13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 831

Seismic Evaluation of Several Hospitals in Seismic Zones 3 and 4 in Austria

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SUMMARY

This paper presents the assessment of two large hospitals in the Austrian seismic zone 3. The first step was to evaluate the dynamic structure properties by in situ measurements. Not only ambient methods but also forced vibrations with the reaction mass exciter VICTORIA of arsenal research have been used to identify the parameters. After analyzing the measured data and by using modal analysis, the natural frequencies, mode shapes and damping factors can be displayed here. There is also a comparison between the results of the ambient and the forced vibration. The next step was to measure the soil parameters by in situ measurements with VICTORIA again. Furthermore models were built by using Finite Element method to numerically calculate the forces and stresses caused by an earthquake. After every single analysis of each structure, the percentage of capacity to withstand an Austrian code earthquake will be expressed. Finally all the results were combined into risk indices to show the vulnerability of each structural member. With these indices, it is possible to draw a risk mapping plan for an easily visualization of the endangered areas.

INTRODUCTION

A national project was started in Austria to assess the seismic vulnerability of 6 large hospitals in seismic zones 3 and 4. In all cases the investigations will start with in situ measurements of structural- and soil response using a reaction mass exciter VICTORIA of arsenal research. In this way Finite Element models of the most important structures will be elaborated. Models and methods with different accuracy will be used in parallel in order to find out the most adequate model and most adequate method of analysis for each case. This is part of a long term strategy to collect information about the optimum way in order to avoid unnecessary amount of work but also to guarantee at the same time the necessary safety level. The assessment will cover the structure and the soil, but will include also functional topics, e.g. the serviceability of operation theatres, of supply of energy, water etc. Furthermore also sources for secondary risks from moving or falling equipments and secondary elements will be identified.
For the assessment mainly FEMA 310 will be used, but the Austrian seismic code B 4015 has to be considered in parallel.

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In this national project the research team has to deal with very different structures. Many of them are URM buildings and RC buildings. But all of them were built without aiming for a ductile behavior. Further, effective ground accelerations in the Austrian seismic map have been considerably increased within the last years. Facing all these facts and circumstances, it looked like the project was going to be a very demanding one. It will be of crucial importance to fully understand the structural behavior and to find out any existing overstrength- and/ or ductile potential before requesting any retrofit. This project is still not finished yet. This paper will present the result of two hospitals.

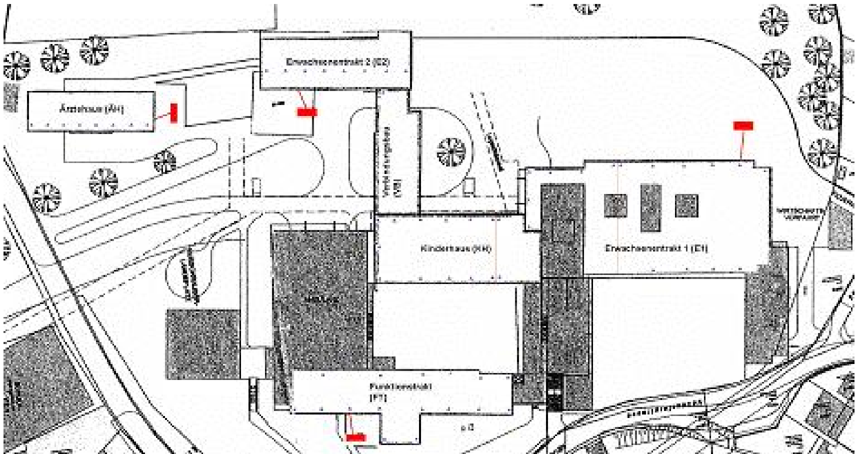
1.) IN SITU MEASUREMENTS

First step was to measure the natural frequencies of all the structures by ambient vibration and forced vibration using the dynamic reaction mass exciter VICTORIA of arsenal research (Figure 1.1.). Hereby, a rod-chain had to be fixed under 45° between the exciter and a window of the structure. This exciter works in a bandwidth between 0-80 Hz, where it is possible to execute a continuous sine-sweep with an adjustable sweep rate. The available piston elevation is max. 250 mm, the piston velocity 0.56 m/s and the piston force is 35kN. Afterwards, a sweep was brought in, which excites the whole structure and can be measured by accelerometers positioned in the building. The advantage of this method compared to the ambient measurements is that every frequency within the sweep is stimulated with high amplitudes. Even higher modes can be detected by this method.

Figure 1.1.: reaction mass exciter, VICTORIA with rod-chain

1.1.) Hospital LKH Leoben

This hospital consists of 5 main structures, Aerztehaus (AH), Erwachsenentrakt 1 (E1), Erwachsenentrakt 2 (E2), Kinderhaus (KH) and Funktionstrakt (FT). All these structures consist of unreinforced masonry in combination with reinforced concrete, or only one of these materials as mentioned before (Figure 1.1.1.).

