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A study of earthquake induced structural response: Measurements and prediction

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ABSTRACT: The objective is to develop and calibrate computational structural models in such a way that response induced by base motion can be reproduced. Earthquake induced base acceleration and structural response has been measured, in a 14 story office building in the city of Reykjavík. A linear finite element model has been constructed for the building. System identification has been performed utilizing the measurements and the FE-model, by applying both parametric and non-parametric methods. Calculated response using measured base acceleration as input has been compared to measured response. It is found that a FE-model can be calibrated to give a reasonably good prediction of earthquake induced response.

1 INTRODUCTION

Base acceleration and structural response has been measured, on different occasions, in an office building in Reykjavík, located just outside the epicentre area of two earthquake zones, i.e. the Reykjanes Peninsula and the South Iceland Seismic Zone. The building is one of the tallest buildings in Reykjavík, but at the same time very traditional with regard to both construction material and method.

The objective of this study is to utilize this data to develop and calibrate linearized computational structural model in such a way that response induced by base motion can be reproduced. The aim is to apply these models in reliability studies and to define parameters that could be used as damage "indicators" in a damage assessment. Furthermore, for risk management of equipment, sensitive to shock and vibrations.

2 DESCRIPTION OF THE DATA

2.1 *The Building*

The Building considered, is a 14 story reinforced cast-in-place concrete structure, basically composed of shear walls and slabs. The building geometry is rather complex. The floor plans vary, changing vertically, as shown in Fig.1 and 2, respectively. The alignment of the building is such that the translational modes are approximately in the E-W and N-S directions.

2.2 *The Instrumentation*

The building instrumentation is a part of a network of strong motion accelerometers located throughout Iceland, organized and operated by the Engineering Research Institute at the University of Iceland.

This particular building is instrumented at three levels (see Fig.2); the basement, the 8th floor and the 14th floor. A triaxial accelerometer (Kinemetrics FBA-23) is located in the basement, measuring the three components of base (ground) acceleration. On the 8th floor two uniaxial accelerometers (Kinemetrics FBA-11) are located measuring the two horizontal components of the response. On the 14th floor (the top floor) three uniaxial accelerometers are located, one measuring the N-S vibrations and two measuring the E-W vibrations in opposite corners (i.e. N-E and S-W) to detect torsional motion. The eight sensors are connected to two interconnected data acquisition units (Kinemetrics SSA-1), operating with a sampling rate of 200 Hz.

The data acquisition is triggered to start when the motion exceeds a specific acceleration level. Currently data acquisition begins when acceleration on the 14th floor exceeds 0.4 per cent of g.

To date the system has measured 18 earthquakes ranging in magnitude from 2.5 to 4.7 on the Richter scale. Their epicentre have in all cases but one, been on the Reykjanes Peninsula.

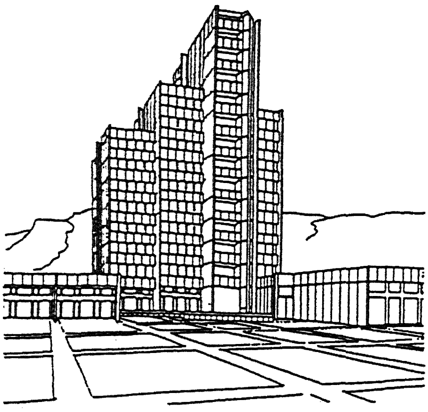


Fig. 1 The Building

2.3 *The Earthquake*

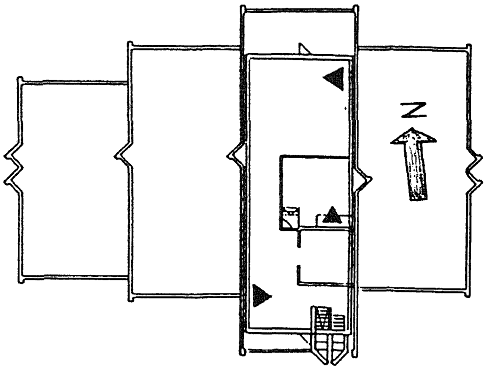
The Reykjanes Seismic Zone which is only about 2 km wide, extends from the tip of the Reykjanes Peninsula (63.8°N, 22.8°W) to the mountain Hengill (64.0°N, 21.3°W) and is regarded as the plate boundary (see Fig.3). It is one of the most seismically active zones in Iceland. The earthquakes have a focal depth of 1 to 5 km and are not located on any single fault. The focal mechanism in the area of Sveifluháls is generally a strike-slip on a N- or E-striking fault (Einarsson 1991). Earthquakes of magnitude 6 or greater have been recorded four times, the largest of magnitude 6.25 in 1929.

