there were basically three active modes of vibration in the ESE-WNW direction, i.e. at 6 Hz, 8 Hz and 10 Hz. The randomness in the frequency response depicted in figure 14 is most likely caused by the fact that the retrofitting elements are not fully integrated into the structural system of the building and their resistance, or activeness, can therefore be expected to vary considerably depending on the amplitude and character of both excitation and response. The slight increase in frequency throughout the observation period is similarly likely to be caused by a gradual unification between the original structure and the retrofitting elements. Any small gaps, due to shrinkage in the later inserted wall, will result in a not fully functional element at small displacements associated with lesser earthquakes. Repeated grinding of the walls during earthquakes may slowly fill the gaps. Secondly, the retrofitted walls will stiffen with age as the concrete continues to cure.

Figure 15 shows the development and changes in the system parameters for a N-S mode of vibration at 6 Hz and an E-W mode of vibration at 8.5 Hz during a 4 years period with 40 recorded events. The development of these two modes is somewhat different. The frequency for the N-S mode (figure 15a) is increasing throughout the observation period, whereas the E-W mode (figure15b) has a more constant Frequency. The damping for N-S mode seems to be decreasing throughout the summer of 2000 until it stabilises at about 2%, whereas the damping for the E-W mode varies around a relatively constant mean value. The damping values are in general agreement with the findings of [4]. As mentioned before, the trends seen in the N-S mode are likely caused by a gradual locking between the original structure and the retrofitting elements. The majority of the retrofitting wall elements have a N-S orientation and their primary purpose is to reduce rotational effects and translational response along the N-S axis. These additional elements should therefore not have significant effect on the E-W modes of vibration, which is supported by the data.

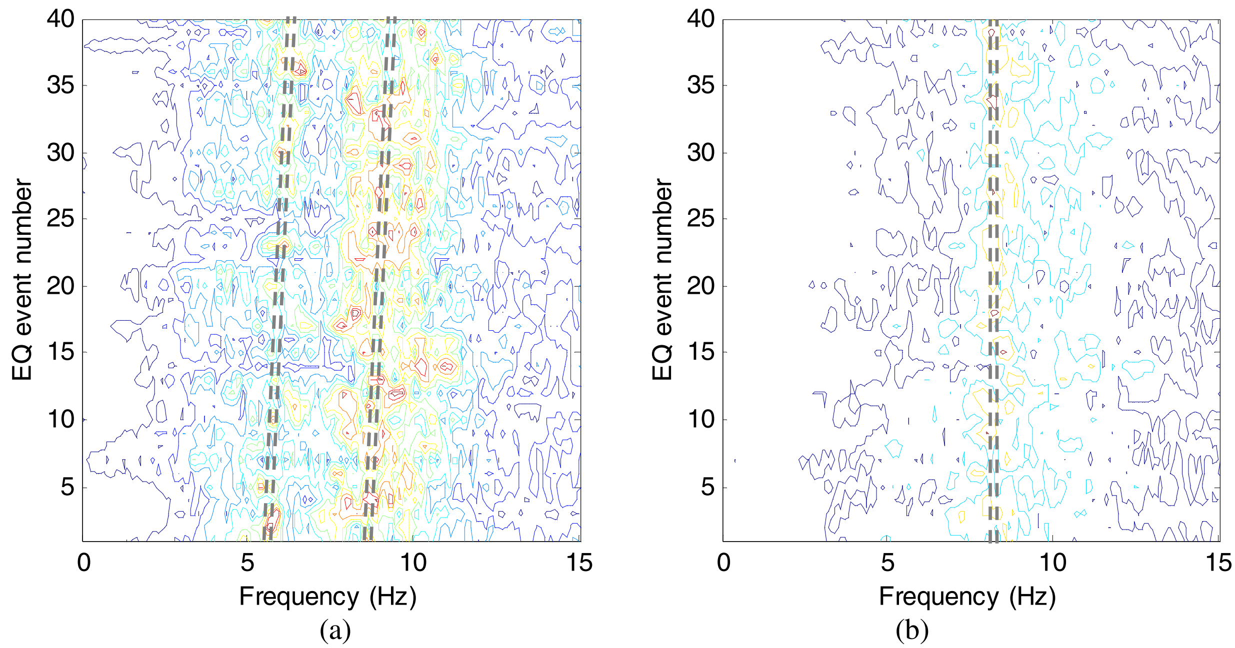


Figure 14. Contour graphs of the Fourier spectrum for the 40 events analysed. (a) Vibration in the NNE-SSW direction. (b) Vibration in the ESE-WNW direction. The double dotted lines point out the slight drift in natural frequency of each mode of vibration throughout the observation period.

