Report

Basel Earthquake Risk Mitigation: Computation of scenarios for school buildings

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Basel Earthquake Risk Mitigation

Computation of scenarios for school buildings

Clotaire Michel, Donat Fäh

Legend
PSA(1s) in g
- 0.05 - 0.07
- 0.07 - 0.10
- 0.10 - 0.14
- 0.14 - 0.19
- 0.19 - 0.26
- 0.26 - 0.37
- 0.37 - 0.51
- 0.51 - 0.72
- 0.72 - 1.00
- 1.00 - 1.40

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School buildings
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Abstract

The Basel earthquake mitigation project aims at computing loss scenarios for the 121 school buildings in the City of Basel, hosting nearly 17000 pupils. Contrarily to most of the previous studies in Basel, mechanical models have been used (based on spectral acceleration) instead of empirical ones (based on macroseismic intensity). They allow us to finely account for important effects of earthquakes such as resonance of ground and individual buildings that cannot be properly included with empirical methods. Our project includes a comprehensive study of the uncertainties due to the ground motion prediction and the vulnerability of buildings. In the frame of the project, the Swiss Strong Motion Network has been densified in the city and new geophysical measurements have been acquired to better understand the effect of the local geology on ground motion. These data were used to verify the results from the 2006 microzonation that is found to be adequate, in general, but that could be refined or simplified in some areas. A new amplification map is proposed based on independent results, mostly from earthquake recordings on the network. Moreover, a new method to derive fragility curves from capacity curves, based on the conditional spectrum has been proposed. It allows to better propagate the uncertainties from the capacity curves through the computation of vulnerability. The obtained fragility curves have been verified against empirical functions to ensure that they were not conservative. Vulnerability curves, expressing the losses as a function of the ground motion have been derived using these fragility curves. Loss ratios have been defined from a comprehensive literature study. The scenarios have been performed using the Openquake software, including significant pre-, intermediate and post-processing. Several scenarios based on the earthquake history of Basel (250 Augusta Raurica, 1356 Basel events) and synthetic scenarios based on the disaggregation of the Swiss Hazard 2015 for a 475 yrs. return period have been run. The distribution of damage, the number of pupils without school, injured and fatalities as well as direct financial losses are computed. The results range from few fatalities (M=5 at 5 km) to hundreds of fatalities (scenario 1356). For the 475 yrs. return period scenario, though complete collapse of a building is not certain, a large amount of buildings would be unusable. These results are associated to uncertainties. Scenarios assuming the stand before and after the retrofitting project HarmoS (33 buildings with limited earthquake safety measures) are evaluated. On average, retrofitted buildings would generate 50% less fatalities and 25% less financial losses than before their retrofitting. This project is therefore proposing a solid basis for an extension of the model to the whole city.
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1. Introduction

Earthquake scenarios aim at estimating the losses for well defined earthquakes on a portfolio of assets. They comprise monetary and human losses. They are an important tool for decision makers to design appropriate measures to face an event regarding the number of rescue teams, temporary shelters etc. They are also necessary to quantify the financial consequences of earthquakes and to evaluate the impact of safety measures such as the retrofitting of buildings.

Scenarios produced up to now for the city of Basel were either based on empirical methods (based on macroseismic intensity) and therefore relatively rough or limited to the computation of ground motion. The goal of the project is to produce scenarios following the latest standards (and beyond) to tailor them to the Basel City and therefore improve their accuracy. Such a state-of-the-art scenario modelling has to be mechanical-based and probabilistic.

Probabilistic means that the inputs of the scenarios are considered as uncertain and that the whole distribution of expected losses is computed by combining all the uncertainties. It ensures robustness to the results, points out the elements with the largest uncertainties and evaluates the final uncertainties on the results. Although this is commonly achieved in hazard assessment, this is a remarkable step forward for loss computations. Mechanical-based means that the intensity measures used are physical quantities that can be measured by instruments such as peak ground acceleration (PGA) or spectral acceleration and not macroseismic intensity. They ensure a better control on physical phenomena to extrapolate relationships to events that have never been observed in the area of interest.