Reykjanes is a region of recent volcanic activity, which reduces the possibility of high stress concentration earthquakes. This is, however, an important region due to its proximity to the industrial and population centre of Reykjavík and vicinity.

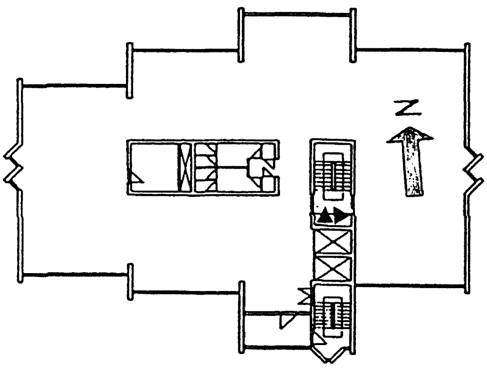
The data presented and used in this study, were measured on March 3, 1990, during an earthquake of Richter magnitude 4.7. The earthquake origin was estimated to be near Sveifluháls, a mountainous area on the Reykjanes Peninsula, SSW of Reykjavik.

The distance between the building site and earthquake epicentre was approximately 20 km. The maximum acceleration measured is given in Table 1 below. As can be seen the motion measured in the E-W direction is considerably larger than the motion measure in the N-S direction, even at ground level, although the earthquake origin is almost directly south of the building. This could indicate that S-waves dominate the P-waves, travelling north towards the building, from a N-striking fault.

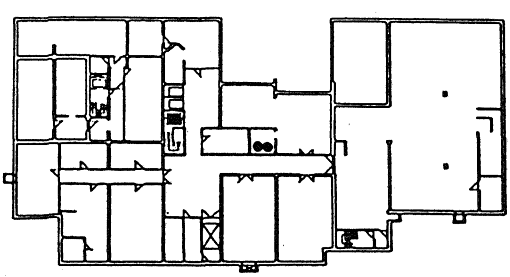
Fig.4 shows the components of recorded base acceleration. Examples of the measured response can be seen in Fig.6 and 7.



(a) The 14th floor



(b) The 8th floor



(c) The basement

Fig. 2 Floor plans of the building, showing the location of instrumentation.
● - SSA-1 Data acquisition system
▮ - FBA-11 Triaxial accelerometer
▲ - FBA-23 Uniaxial accelerometer

3 STRUCTURAL MODELLING

A linear finite element model of the building has been constructed, using 1720 elements (mostly shell-elements), and 1197 nodes, resulting in 6120 degrees of freedom.

Material related stiffness parameters used in the FE-

Table 1. Summary of earthquake information.

THE EARTHQUAKE								
Date	Time	Magnitude	Epicentre					
19.03.'90	10:46:31	4.7 (M)	63.95 N, 21.93 W					
MEASURED PEAK ACCELERATION IN THE BUILDING								
[g]	0.009	0.033	0.006	0.017	0.04	0.033	0.107	0.148
Direction	N-S	E-W	Vertical	N-S	E-W	N-S	E-W	E-W
Location	Elevator shaft			Elevator shaft		Centre	SW-corner	NE-corner
Floor no.	Basement (-1)			8			14	

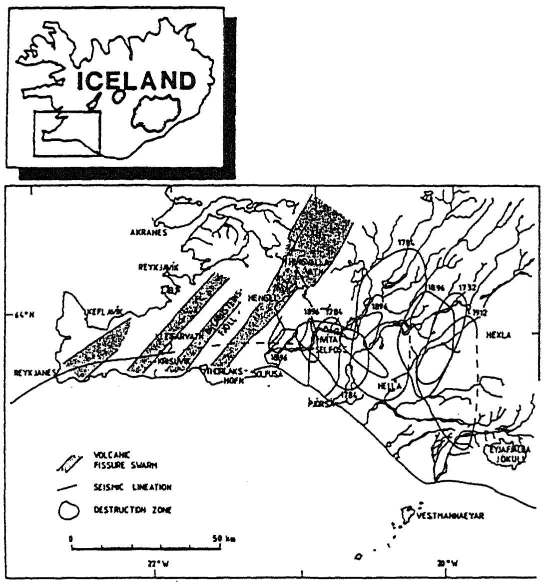
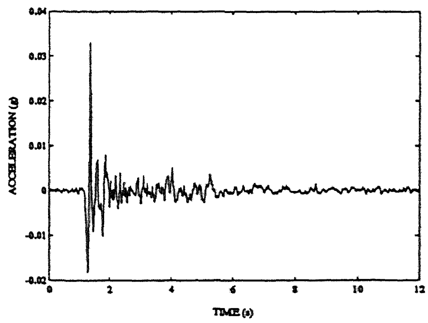
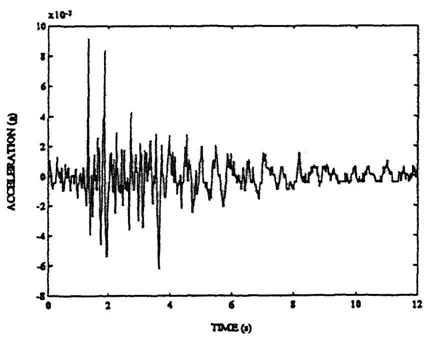