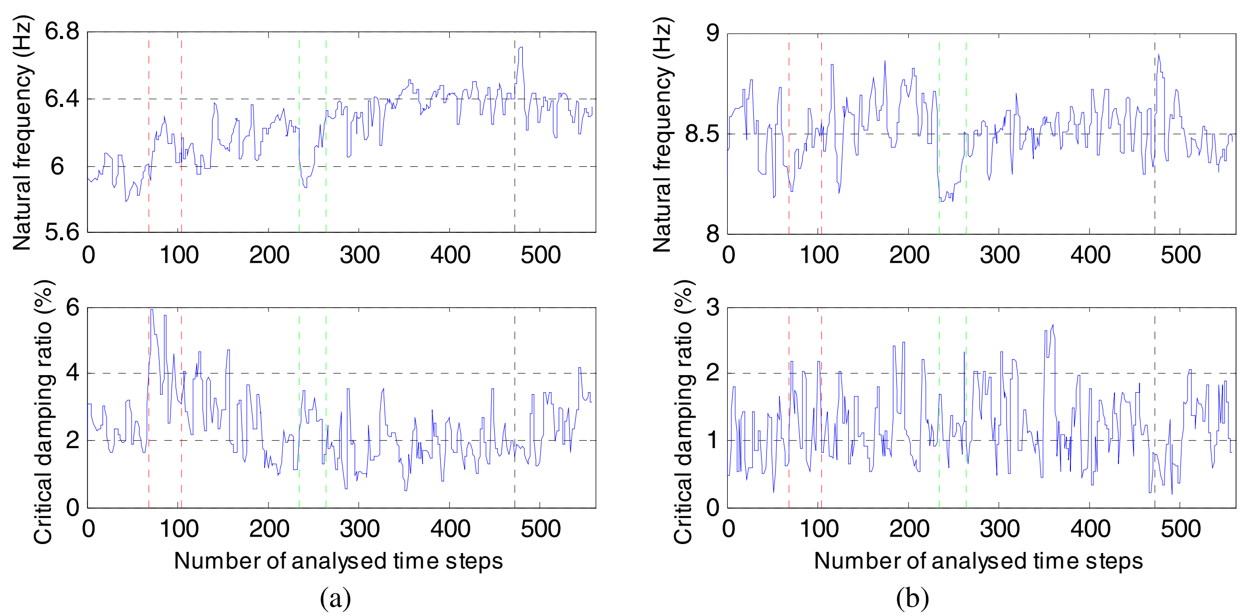
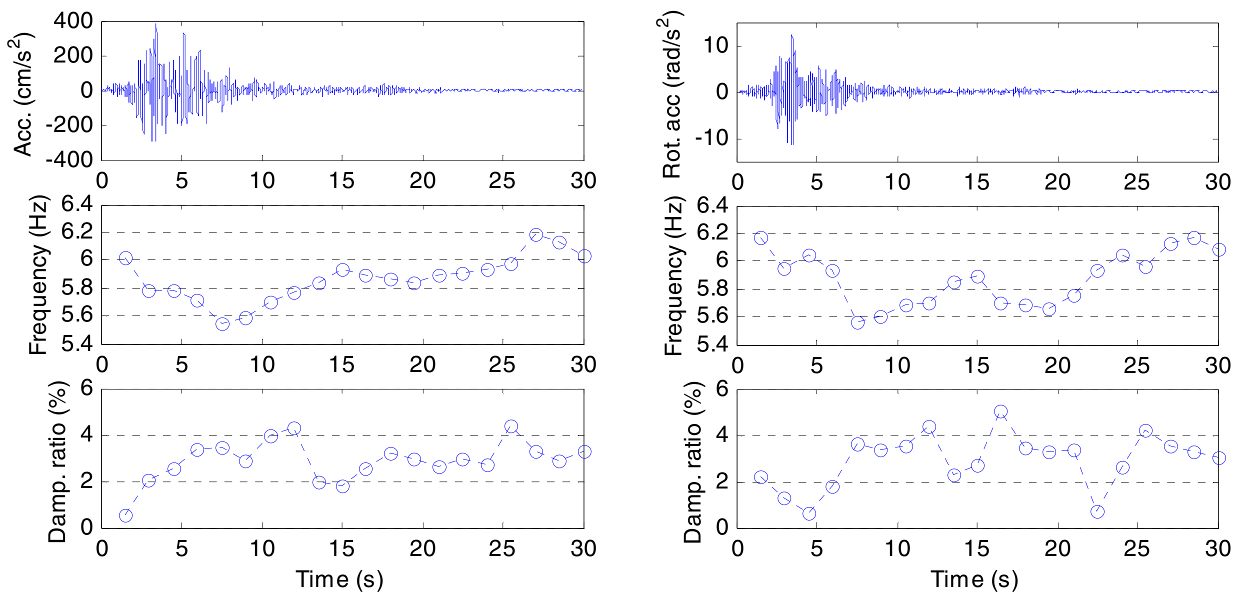