In 250 AD, a presumed event with an estimated magnitude Mw=6.0 destroyed the roman city of Augusta Raurica, located close to Basel, but this event is only known through archeological findings and it remains uncertain if this event really occurred (Fäh et al., 2006b). The city of Basel has been struck in 1356 by the largest known earthquake north of the Alps. The latest studies found that this earthquake had a moment magnitude of Mw=6.6 (Fäh et al., 2009; Schwarz-Zanetti and Fäh, 2011). It destroyed the city and caused damage in many villages around, though the number of fatalities remained limited due to numerous foreshocks that made the people probably stay out of their houses. Two fatalities are known by name, the real number is unknown (Fäh et al., 2009). Other historical events of magnitude 5 and above occurred in 1650 and 1721, causing slight damage in Basel (cracks in buildings, fall of chimneys and roof tiles) (Gisler and Fäh, 2011). More recently, the 2006 geothermal induced event (Mw=3.2) caused widespread minor non-structural damage (e.g. Ripperger et al., 2009).

The present project is focused on the school buildings of the canton Basel-City. It aims at computing the monetary and human losses for different scenario earthquakes, selected based on historical events and disaggregation of the 2015 Swiss Seismic Hazard Model (Wiemer et al., 2015). Another goal is the estimation of the benefit of the retrofitting measures undertaken by the authorities in the frame of the long-term project HarmoS (www.schulharmonisierung-bs.ch/).

Fäh et al. (2001) first proposed earthquake scenarios for the city of Basel, based on macroseismic intensity, ground amplification and vulnerability classes from the EMS98 macroseismic scale (Grünthal et al., 1998). They computed the distribution of damage for different scenario earthquakes and compared the influence of the ground-motion amplification and that of the vulnerability that they both considered as critical. Wyss and Kästli (2007) proposed a loss assessment of a repeat of the 1356 event based on macroseismic intensity. They expect a number of fatalities in the whole country between 6000 and 22000, including 1000 to 8000 in Basel. Similar numbers where used in the large-scale exercise called “Seismo 12” that was based on the 14th century earthquake and tasked real-life civil servants and military personnel to respond to such an event (BABS, 2012). Mignan et al. (2015) proposed a risk model for the Basel geothermal project based on empirical methods. They showed that the epistemic uncertainties are playing a major role in the risk assessment. They also proposed a calibration of the empirical method of Lagomarsino and
Giovinazzi (2006) to match the observed losses during the 2006 geothermal event. Lang and Bachmann (2004) introduced mechanical models for the study of the vulnerability in Basel and computed the damage expected for a ground motion corresponding to the design code. They found that 45% of the unreinforced masonry structures would experience at least partial collapse and concluded that Basel was highly at risk. At that time, these results seemed very pessimistic but no improvement was proposed until the present project. This last example showed that changing from empirical assessment to mechanical-based assessment was not as simple as it seemed since the currently used simplified vulnerability assessment methods are based on design methods and are therefore conservative (i.e. they include implicitly safety factors). Another issue in seismic loss assessment is the consistency in the uncertainty in the models. Elms (1985) developed the “principle of consistent crudeness” that says the quality of a simulation depends only on the quality of the most uncertain element.

Three major scientific targets have been addressed during this project to improve the loss assessment results at the city-scale: comprehensive consideration of uncertainties, accurate ground motion amplifications for the whole city and realistic vulnerability models. Our concept of consideration of uncertainties is developed in section 2. It details the sources of aleatory and epistemic uncertainties in the scenario computations and the importance of their correlation.

Fäh et al. (1997) already showed that the local geology was critical for the ground-motion estimation in Basel. The sedimentary cover from the Quaternary and the Tertiary amplifies the ground vibrations of about a factor of 3 in the Rhine Graben. However, this factor can be regarded as moderate compared to amplification in deep alpine valleys such as the Rhone valley where it reaches factors of about 10. A large amount of geological, geophysical and geotechnical data has been collected in Basel since the 1990s. With the available data in 2006, a microzonation of the city has been published (Fäh and Huggenberger, 2006) but more data has been collected since then. An important source of data for the present study is the earthquake data recorded by the permanent network of strong motion stations (SSMNet) operated by the Swiss Seismological Service (SED). Edwards et al. (2013) described how the site amplification is routinely derived from these recordings at SED. Section 3 describes the SSMNet in Basel, as well as the new findings from the geophysical measurements and proposes amplification maps used for the scenario computation.