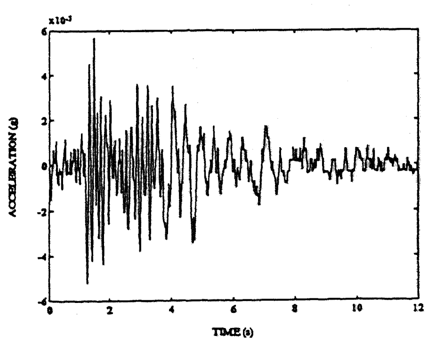
Fig. 3 The Reykjanes Peninsula and the South Iceland Seismic Zones (Sólnes 1988).



(a) E-W component



(b) N-S component



(c) Vertical component

Fig. 4 Measured base acceleration (The start of the quake is not shown).

model, have to be considered as equivalent coefficients, especially for a material like reinforced concrete, which has inherently nonlinear properties. Design values for reinforced concrete are in general not applicable, particularly not for low intensity motion such as is described above. Therefore, it is extremely valuable to have measured data that makes it possible to calibrate the different parameters according to different intensity of motion.

The finite element model was used to characterize the buildings dynamic behaviour. Table 2, shows the estimated natural frequencies, along with participation factors. The top view of the building in its natural modes for the corresponding fundamental frequencies is shown in Fig.5. The torsional nature of the structure should be noted (Hart et al. 1975).

A system identification has been performed utilizing the measured data. Firstly, non-parametric spectral methods were used to estimate natural frequencies and damping of the structure (Vanmarcke 1972, Taoka et al. 1975). Secondly, parametric methods were applied

Table 2 Dynamic characteristics of the building

Mode of Vibration	Finite Element Model: Natural Frequencies (Hz)	Participation Factors: E-W	Participation Factors: N-S	Participation Factors: Vertical	System Identification: Natural Frequencies (Hz)	System Identification: Critical Damping Ratio
E-W translation	1.7	-1823.30	438.80	-3.35	1.75	0.012
N-S transl. & torsion	2.1	-365.00	-1733.20	38.17	2.42	0.0045
Torsion	2.3	235.15	625.00	-52.10	3.30	0.020
E-W transl. & tors.	4.3	-796.80	-18.28	-14.99	4.20	0.012
E-W transl. & tors.	5.3	-267.90	-42.20	9.52	7.15	(0.020)
N-S translation	6.4	25.90	-770.70	-243.20	7.76	(0.020)
E-W transl. & tors.	8.3	-671.90	53.13	231.30	8.3	0.007
Vertical & tors.	8.7	5.31	264.40	-1658.80	9.0	0.050

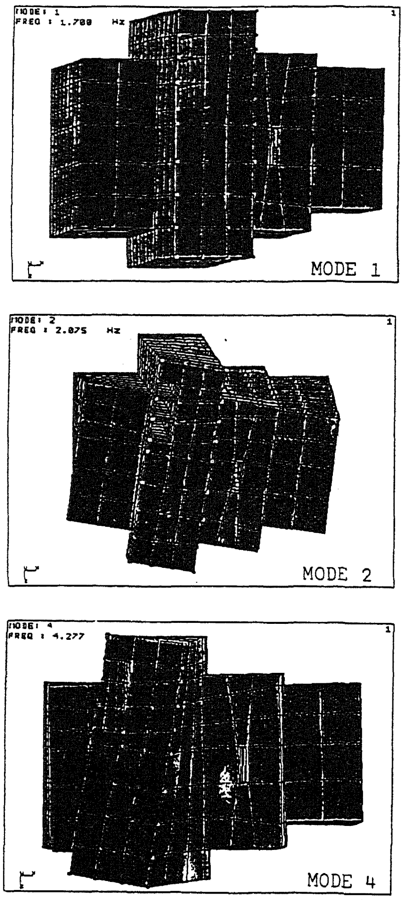


Fig. 5 Fundamental mode shapes of the building

(Gawronski & Natke 1986, Imai et al. 1989, Shinozuka et al. 1982). The equations of motion of the system were discretized and put into state-space form and an ARX model was derived. Model validation techniques arrived at the 28th order as optimum for the model, which is equal to twice the number of stories and can be interpreted physically as "degrees of freedom". The horizontal components of the acceleration measured in the basement were used as input and the response used as output. Only one input and one output was considered at a time, resulting in a single input/ single output system (Ólafsson 1990).