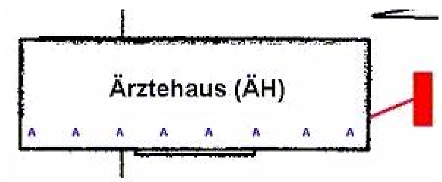


Positions of VICTORIA with its rod-chain

Figure 1.1.1.; site plan of LKH Leoben

1.1.1.) Measurements in Aerztehaus

Four measurements have been made on this structure. Firstly, an ambient measurement with seismic accelerometers has been undertaken, to find out, if the first naturally frequencies less than 1 Hz. Afterwards, three measurements have been made, two in the attic floor, and one in the staircase, with each 4, 5 respectively triaxially sensor positions. The accelerometers in the staircase were placed in every storey. The VICTORIA unit has been intentionally placed a little bit sideward of the window, so that the rod chain is angularing to the longitudinal axis of the structure to get a better excitation of the torsional modes (Figure 1.1.1.1.).



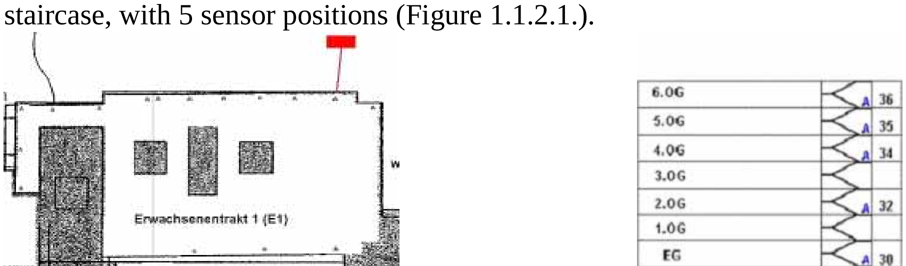
Position of VICTORIA with its rod-chain

sensor position

Figure 1.1.1.1.; position of sensors and VICTORIA

1.1.2.) Measurements in Erwachsenentrakt 1

Six measurements have been made in total on this structure. The first one was an ambient measurement with seismic accelerometers, to clarify, if the first naturally frequencies are less than 1 Hz. Afterwards, five measurements have been made, four in the attic floor with 16 sensor positions, and one in the staircase, with 5 sensor positions (Figure 1.1.2.1.).



Position of VICTORIA with its rod-chain

sensor position

Figure 1.1.2.1.; position of sensors and VICTORIA in the attic floor and staircase

1.1.3.) Measurements in Erwachsenentrakt 2

Four measurements have been made on this structure. The necessary of an ambient measurement with seismic accelerometers was again to find out, if the first naturally frequencies are less than 1 Hz. Then, two measurements have been made with 8 triaxially sensor positions in the attic floor, and one with two accelerometers at the roof between structure E2 and the neighbour house (Figure 1.1.3.1.).

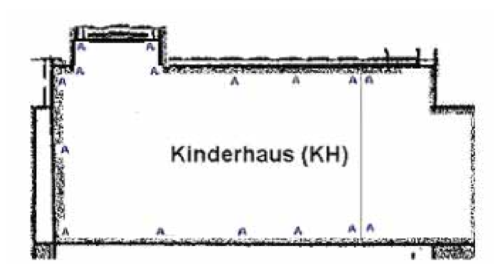
Position of VICTORIA with its rod-chain

sensor position

Figure 1.1.3.1.; position of sensors and VICTORIA

1.1.4.) Measurements in Kinderhaus

The Kinderhaus has been measured by the ambient method without forced vibration. Four measurements have been made with 12 accelerometer positions including one reference sensor which was always placed on the same position. Seismic accelerometers have been used for this test (Figure 1.1.4.1.).

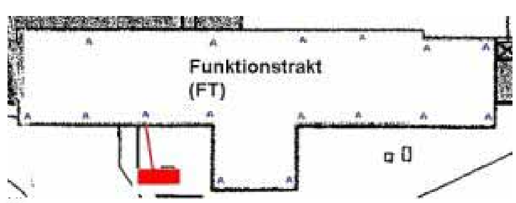


sensor position

Figure 1.1.4.1.; positions of the sensors

1.1.5.) Measurements in Funktionstrakt

Firstly an ambient measurement with seismic accelerometers has been undertaken, to clarify, if the first naturally frequencies are less than 1 Hz. Four different set ups with 14 sensor positions have been made for the forced vibration by VICTORIA (Figure 1.1.5.1.).



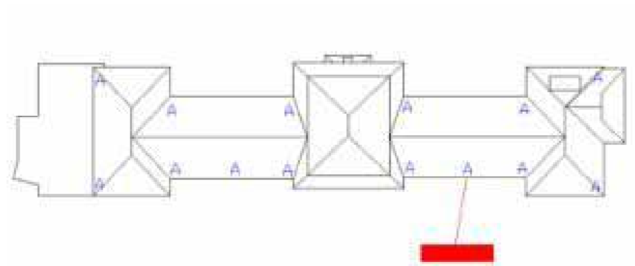
Position of VICTORIA with its rod-chain

sensor position

Figure 1.1.5.1.; position of sensors and VICTORIA

1.2.) Hospital LKH Knittelfeld
This hospital has one main structure which consists of unreinforced masonry with timber slabs in the upper floors and masonry slabs in the basement and at the aisles in the upper floors. Special investigations have been made on the masonry arch slabs.

1.2.1.) Measurements in the main structure
This building has been tested by the ambient vibration and the forced vibration method. After these two test methods, it was possible to compare the different results and to find out the utilisability of each method. Five measurements with 14 different sensor positions each have been made on this structure (Figure 1.2.1.1.).