The assessment of the buildings’ response is presented in the report by Resonance SA (2015) and consists of so-called capacity curves. A new method to compute fragility curves from these capacity curves is developed and applied in section 4. Fragility curves describe the probability of reaching a given damage grade for a given ground-motion amplitude. Its major improvement is that it accounts better for uncertainties and propagates them in an improved manner. A large part of the work was dedicated to the verification of the fragility curves with respect to observation. For that purpose, a new method is developed and described in section 5.

In section 6, vulnerability curves are developed from the fragility curves and the loss ratios. The current state of research related to loss ratios in the literature is discussed. Finally, section 7 presents the loss assessment software and the scenarios themselves. Results for different historical events and events from the disaggregation of the Swiss hazard model 2015 are presented. The results before and after retrofitting of the schools are detailed and a sensitivity study performed.

2. Uncertainties in earthquake scenarios

Civil engineering, and therefore earthquake engineering, is generally working with deterministic approaches. However, in the last decade, following the trend of probabilistic seismic hazard assessment (engineering seismology) and structural safety assessment (civil engineering), performance-based earthquake engineering has been developed, proposing a probabilistic framework for earthquake engineering. However, little attention is paid to uncertainties within this
context (Der Kiureghian and Ditlevsen, 2009). More generally, fully probabilistic damage and loss assessment are still rare. For scenario computations, the seismic source is assumed without uncertainty, but all the other components of the computation should be considered as uncertain so that this kind of computation can be considered as “conditional probabilistic” (Crowley, 2014).

Conditional probabilistic computations aim at correctly propagating the aleatory uncertainties and their possible correlation and account for epistemic uncertainties. This is achieved using Monte Carlo sampling (or more advanced surrogate models) or inference of statistical distributions. Monte Carlo sampling is necessary if no analytical solution exists for combining distributions, which is generally the case, especially if correlation is considered. It consists in (randomly) sampling the distribution of the parameters in the model, computing the output of the model for these known parameters and repeating this procedure a large number of times to obtain the full distribution of the outputs. The components of the computation and their uncertainties have to be defined first.

2.1 Aleatory and epistemic uncertainties

Generally, and particularly for seismic hazard computations, uncertainties are separated into epistemic and aleatory and treated differently. The word “probability” has indeed both the sense of a prediction (“there are good chances that...”) and the sense of an (observed) frequency of occurrence. The first sense is often related to epistemic uncertainties and is the domain of expert decisions, while the second is often related to aleatory phenomena and statistics.

For a given problem to solve, aleatory uncertainties cover all random variations that are not explained by the used physical model and that are observed in nature. They can be intrinsic or only apparent (if a better model that has been intentionally ignored to could explain them). One can intentionally ignore a model for cost reasons or lack of data, for instance. They are generally modeled using a statistical model, generally with a normal or lognormal distribution.

For example, a ground-motion prediction equation (GMPE) has aleatory uncertainties due to phenomena such as source or site effects that are not modeled by the GMPE but these variations are clearly observed on a station recording similar earthquakes. Similarly, the vulnerability of a type of building leads to aleatory variability due to the grouping of buildings with different characteristics within the type. After an earthquake, this variability can be observed by looking at the distribution of damage within this type.

Epistemic uncertainties cover all the uncertainties due to the modeling hypotheses (the physics). Two different models will lead to 2 different results, themselves different from the unknown reality, and this difference reflects epistemic uncertainties. In seismic hazard assessment, epistemic uncertainty is treated using logic trees: each model (with its own aleatory uncertainties) is treated separately and constitutes one branch of the tree. The results for each branch are then summed up with defined weights depending on the expert judgment and belief in these models (more weight on preferred models). However, other ways of accounting for this uncertainty exist such as simple interval analysis (maximum and minimum values), fuzzy logic or the use of statistical distributions. This last technique can be used for measurement errors for instance but includes strong assumptions that may be impossible to verify. It is however the preferred way in computer science, where the latest research tends to treat aleatory and epistemic uncertainties the same way. Epistemic uncertainties might include biases that are difficult to evaluate. Moreover, they are likely correlated since part of the used datasets and methodologies are common to many models. However, this correlation is difficult to assess.