The natural frequencies derived form these methods are shown in Table 2, and compared with the FEM results. As can be seen, a reasonably good comparison was achieved for the fundamental modes, after some adjustment of stiffness parameters. For the higher modes the comparison is not as good; natural frequencies do not match and when the frequencies match the corresponding FEM mode shapes do not seem to fit the data, i.e. some switching of modes in the FE-model is occurring. For instance; (a) for mode nr. 4 the data indicates less torsion than the FE-model, (b) for mode nr. 6 the data indicates torsion with some E-W motion when the FE-model gives N-S motion with torsion, and (c) for mode nr. 7 the data indicates a N-S motion with some torsion, when the FE-model gives E-W motion with torsion. This indicates on one hand, that the FE-model can probably be improved, and on the other hand, it demonstrates that higher modes, especially torsional ones, may not be well presented by a FE-model unless special care is taken. The complicated geometry of the building is at least partly to blame.

Damping is a crucial parameter in the dynamic behaviour of every structure, but at the same time one that is difficult to evaluate with certainty as it is rather small for most civil engineering structures (Haviland 1976), at least at moderate response levels. As can be seen in Table 2, the damping values derived from the spectral methods are low, i.e. less than 2 per cent of critical, for the dominating modes of vibration.

4 RESPONSE ESTIMATION

When the FE-model had been adjusted to fit the fundamental frequencies found in the data, reasonably well, the frequency response method, applying FFT, was used to evaluate the response at floor 8 and 14, approximately at the locations of the accelerometers. The measured base acceleration, or rather its Fourier

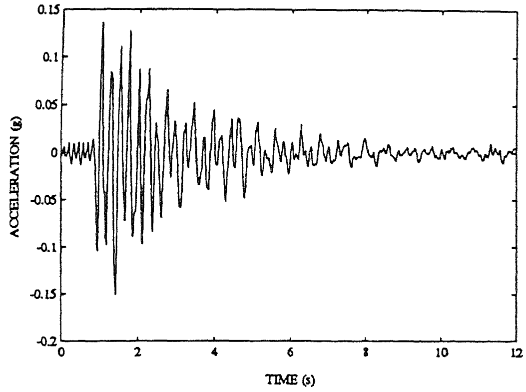


Fig. 6 Measured acceleration in E-W dir. on floor 14, N-E corner.

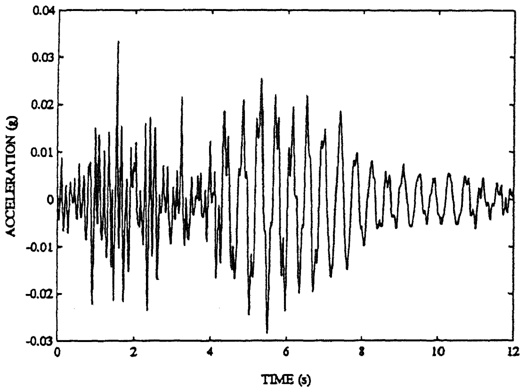


Fig. 7 Measured acceleration in N-S dir. on floor 14.

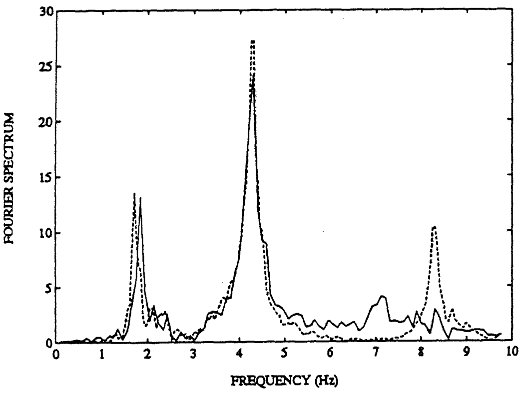


Fig. 8 Fourier spectrum of acceleration, E-W comp. on floor 14, N-E corner, measured(solid) and predicted(dashed).

transform, was used as input. Modal representation of the system made it possible to evaluate the response at the desired locations only. The finite element model provided the modal parameters, i.e. natural frequencies, mode shapes and modal participation coefficients. This resulted in a fairly straight forward, calculation of the three components of estimated response at four locations in the building; one on the 8th floor and three on the 14th floor (see Fig. 2).