Position of VICTORIA with its rod-chain

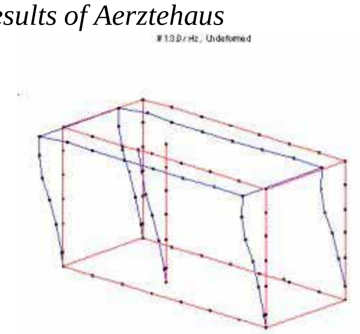
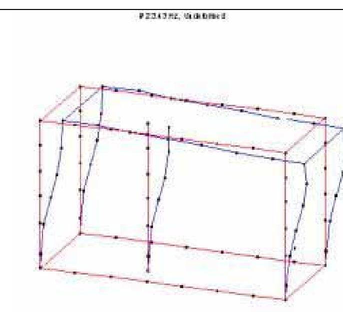
sensor position

Figure 1.2.1.1.; position of sensors and VICTORIA

2.) RESULTS AND MODAL ANAYLSIS OF MEASURED DATA

The modal analysis has been carried out by the software STAR in case of forced vibration and the software MACEC in case of ambient vibration after the in situ measurements, to get the several natural frequencies, mode shapes and damping factors. These results are listed below.

2.1.) Hospital LKH Leoben
In Leoben all the structures have been excited by forced vibration with VICTORIA, except of Kinderhaus, which has been only excited by ambient vibration (Figure 2.1.1.1. – Figure 2.1.5.3.).

<table>
<tr>
<td>*2.1.1.) Results of Aerztehaus*

f = 3.07 Hz; ξ = 4.19 %
Figure 2.1.1.1.; Mode Shape 1</td>
<td>
f = 3.34 Hz; ξ = 1.88 %
Figure 2.1.1.2.; Mode Shape 2</td>
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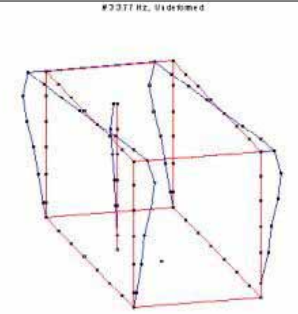
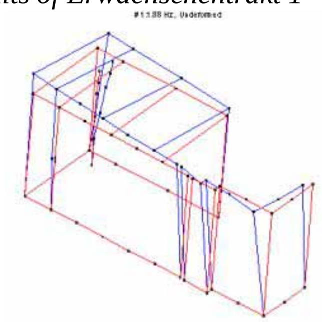
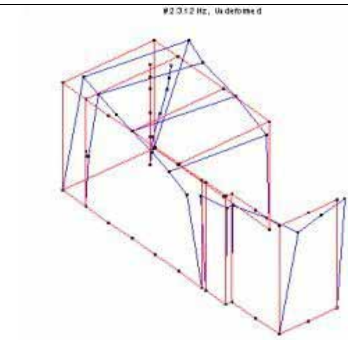


f = 3.77 Hz; ξ = 1.88 %
Figure 2.1.1.3.; Mode Shape 3

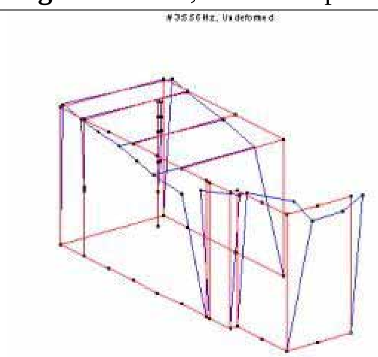
2.1.2.) Results of Erwachsenentrakt 1



f = 1.88 Hz; ξ = 11.40 %
Figure 2.1.2.1.; Mode Shape 1

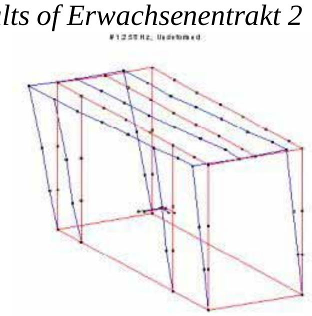


f = 3.12 Hz; ξ = 4.18 %
Figure 2.1.2.2.; Mode Shape 2

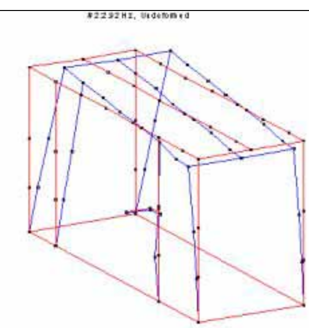


f = 5.56 Hz; ξ = 2.99 %
Figure 2.1.2.3.; Mode Shape 3

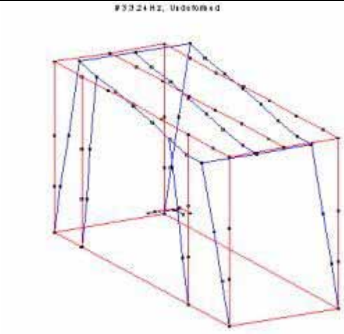
2.1.3.) Results of Erwachsenentrakt 2



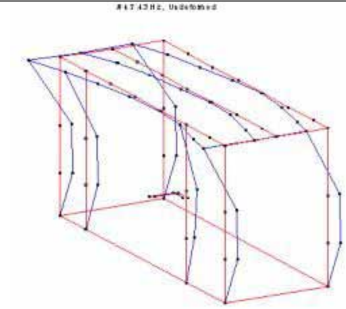
f = 2.58 Hz; ξ = 5.48 %
Figure 2.1.3.1.; Mode Shape 1