For example, in the Swiss Hazard Model 2015, 3 different source zonation models of the country are considered, each of them with its seismicity distribution and uncertainty. Different models to predict ground motion (empirical or simulation-based) are also used, each of them having its own aleatory uncertainty. The task of the expert is to assign weights to these models so that they cover the center, body and range of the possible models.
Assessment, experts are not sure about the actual capacity of existing buildings since only few data exists but they can provide their degree of belief on the models.

Modeling a new physical phenomenon will allow to decrease the (apparent) aleatory variability but an epistemic uncertainty (hopefully smaller) will be added because the model used is not perfectly representing this phenomenon (by definition). The segregation between aleatory and epistemic uncertainties is therefore problem-dependent (Der Kiureghian and Ditlevsen, 2009) and depends also on the means invested to solve the problem. As explained above, more means allow to move some uncertainties from the aleatory part into the epistemic part by using more complex models and decrease the epistemic uncertainties by improving the accuracy of the models by additional measurements or research. However, a mistake that should be avoided is the double-counting of uncertainties. Indeed, among a class of existing buildings, there is an expected variability (aleatory) of the capacity due to variability in the construction quality etc., but also an intrinsic uncertainty (epistemic) on the capacity whether this class of structure is behaving better or worse than expected. Accounting for these two types of uncertainty without double-counting is not an obvious task. In ground motion modelling, a typical case of double-counting concerns site effects: aleatory uncertainties of the ground motion prediction generally already include those due to site effects, so that this uncertainty should not be counted a second time when accounting for site amplification. Another problem, often observed in the case of few data points, is that epistemic uncertainties can be underestimated. They have to be taken large enough so that additional data would decrease these uncertainties, although, in practice, it is often observed that additional data increase the uncertainties. It therefore depends on the skills of the experts to cover the full range of epistemic uncertainties.

2.2 Sources of uncertainty for hazard assessment

The modeling of scenarios imposes the source location, geometry and the magnitude without uncertainties. Ground-motion prediction equations (GMPEs) that predict the expected ground motion at each point of the grid for a given source are provided with their aleatory uncertainty. The aleatory variability can either include all the effects source/path/site or just source/path (and is then called single station sigma). The details of all the sources of uncertainty for each component source/path/site are not given here (e.g. Strasser et al., 2009). Modern probabilistic hazard computations refer to a particular rock profile and GMPEs are used with their single station sigma. Single-station sigma includes the inter-event variability and the share of the intra-event variability that is not related to the site (single site – SS – variability). For scenario computations, one could even argue that the inter-event variability, depending only on source and path, could be omitted since source properties (magnitude and location) are anyway fixed, although this has not been done in the following.

In probabilistic seismic hazard assessment, in order to account for the epistemic uncertainties in the GMPEs, several of them are used in a logic tree. The results for one branch of the tree are summed up at the end with weights reflecting the confidence in the model used. We did not account for epistemic uncertainties in the GMPEs for technical reasons (see section 7) and also because one can argue that fixing the source properties makes it irrelevant.

Site conditions are treated with their epistemic uncertainties for example using different velocity profiles or amplification maps that reflect our lack of knowledge. In this case no additional aleatory uncertainty is generally added, although uncertainties related to non-linear phenomena could be accounted for.

2.3 Sources of uncertainty for vulnerability assessment

In the Basel project, we decided to estimate the vulnerability of building types, represented as the probability of exceedance of given damage grades as a function of an intensity measure, hereafter called fragility curves. They include all the assessed uncertainties (aleatory and epistemic). However, since they include also epistemic uncertainties, they cannot be directly com-
pared or calibrated with the observed damage distribution after an earthquake: a real damage distribution only includes aleatory effects, not our uncertainty in the models.

The **aleatory** uncertainties are related to: 1. the **quality of the intensity measure (IM)** chosen as x-coordinates of the fragility curves (performance in representing the response of the structure) and 2. the variability of the **building construction characteristics** within the considered type.