As can be seen in Fig.6 to 11, there is a fairly good comparison for the E-W direction. The comparison is not as good for the N-S direction, especially it is difficult to reproduce the amplification that is seen in the N-S motion after the earthquake has passed. This might, at least partly, be due to the small intensity of the N-S component of the base motion. The fact that the FE-model does not show the "true" high frequency mode shape excited by the earthquake, adds to the difficulties. The amplification can, perhaps, partly be explained by extremely low damping in the fundamental N-S mode of vibration.

The maximum acceleration amplitudes are controlled by relatively few modes, and are most sensitive to the damping ratios associated with those modes. Slight uncertainties in damping, associated with the dominating modes, do not drastically change the peak values.

The Fourier spectrum of the acceleration is very dependent on damping. It is also very dependent on the length of the record considered since the frequency content of the response is changing during and after the earthquake. This creates difficulties in the calibration of the measured and predicted Fourier spectra. Similarly, this makes damping estimations complicated and unreliable.

Mode shapes influenced by torsion seem to have lower damping associated with them than might be expected, this is especially true for higher modes.

5 CONCLUSIONS

A linear finite element model has been constructed for an office building. A system identification has been performed utilizing the measurements and the FE-model, by applying both parametric and non-parametric methods. Natural frequencies and damping ratios have been estimated along with other system parameters.

Calculated response using measured base acceleration has been compared to measured response, both for time series directly and for spectral quantities.

It is found that a FE-model can be calibrated to give a reasonably good fit to measured response. However, as the number of frequencies and mode shapes involved increases, this becomes more difficult. None the less, this study indicates that linear FE-models, after proper calibration, can be used, with fairly good reliability,

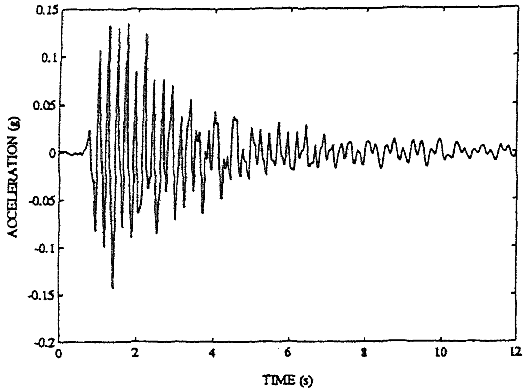


Fig. 9 Predicted acceleration in E-W dir. on floor 14, N-E corner.

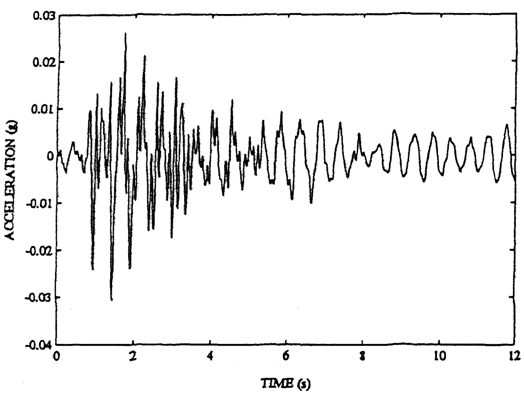


Fig. 10 Predicted acceleration in N-S dir. on floor 14.

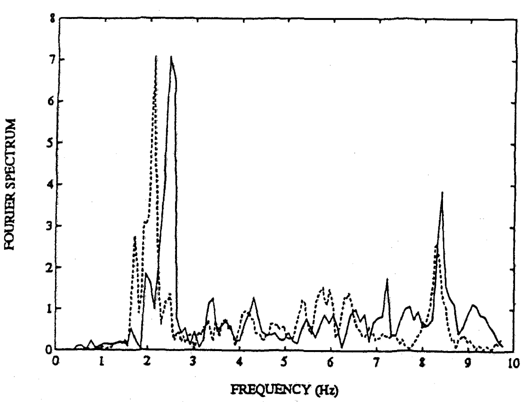


Fig. 11 Fourier spectrum of acceleration, N-S comp. on floor 14, measured(solid) and predicted(dashed).

for prediction purposes, such as in risk management and to estimate response of secondary structures, at least for moderate response levels.

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