f = 2.92 Hz; ξ = 4.39 %
Figure 2.1.3.2.; Mode Shape 2

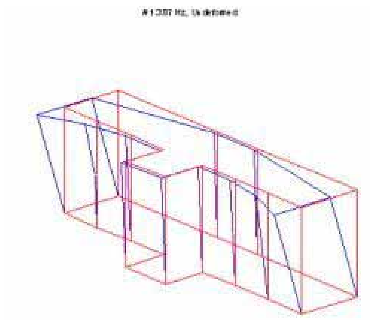


f = 3.24 Hz; ξ = 2.67 %
Figure 2.1.3.3.; Mode Shape 3



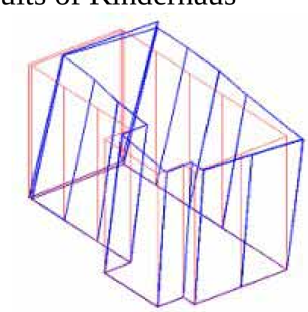
f = 7.43 Hz; ξ = 3.56 %
Figure 2.1.3.4.; Mode Shape 4

2.1.4.) Results of Funktionstrakt

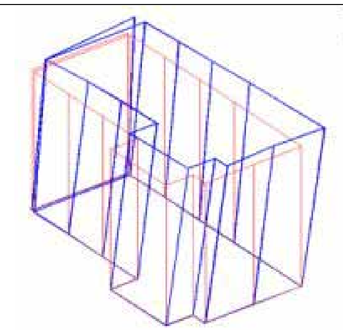


f = 3.87 Hz; ξ = 4.18 %
Figure 2.1.4.1.; Mode Shape 1

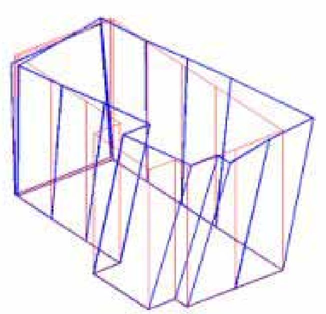
2.1.5.) Results of Kinderhaus



f = 1.60 Hz; ξ =1.20 %
Figure 2.1.5.1.; Mode Shape 1



f = 1.78 Hz; ξ = 1.00 %
Figure 2.1.5.2.; Mode Shape 2

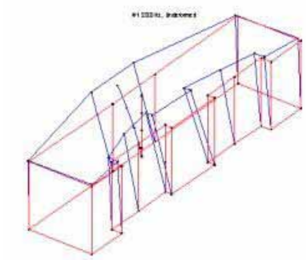


f = 2.21 Hz; ξ = 1.00 %
Figure 2.1.5.3.; Mode Shape 3

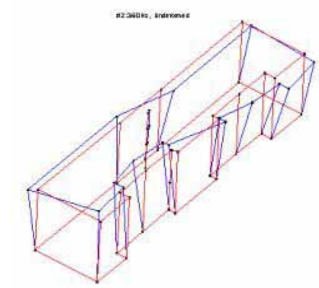
2.2.) Hospital LKH Knittelfeld

This hospital has been tested by ambient and forced vibration. The differences between the frequencies of these two types are pretty small; remarkable was the damping ratio, which has immense discrepancies to each other. Mode shapes, naturally frequencies and damping ratios of each vibration type are listed below (Figure 2.2.1.1. – Figure 2.2.1.5.).

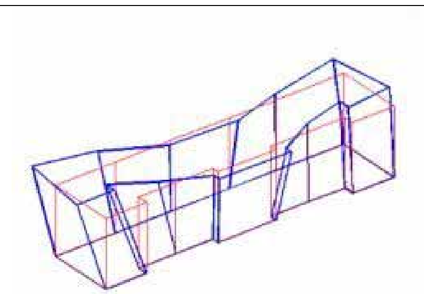
2.2.1.) Results of the main structure



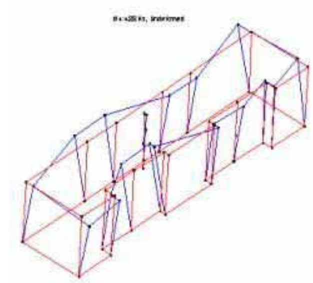
f_{forced} = 2.88 Hz; ξ_{forced} = 3.72 %
$f_{ambient}$ = 2.87 Hz; $\xi_{ambient}$ = 1.27 %
Figure 2.2.1.1.; Mode Shape 1