1. In order to estimate fragility curves, simulations of the structural response are performed using a set of ground motions in order to estimate the distribution of damage with respect to the ground motion characteristics (mainly amplitude) (see Fig. 1). Depending on the parameters (IMs) chosen to represent these characteristics, variability in the results is found. Indeed, two ground motions with the same peak acceleration for instance, may have very different impact on the same structure due to their possibly different frequency content and duration. This variability is larger if PGA is chosen instead of spectral acceleration (Michel et al., 2012) for instance. If the vulnerability curves are used in a specific location like Basel, the set of ground motions used should be ideally representative of the study-scenario. This should avoid the overestimation of uncertainties (and possible biases). For example, the response of the buildings to Magnitude 8 events should not be accounted for in the computations. Note that this would not be correct if the vulnerability curves were planned to be used worldwide for a given type of buildings, this is again problem-dependent. It should be also noticed that this uncertainty is due to the use of fragility curves as intermediate parameters in the scenario computations: if structural models were directly implemented, this issue would disappear since one would directly compute the response to the considered ground motion for the scenario.

![Fig. 1 General scheme to derive fragility curves. Note that the structural model is considered as uncertain.](image)

2. Grouping building into types necessarily produce variability in terms of construction characteristics. According to Spence et al. (2003), this is the major source of uncertainties. It depends however on the heterogeneity of the types. The types can be relatively homogeneous for a single city but very heterogeneous if few types for the whole Europe are considered. This variability is not physically modeled on purpose, because it would imply a too large effort to model each building separately with its own characteristics. Therefore, it is modeled as a statistical distribution (aleatory uncertainty).

The fragility curves with their aleatory variability are generally modeled using lognormal distributions characterized by a median $\mu$ and a standard deviation $\sigma$. Since sources of uncertainty 1 and 2 are independent, they can be summed up following:

$$\sigma = \sqrt{\sigma_1^2 + \sigma_2^2}$$
The epistemic uncertainty comes from the fact that no model is able to perfectly compute the response of a building to an earthquake. Using a simple engineering model will lead to a different result than using a finite-element code. The uncertainty depends on the amount of physical phenomena included in the model and the relevancy of the simplifications that are made. The error due to the assumed distribution model also counts as epistemic uncertainty (lognormal distribution often used in the case of the fragility curves). Epistemic uncertainties in the fragility functions are in practice rarely modeled (Bradley, 2010).

Though researchers around the world are working to improve this, the uncertainty in vulnerability modeling is still large and not well taken into account. Fig. 2 illustrates what could happen if the used models are biased: modelling only aleatory uncertainties (black and grey curves, generally done) will not allow to cover the true behavior (blue curve), while accounting for epistemic uncertainties (red curve, rarely done) will, but with such a large uncertainty that it might not provide useful results. According to Fig. 2, the only way to improve is to better model (avoid biases) the structure. Biases in modelling are due to model simplification for instance in the geometry, approximations in the dynamics of structures, conservativeness of the assumptions etc. (e.g. Goulet et al., 2014).

![Graph illustrating the comparison between probabilistic modeling accounting for aleatory and/or epistemic uncertainties and reality](image)

**Fig. 2** Illustrative comparison between probabilistic modelling accounting for aleatory and/or epistemic uncertainties and reality: if models are biased, accounting for uncertainties hardly solves the issue. GM means ground motion. EDP stands for Engineering Demand Parameter, for instance displacement response of the structure (from Bradley, 2013).

The vulnerability assessment is further performed by the combination of the fragility curves and loss ratios (fatalities, injured, financial...). This generally implies the conversion and the use of proxies in the definition of damage grades that are also a source for epistemic uncertainties: mechanical definitions may not perfectly match with a definition based on observation of damage. Moreover, the loss ratios generally used include aleatory and epistemic uncertainties: the aleatory uncertainty comes from the large observed variability in the loss ratios from damaging earthquakes and the epistemic uncertainty comes from the expert judgment that selects and modifies the observed ratios to better match how the expert thinks the study-area would be affected.