Figure 15. Natural frequency and critical damping ratio as a function the number of analysed time steps for: (a) N-S mode of vibration at 6 Hz and (b) E-W mode of vibration at 8.5 Hz. The June 17 and June 21 events are enclose by the red and green dotted stripes, respectively, while the black dotted line marks the end of the July 26 EQ, the final event in the summer of 2000.

The noticeable changes after July 2000 (see the black dotted line in figure 15) are due to the fact that the top floor was being renovated and had been stripped of all partitions and furniture and a 2.5 cm levelling layer had been removed from the concrete floor. During the renovation process the top floor was strengthened with a 5 cm reinforced concrete layer. The data recorded after the renovation shows again increased damping and decreased frequency due to the added mass from a thicker floor and heavy double gypsum partitions.



(a) (b)

Figure 16. The acceleration time series from the June 21 main event along with the frequency and damping ‘time-series’ from the same event. (a) For the first mode of N-S vibration and (b) for the rotational part of the same mode of vibration.

During the 6.5 magnitude earthquake on June 21 in 2000 [14], which had an epicentre about 15 km east of the Town Hall, the acceleration levels at the top floor reached about 39% g. An acceleration time series recorded during that earthquake is displayed in figure 16, which shows the observed N-S response of the NE corner on the top floor. The overall N-S acceleration is shown on figure 16a and the rotational component on figure 16b. The N-S direction lies across the weaker axis of the structural system, therefore the building response is highest for that direction. The frequency content of the acceleration changes throughout the earthquake. The time-series of natural frequency show how the frequency changes with amplitude. The natural frequency related to rotation seems to be slightly higher during the first 6 s of the EQ, but the frequency in both data series is seen to decrease as the amplitude of motion increases and then as the motion decrease the frequency level stabilises at about 6.1 Hz. An interesting observation considering the damping is that as the acceleration is reaching its peak the damping related to rotation is decreasing while the overall damping is increasing with increased amplitude of motion.

ELASTIC- AND INELASTIC ANALYSIS OF SELFOSS TOWN HALL

The structural model

The analysis was undertaken using the RUAUMOKO3D program [2], which is designed to undertake dynamic elastic and inelastic analysis of structures subjected to earthquake and other dynamic loadings [7]. The mesh of the model for the Town Hall building is shown in figure 17. It includes 251 nodes and 431 elements: beams, columns, wall units. The roof structure is not included in the model, but the roof mass is applied as point mass to the column tops. The two upper floors are modelled as rigid diaphragms. The model was found to represent the actual geometry and the structural behaviour adequately.