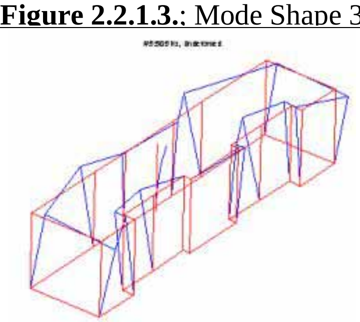
f_{forced} = 3.60 Hz; ξ_{forced} = 3.59 %
$f_{ambient}$ = 3.65 Hz; $\xi_{ambient}$ = 3.32 %
Figure 2.2.1.2.; Mode Shape 2



f_{forced} = no results; ξ_{forced} = no results
$f_{ambient}$ = 4.10 Hz; $\xi_{ambient}$ = 1.12 %
Figure 2.2.1.3.; Mode Shape 3



f_{forced} = 4.88 Hz; ξ_{forced} = 2.37 %
$f_{ambient}$ = no results; $\xi_{ambient}$ = no results
Figure 2.2.1.4.; Mode Shape 4



f_{forced} = 5.65 Hz; ξ_{forced} = 3.42 %
$f_{ambient}$ = 5.69 Hz; $\xi_{ambient}$ = 1.85 %
Figure 2.2.1.5.; Mode Shape 5

3.) IN SITU SOIL MEASUREMENTS

The method, used here for in situ soil testing for the assessment of the basic dynamic properties of soil at low strain levels is based on the principles of refraction seismic and phase velocities, Ralbovsky [1]. The velocities of compression-, shear- and Rayleigh waves and then elastic- and shear module are evaluated by virtue of experimental data, making the set of the basic dynamic soil properties being completely based on in situ measurements. Advantages of the procedure are: (1) gaining information on real soil properties on site, (2) the use of surface in situ testing methods which are relatively simply executable. (3) combining refraction and phase velocity methods.

Measurements of dynamic properties have been performed to provide more accurate inputs for calculations of the earthquake behavior of structures. Seismic Refraction Methods and measurements of Rayleigh wave velocities have been implemented in order to identify the wave velocities in the top soil layer. An optimum evaluation quantity for the process of identification was elaborated. The derivative of the RMS-value of soil velocity was used to identify more accurately the point of time, at which the force impulse arrived at the particular transducer. For this identification recommended triggering conditions were stated. The frequency derivatives of phase differences between transducers were used in the phase velocity identification process in order to provide an estimate for the consecutive identification procedure.

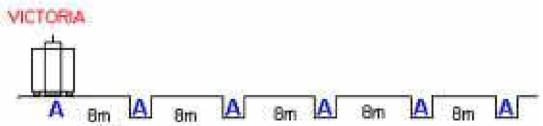
For both types of measurements the vibrations were induced by VICTORIA. The vibrations were measured up to the distance of 64 m from the exciter (Figure 3.1.).

The Poisson's ratio and shear wave velocity were calculated from the measured P- and R-wave velocities. In order to complement the measured properties, measured soil densities are necessary. Then, elastic- and shear modulus can be calculated, making the set of basic dynamic soil properties complete. Frequency dependence of the wave velocities have been investigated, resulting in a dispersion curve for Rayleigh waves.

Figure 3.1.; In situ soil measurement with VICTORIA

3.1.) Hospital LKH Leoben

Here the soil was measured with the method as mentioned before with six countersinked accelerometers (Figure 3.1.1, Diagram 3.1.1. and Table 3.1.1.) in the distance of 0, 8, 16, 24, 32 and 40 m away of the exciter.



A sensor position

Figure 3.1.1.; schematically test arrangement

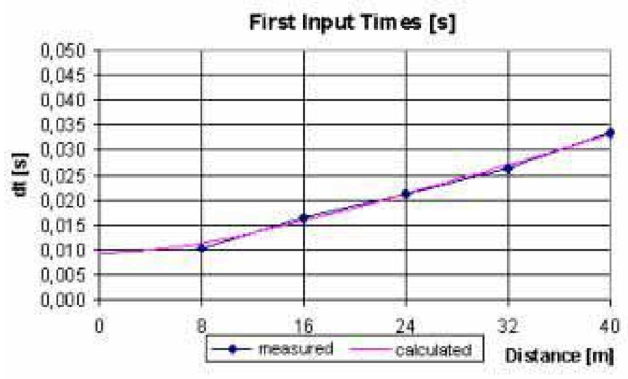


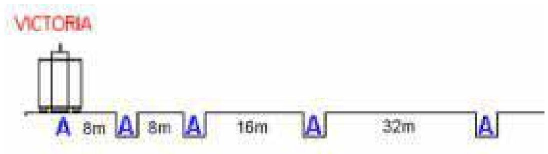
Diagram 3.1.1.; first Input times

v_P - compression wave velocity	1261 m/s
v_S - shear wave velocity	600 m/s
H - depth of upper soil layer	6.0 m

Table 3.1.1.; measured results

3.2.) Hospital LKH Knittelfeld

The soil was measured with five countersinked accelerometers (Figure 3.2.1.) in the distance of 0, 8, 16, 32 and 64 m away of the exciter.



A sensor position

Figure 3.2.1.; schematically test arrangement

It is possible to get out the velocity of the surface waves (Rayleigh waves) at different excitation frequencies with the method of phase velocities whereby phase shift of the signals of the accelerometers to the reference accelerometer is the searched value. The reference sensor was placed directly at the exciter (Diagram 3.2.1. – Diagram 3.2.2.).