### 2.4 Correlation of probabilistic distributions

When the parameters of the computation are considered as random variables with a given probability distribution, their combination in the computation should also account for their possible correlation. The expected damage and its distribution can be simulated independently for each structure, since no correlation exists between the ground motion and the vulnerability of the structures. However, the computation of the expected aggregated losses for a portfolio of assets (buildings) should account for the correlation of the loss distributions of the different structures. Indeed, the sum of two correlated variables is different from the one of independent variables (see appendix A).

It has to be stressed that this concerns the correlation in the uncertainties only, since the correlation in the mean values is deterministically modeled and has therefore not to be accounted for...
Additionally: two structures at the same distance from the source will naturally experience a similar high ground motion, for instance.

Some sources of correlation in the statistical modeling of uncertainties are generally not accounted for. Weatherill et al. (2015) studied the two most important sources they found: the spatial correlation of the intra-event residuals of the GMPEs and the inter-period correlation in the generation of response spectra from the GMPEs. They are detailed in the following paragraphs.

The aleatory uncertainty in the ground-motion prediction can be separated into an inter-event variability and an intra-event variability that shows a spatial correlation (e.g. Jayaram and Baker, 2009). As an example, the radiation pattern of the earthquake is modelled in GMPEs in the aleatory uncertainty, and is clearly spatially correlated: the effect of the radiation pattern on two buildings is larger if they are close than if they are far away from each other. The same is true for directivity effects. A part is also due to the too rough modeling of site effects in classical GMPEs. Jayaram and Baker (2009) or Esposito and Iervolino (2012) showed for example this spatial correlation of intra-event residuals from GMPEs. We made additional tests using the Swiss Stochastic model (Edwards and Fäh, 2013) and found comparable results to the ones of Jayaram and Baker (2009) (see appendix B). The model of Jayaram and Baker (2009) will be further used in the scenarios.

Another lack in most of the GMPEs is the absence of the correlation between the different estimated parameters (PGA, Sa(1s), Sa(2s) etc., Fig. 3), with the notable exception of Akkar et al. (2014a,b) as developed in section 4. When generating a response spectrum from a GMPE, one has two possibilities: either assuming a full correlation (typical assumption) and therefore choosing only one random value \( \epsilon \) in the distribution and adding this value to the mean spectrum, or assuming no correlation and choosing one random value \( \epsilon \) per period. The first assumption leads to a too smooth spectrum, the second to a too fuzzy spectrum (Fig. 3). Observations lay in-between, with a correlation for close periods, decreasing when the periods have larger difference (Baker and Cornell, 2006). This is important for loss estimation, combined with spatial correlation, if spectral values at different periods are used (typically for buildings of different sizes and types). Weatherill et al. (2015) showed that it impacts especially the losses for large ground motions and for small-scale studies like cities. In both cases, the losses are large and the correlation plays an important role. The impact of the spatial correlation is not critical according to their results, contrary to the inter-period correlation. The assumption of a full inter-period correlation (e.g. using a Newmark spectrum) leads to unrealistically high losses. When it is not possible to account correctly for inter-period correlation, Weatherill et al. (2015) recommend to use a single intensity measure for the whole building stock (not at different periods) to avoid this issue, leading clearly to higher uncertainties.

![Fig. 3](image) Comparison of simulated spectra with no inter-period correlation (a), full inter-period correlation (b), modelled inter-period correlation (c) with real spectra (d) (from Baker and Cornell, 2006).
Another source of correlation has been neglected up to now: that in the epistemic uncertainty in the fragility curves (Crowley, 2014). Part of this uncertainty is due to the fact that data is lacking for the region of interest. The epistemic uncertainty in the vulnerability of buildings located in the same area is most probably correlated since the techniques and material used for the construction are more homogeneous for a given region than assumed if a worldwide set of buildings is considered. In order to cover all cases, Crowley (2014) suggests to run scenarios with full and without correlation to estimate bounds in the loss results.

Fig. 4 summarizes the workflow of scenario computations stressing on the type of uncertainties that should be accounted for. This scheme is not fully general and reflects the assumptions taken for our project, as listed in Tab. 1.