Modal Analysis

The results of the modal analysis are summarised in figure 18, which shows the mode shapes for free vibration of the structure, and in Table 1, which gives the natural frequencies. Modes 1 to 3 are the first modes in the sense that both upper floors rotate or translate in the same direction whereas in modes 4 to 6 the two floors move in opposite senses. The six modes give full participation as the two upper floors are modelled as rigid diaphragms and the structure has therefore only 6 dynamic degrees of freedom.

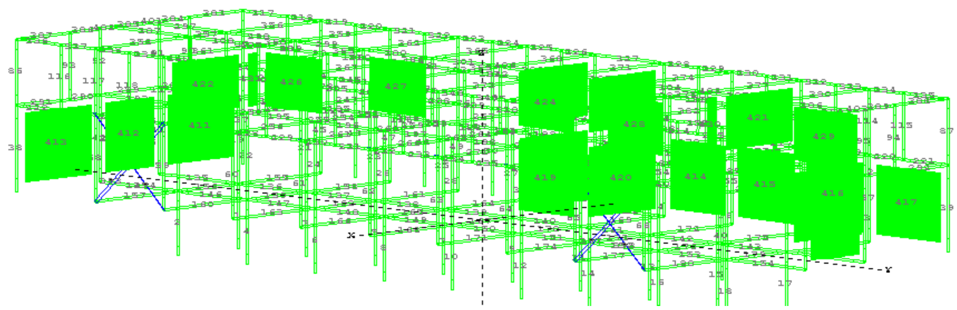
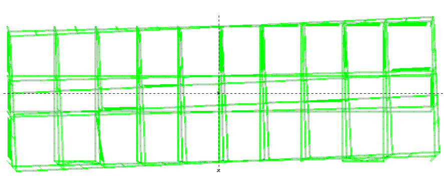


Figure 17. Overview of the building model.

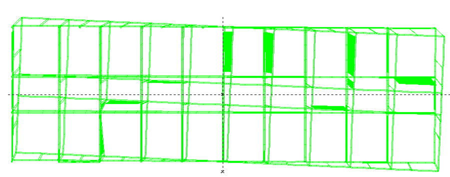
The natural frequencies of free-vibration without the cross-braces in the North Wall were evaluated to assess their effects. It will be noted that there is only a small change of about 3 % in the first mode frequency. However, there was a noticeably greater torsional action in the model without the cross-bracing system, which is consistent with analysis of the acceleration data recorded in 1999.

Table 1. Natural frequencies and modal properties for two excitation Components.

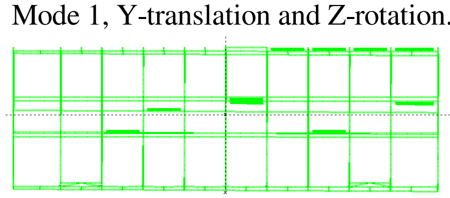
Mode No.	1	2	3	4	5	6
Frequency (Hz)	6.15	8.64	11.01	14.13	19.86	23.99



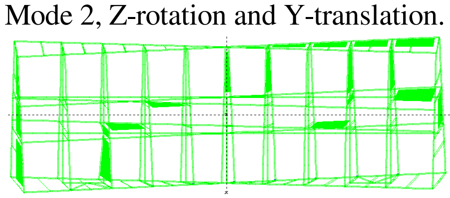
Mode 1, Y-translation and Z-rotation.



Mode 2, Z-rotation and Y-translation.



Mode 3, X-translation.



Mode 4, Y-translation and Z-rotation.

Figure 18. The first four mode shapes of free vibration.

Time-History Analyses

Analyses were carried out with the model structure for the ground motion recorded in the basement of the building in the 21st June 2000 earthquake, which has a peak ground acceleration of 0.125g. A time-step of 0.005 seconds was used for all the analyses and provision was made for iteration on residuals within the time-step if necessary. This time-step was found to be small enough to follow the response of the structure with sufficient accuracy. Several sets of analysis have been carried out, considering both the past and present building configuration and the capacity to sustain larger excitation [3].

For the event studied, the structure is essentially elastic with only a small degree of in-elastic behaviour indicated in few elements on the ground and first floor. The critical elements are the columns in the ground floor North wall and the first floor transverse beams. Figure 19 shows the hysteretic behaviour for these elements. Both hysteresis loops show a largely one-way action with progressive ratcheting of the curvatures. The stiffness difference between the two elements is reflected in the slope of the hysteresis loop.