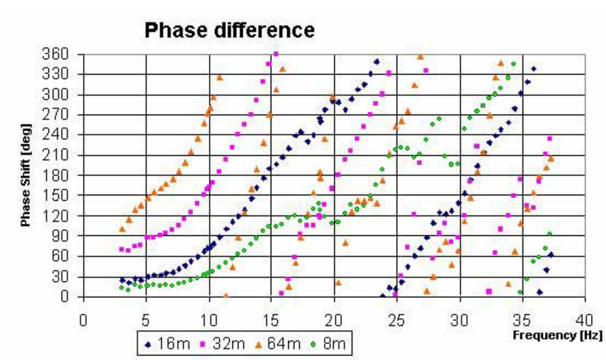


Diagram 3.2.1.; Phase shift between the sensors in the distances of 8, 16, 32, 64m and the reference sensor

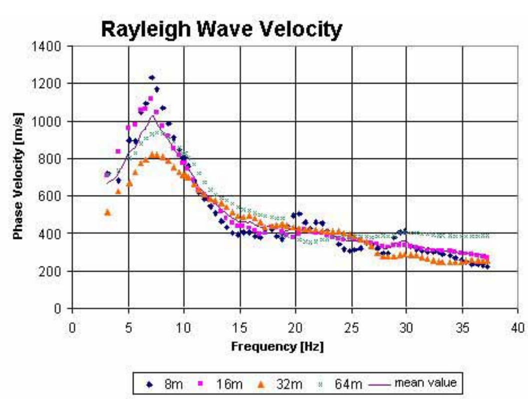


Diagram 3.2.2.; Identified wave velocities

It is possible to associate the velocity of the Rayleigh waves to several soil depths. The effective depth is between λ/2 and λ/3 respectively, where λ is the length of the wave. The area of these two borders is illustrated in the diagram below (Diagram 3.2.3.).

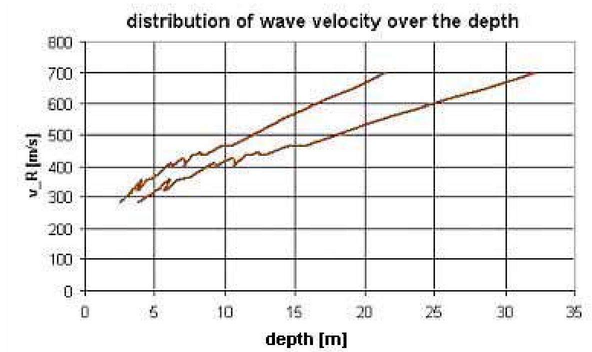


Diagram 3.2.3.; Rayleigh wave velocity in different soil depths

The shear wave velocities are normally 1.03 to 1.09 times bigger than the Rayleigh wave velocities.

4.) NUMERICAL ANALYSIS

The numerical analyzing of the structures has been made by different Finite Element softwares like ANSYS, FEMAP, STRAP or simply by hand with support of MS EXCEL. A checklist has been carried out in accordance to Tier 1 of FEMA 310 [2] for each structure, Lu [3]. Extended analyzing was calculated in accordance to the Austrian national code for designing of buildings with seismic actions, ÖNORM B 4015, version 2002 [4]. All the calculations were made for the weak axis of each structure.

4.1.) Hospital LKH Leoben

Leoben´s geographically position is: 14.194 longitude and 47.121 latitude. The soil here is very dense (soil class C in accordance to FEMA 310). After a study of ZAMG, Lenhardt [5] (Zentralanstalt für Meteorologie und Geodynamik), the Austrian Central Institute of meteorology and geodynamics, the location of Leoben has no main earthquake direction, because in the past Leoben was often an epicentre itself. The mean return period for a code earthquake with the PGA of a_{max}=0.91m/s² is 165 years. For an earthquake with mean return period of 475 years, the PGA should be a_{max} 1.23 m/s². So the Austrian code ÖNORM B 4510 is based on the knowledge of 1993 and has to be set up to the new cognitions in the future. All the calculations were still made with in accordance to the still effective code.

4.1.1.) Analysing of Aerztehaus

The Aerztehaus has been checked by the FEMA 310, and then has been calculated by the response spectra method with followed CQC procedure to combine the different natural modes. This building consists of one basement, ground floor five upper floors of unreinforced masonry at the first floor and the slabs from the second to fifth floor were supported by the externally reinforced concrete columns. This structure has been modelled with FEMAP as a three dimensional frame with columns and ribbed slabs. The loading capacity against an ÖNORM code earthquake is about 50% - 80%.

4.1.2.) Analysing of Erwachsenentrakt 1

The Erwachsenentrakt 1 consists of one ground floor, six upper floors and an expansion joint. The whole structure has been modelled as a three dimensional model, by ANSYS. The supporting elements were made of reinforced concrete. Because there was no information about the soil mechanics on this location, five different versions have been modelled to set up with the measured data. All the calculations were in accordance to EUROCODE ENV 1998-1-1 until ENV 1998-1-3, prEN 1998-1/ version January 2003 and

ÖNORM B 4015. The critically areas here are the second, fourth and sixth upper floor, whereby the capacity of the columns are about 80% of an ÖNORM code earthquake.