![Workflow of scenario computations with the types of uncertainties that should be accounted for.](image)

**Tab. 1 List of uncertainties accounted for in our computations.**

<table>
<thead>
<tr>
<th>Uncertainty</th>
<th>Aleatory</th>
<th>Correlation in aleatory</th>
<th>Epistemic</th>
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</thead>
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<td>None</td>
<td>None</td>
</tr>
<tr>
<td>GMPE</td>
<td>From GMPE (single station sigma)</td>
<td>From Jarayam and Baker (2009)</td>
<td>None</td>
</tr>
<tr>
<td>Site amplification</td>
<td>None</td>
<td>From Jarayam and Baker (2009)</td>
<td>None</td>
</tr>
<tr>
<td>Seismic capacity</td>
<td>Variation in model parameters</td>
<td>None</td>
<td>Different modelling assumptions (collapse mechanism)</td>
</tr>
<tr>
<td>Seismic demand</td>
<td>From GMPE for a single scenario</td>
<td>From GMPE</td>
<td>None</td>
</tr>
<tr>
<td>Loss ratios</td>
<td>From literature study (undifferentiated)</td>
<td>None</td>
<td>None</td>
</tr>
<tr>
<td>Exposure</td>
<td>None</td>
<td>None</td>
<td>None</td>
</tr>
</tbody>
</table>
3. Amplification of ground motion in Basel

Basel is located at the southern-east edge of the Upper Rhine Graben, where deep sedimentary layers deposited in the last 35 Myrs. Outside of the Rhine Graben, several valleys are filled with unconsolidated Quaternary sediments. It has been recognized since 20 years that the surface geology amplifies the earthquake ground motion in Basel. This amplification depends however on frequency and varies with the mechanical properties of the deposits. Fäh et al. (1997) first proposed a qualitative microzonation of the city of Basel. They collected geological and geotechnical data, used SPT (Standard Penetration Test) to retrieve the shear-wave velocity (Vs) and used about 20 single station measurements interpreted using the Horizontal-to-Vertical (H/V) Spectral Ratios to retrieve the resonance frequencies. They interpret the fundamental frequency as the resonance of the Tertiary layers down to the Sannoisian marl, based on the transition between soft sediments and rock observed in deep boreholes. They further proposed a qualitative microzonation based on indices, mostly relying on the Quaternary geology (14 indices out of 20). It was implemented in the whole city and related to amplification in terms of macroseismic intensity in Fäh et al. (2001). In parallel, the Swiss Strong Motion Network (SSMNet) started to develop in Basel. Nine stations were installed in 1990, 1993 and 1997/1998. Kind (2002) further developed the use of single station measurements (255 measurement points) and introduced ambient vibration array measurements to retrieve the Vs profiles (5 sites) (Kind et al., 2005). Further 2D numerical models were performed, and the Rhine Graben was split into five zones with assumed similar amplification. A vertical seismic profiling was performed at site Otterbach in the frame of the Deep Heat Mining Project (geothermal project) and helped constraining the velocity at depth. Steimen et al. (2003) studied 2D resonance in the Rhine Graben using observation and modeling and concluded that no 2D resonance was occurring in Basel. Oprsal et al. (2005) proposed the first 3D model to simulate earthquake ground motion in Basel with an extended source using the finite difference method. The velocity structure was based on the model of Kind (2002). Fäh and Wenk (2009) interpreted all measurements and simulations, and presented a quantitative microzonation for Basel that was then implemented in the design codes. In this project, additional single stations (700 points) and array measurements (30 sites) were performed, together with active seismic tests to improve the velocity model (Havenith et al., 2007). Ground-motion modelling for 1D, 2D and 3D structures were performed. For the microzonation, the city and adjacent areas have been split into 14 zones (and more subzones). This project was also the opportunity for an extension and a modernization of a part of the SSMNet in Basel, where 8 modern stations were installed in 2005. Following the 2006 geothermal event (Mw=3.2) that lead to about 9 MCHF damage claims, the velocity model was improved by Ripperger et al. (2009). They integrated the latest geological and geophysical results. The vertical seismic profiling at the deep boreholes of Otterbach and Basel-1 were made available for the study. Some limitations of the original 3D model to reproduce the ground motion were noted (Fäh et al., 2008).