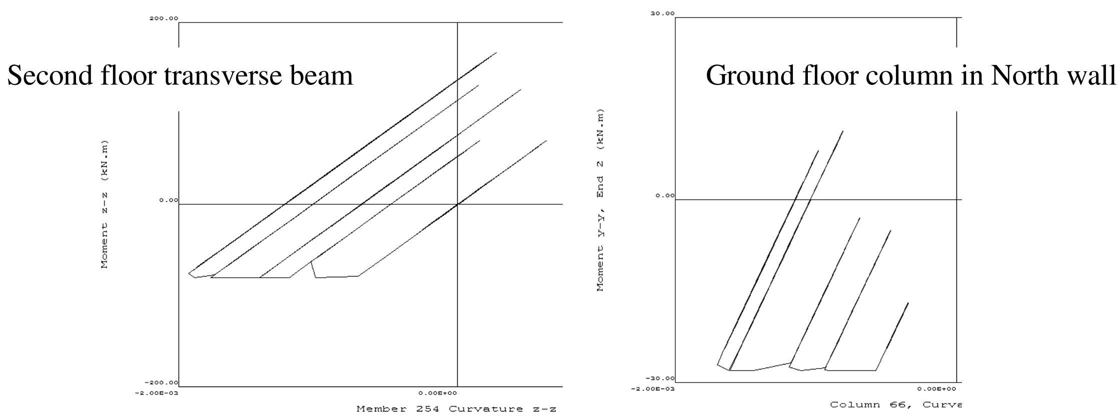


Figure 19. Hysteresis loops for a second floor transverse beam and for a ground floor column in the north wall.

The effect of adding the bracing system is seen to give a considerable reduction in the rotation of the floors about a vertical axis with a reduced danger of yielding in the ground floor columns and a lower ductility demand for the transverse beams. Doubling the EQ excitation is found to increase the risk of failure significantly. Especially as the detailing in structural members, whilst typical for a building built in the 1940s, are such that little ductility will actually be available. Secondly, there is no guarantee that the reinforcement is actually to the drawings, as there are other features of the structure that are not built according to the drawings.

The structural model will be refined and re-calibrated to account for improved and additional information on the system parameters. This work is underway and will be reported elsewhere.

DISCUSSION AND FINAL REMARKS

The presented study of the dynamic long-term behaviour of concrete buildings in seismic environments reveals some interesting features. Both buildings, which are cast-in-place concrete structures, have been subjected to repeated earthquake excitation. System identification and the available recordings are used to assess the basic dynamic properties of the buildings. These results serve both as a baseline for damage identification as well as calibration data for further structural modelling of the buildings. Changes in the system parameters are observed, which apparently depend both on time as well as excitation level. In the case of the multi-story Commerce Building a slow increase in flexibility is observed during the whole observation period, in addition to an instantaneous decrease in natural frequencies after each earthquake. This pronounced decrease in natural frequencies after the bigger earthquakes are followed by a recovery period where the natural frequencies increase slowly and tend towards the 'initial' ones. The 'instantaneous' decrease in natural frequencies is accompanied by increase in corresponding critical damping ratios, which support the interpretation of weak non-linear behaviour. For the low-rise Town Hall building the change in the dynamic characteristics of the structure is more complex. This is because the building had been modified, renovated and retrofitted, in three distinctive phases, during the observation period. The change in dynamic properties during these three phases is clearly visible. The retrofitting has clearly had beneficial effect on the behaviour of the building. An interesting phenomenon is that while the retrofitting elements are not fully contributing to the structural stiffness, they still have an effect on the response through an increased damping. Probably through friction, but also by their chaotic added stiffness input throughout the earthquake. With time, and repeated excitation, the elements become more structurally active while their damping contribution decreases. The Town Hall building sustained a severe earthquake excitation that according to an inelastic modelling and response analysis was potentially damaging. A non-linear modelling of the building and the observation of minor visible damage in form of cracks in concrete walls support this. However, further studies and back calculation applying the described computational model is needed to be fully able to identify the observed dynamic behaviour.

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