4.1.3.) Analysing of Erwachsenentrakt 2

This tract consists of one gound floor and seven upper floors. The analysing here has been made in two steps. The first step was to make the FEMA 310 checklist. The second step was to calculate the earthquake forces with ÖNORM B 4015 response spectra. This building consists of unreinforced masonry and two reinforced concrete frames in the longitudinal direction. The whole structure was modelled as a three dimensional model with FEMAP. The capacity to withstand a code earthquake is about 85%.

4.1.4.) Analysing of Kinderhaus

The Kinderhaus at the hospital LKH Leoben is a part of three buildings which is split by expansion joints. The used materials here were reinforced concrete and unreinforced masonry. This house has one basement, one ground floor and eight upper floors. 15 models have been built in STRAP to set the measured data with the model and the spring stiffness at the dilation joints up (Figure 4.1.4.1 – Figure 4.1.4.2.). The structure can withstand between 75% - 85% of a code earthquake.

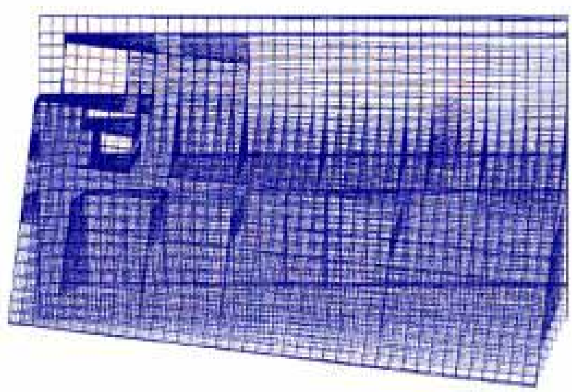


Figure 4.1.4.1.; calculated first Mode Shape f=1.52 Hz

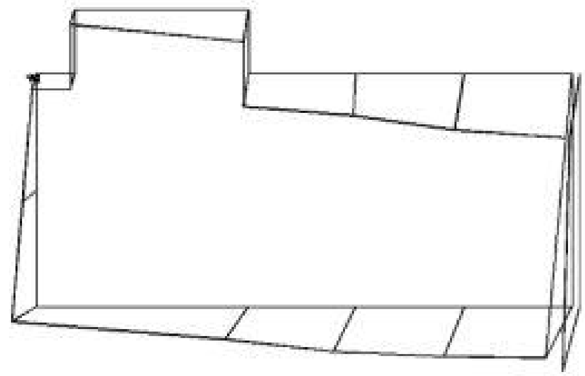


Figure 4.1.4.2.; measured first Mode Shape f=1.60 Hz

4.1.2.) Analysing of Funktionstrakt

This structure was modelled simply as a lumped mass model with five degrees of freedoms, Flesch [6], where the stiffness matrix is set up with the first measured natural frequency. The masses were concentrated at the slab levels of each floor. There are 5 floors in total, including one ground floor and four upper floors. It consists of masonry slabs above the ground floor, in the area of the aisles in the upper floors, of timber decks in the rest of the upper floors and the walls were built in unreinforced masonry. The checklist with FEMA 310 has been made as the first step of the analysing, and further a response spectra calculation by ÖNORM B 4015. The loading capacity against an ÖNORM code earthquake is about 70%.

4.2.) Hospital LKH Knittelfeld

Knittelfeld´s geographically position is: 14.821 longitude and 47.167 latitude, where the soil is middle dense to dense sandy and stony flint. The oldest parts were built in 1897 and the dominant earthquake direction is the south-east direction. The maximum acceleration PGA in the Austrian code is 1.09 m/s. But the strongest earthquake happened at this region in the past has an acceleration of 0.87m/s according to the Austrian earthquake catalogue from ZAMG. So the Austrian code is relatively conservative in this case.

4.2.1.) Analysing of main structure
The whole structure consists of masonry walls, and masonry arch slabs at the ground floor and at the aisles in the upper floors and timber slabs in the other regions of the upper floors. It has a basement, one ground floor and three upper floors. This structure has been checked by FEMA 310. The second tier was analyzed by a three dimensional Finite Element model modeled with FEMAP (Figure 4.2.1.1.) in accordance to the Austrian earthquake code. In generally, the structure has the capacity to withstand 70% of a code earthquake.

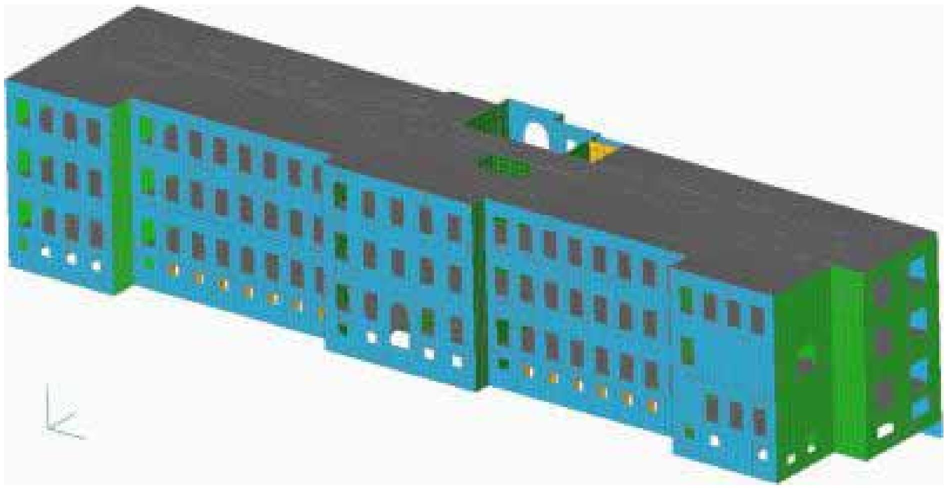


Figure 4.2.1.1.; 3-D Finite Element model of LKH Knittelfeld

4.3.) Main structural weak spots
The capacity of the structures to withstand a code earthquake was evaluated as the earthquake load that causes the failure of the first (i.e. weakest) structural member. The reason for the weak spots in the investigated structures was mainly insufficient vertical dead loading of the members. In the masonry structures (Erwachsenentrakt 1, Funktionstrakt, and LKH Knittelfeld) the bending moment in the shear walls was the crucial effect of failure. It results to tension stresses at the corners and indirectly to reducing of shear capacity of the wall, since the area of wall under compression is reduced. After-cracking Finite Element models were calculated in order to state the bearing capacity of the cracked masonry structure. It is sure, that after cracking, the whole system of the structure would get softer thus that the natural periods would raise up, and the earthquake forces would decrease. As a result, the walls were found to have no reserve in the bearing capacity after cracking due to the reduced cross section.
In the buildings consisting of concrete frames the weakest members were the columns with small axial force from dead load. The earthquake load introduces a small dynamic range of axial forces to the columns, but a large range of dynamic bending moments. The columns in the upper floors were underestimated towards these effects. The columns were designed to have sufficient shear capacity in all floors.

5.) RISK ANALYSIS, VULNERABILITY AND RISK MAPPING

To demonstrate the vulnerability and risks in structures, a risk index system has worked out with a risk mapping visualization furthermore. The risk index is based on two indices the GPR, the primary risk index and the GSR, the secondary index. The GPR depends on the capacity of the structures relatively to the code earthquake load (Table 5.1.) and GSR depends on risks which can be caused by an earthquake like dangers; by radioactive danger potentials, by flammable/not flammable gases, by resigning cold/hot water, overturning objects and by burning of objects.

GPR	relative capacity [%]	
	from	to
1	163%	infinity
2	142%	163%
3	123%	142%
4	107%	123%
5	93%	107%
6	81%	93%
7	71%	81%
8	61%	71%
9	53%	61%
10	47%	53%

Table 5.1.; span of GPR

The index GSR varies from 1 to 5. The combination of these two indices gives the total danger index, GI (Equation: 5.1.).

$$GI_{(GPR,GSR)} = GPR + \eta \cdot (GSR \cdot \exp(\frac{GPR}{\xi}) - 1) \qquad \textbf{(Equation: } 5.1.)$$

with: $GPR \in \{1,....,10\}$

$GSR \in \{1,......,5\}$

$\eta = 0{,}32$

$\xi = 8{,}30$

and: $GI(GPR = 0, GSR = 0) = 0$

After the main risk index is determined, it is possible to point out the endangered areas on each ground plan for every storey and every building (Figure 5.2. and Table 5.2.) for example for LKH Leoben, Funtionstrakt third upper floor.

Figure 5.1.; Risk Mapping

cell	Room no.	Danger potential	Danger potential	Danger index
		GPR	GSR	GI
I	28	6	3	**7,66**
II	26	3	4	**4,52**
	1	1	5	**2,48**
III	23			
IV	3	4	5	**6,27**
V	8	7	5	**10,40**

Table 5.2.; Risk indices

CONCLUSIONS

Six buildings have been investigated using different analysing methods. The most precise, most extensive and most recommended is the using of three dimensional Finite Element models that have been updated to match the experimental vibration data (used in buildings Erwachsenentrakt 1 and Kinderhaus). Less accurate method (used in buildings Erwachsenentrakt 2, Aerztehaus, LKH Knittelfeld) was using the experimental data to derive the distribution of earthquake loading of the structure according to response spectra method and applying these loads on Finite Element models without conducting a model update. The most inaccurate method would be not using any experimental data at all. The comparison between the experimental eigenvalues and not updated FE models showed a significant discrepancy due to high complexity of the structure of a real building that contains many secondary elements.

The buildings in here have been designed according to the rules that were common at the time of their construction – ranging from 1898 to 1972. The calculations showed that the main problem with the masonry buildings were low dead load stresses in the shear walls, because a large portion of the dead loads were not carried by the shear walls. In most of the historical buildings, the slabs are two way slabs which carry their loads just in shear walls of one direction off, whereby the walls in the other direction get no parts of the dead loads coming from the slabs. This can be avoided by proper structural design where load paths lead through the shear walls of both directions. The analyses of the concrete frame buildings showed a relatively good earthquake design in the most critical spots, but also showed an underestimation of structural elements that have low static loads, i.e. columns in the upper floors. The ratio between dynamic and static forces in these members is high and thus they require proper earthquake design.

The estimation of the danger index that was assigned to every room in all hospitals is intended to be used in reassigning of the room utilisation in order to minimize to total risk during an earthquake. The functionality of this lifeline facility after a catastrophic event is the primary goal.

AKNOWLEDEMENTS

This work was supported in part by the KAGES, Steiermaerkische Krankenhaus GesmbH. The authors are grateful for this support and want to thank them.

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