Since then, new geological data have been made available (GeORG project team, 2013). In the frame of our project (2013-2015), in parallel to the national renewal project (2009-2019), the SSMNet was further modernized and extended. For new stations, the procedure includes site characterization. Temporary stations were further installed during our project. New geophysical measurements were performed and existing data have been reprocessed, improving our understanding of the ground properties in Basel. Moreover, the earthquakes recorded by the network delivered important data about the ground motion amplification at the numerous station sites, allowing a verification of the 2006 microzonation. Today, a revision of the 3D model with the entire dataset would however be advisable.

Based on these new findings, ground-motion amplification has been investigated for use in the scenarios and amplification maps are proposed. The primary goal of the amplification modeling is to retrieve the amplification for the studied buildings, i.e. the amplification of the response spectrum between 1 and 6 Hz (range of the resonance frequency of the building types proposed by Resonance SA, 2015). The studied school buildings are all located in the Rhine Graben except the school Zur Hoffnung in Riehen and the Bettingen school (zone Flexur-Nord in the microzonation). The school buildings in the Rhine Graben are located in the zones Basel-Nord and Rheingraben Ost.
of the microzonation, except the 6 buildings of school Bruderholz located in the zone Rheingraben-West. An additional goal of the project is to include the detailed amplification map for the Shakemap system with a high resolution for the Basel region. This second goal is also in line with the aim of the project to propose a strategy for the real-time scenario computation after strong events.

3.1 Geology

This description of the geology of the area is focused on the units that have an impact on the ground response as reported in the following sections. The latest results and naming conventions from the GeORG project (GeORG project team, 2013) are included but the former names may still be used in the rest of the document.

The area of Basel (Fig. 5) is made of two distinct domains:
- The Rhine Graben opened during the Oligocene and Miocene ages (starting 35 Myrs ago) and has been filled with deep marine and freshwater sediments since that time (Fig. 6).
- East of the master fault of the Graben, the Tabular Jura is made of Mesozoic rock, carved by the Rhine and smaller rivers, generally overlaid by Quaternary sediments of variable (but generally limited) thickness. In the South, the Graben extends to the folded Jura with a gradually decreasing thickness.

The limits (East and South) between these two domains are not easy to define and depend on the objectives of a study. These limits can be considered using tectonic arguments (main eastern fault – that is not well defined), geologic arguments (presence of tertiary sediments deposited during the extension of the Graben) or results from engineering seismology (e.g. resonance frequency above a given frequency - 1Hz used in Fäh et al., 2006c). For this project, a mapping of these units using a GIS software based on the geological maps and the 2006 microzonation map is proposed (Fig. 5). It is simplified on purpose, some surface sediments like weathered hillside rock has been considered as rock.

The region outside of the Rhine Graben is further split into zones depending on the surface geology. The surface rock in the North-East is mostly made of Triassic marls, dolomite, sandstone and limestone, whereas the South-East and the South of the zone is mostly showing Jurassic marls and limestone at the surface with some clay layers (Opalinus clay). Small areas with Tertiary sediments are present at the edge of the Rhine Graben, they are developed in the paragraph related to the Graben. Quaternary sediments are well mapped (Fig. 6, GeORG project team, 2013). They exceed 35 m thickness only in the Hardwald forest (station SMZW), North of Pratteln and in small areas in Liestal (up to 70 m locally). They are separated here into Pleistocene, Loess and Holocene sediments as proposed by Fäh et al. (2006c). Pleistocene sediments are mostly made of alluvial terraces of the main rivers and are in general compact. Loess sediments, very soft, can be found on top of hills and were formed by wind transport. Holocene sediments are in general loosely compacted alluvial sediments (gravel and sand).

Inside the Rhine Graben, the Quaternary sediments follow the same classification as outside and are rarely more than 35 m thick except at the top of the Bruderholz and Binningen hills, in the industrial zone between Reinach and Aesch and in the cemetery am Hörmli in Riehen. The Rhine Graben has been divided into 2 subzones: the Loess hills in the South-West and the flat part for reasons detailed in the following sections. The Tertiary deposits (Fig. 6) are mostly marls of lacustrine origin with a degree of consolidation increasing with depth (GeORG project team, 2013) filling the basin with a thickness of 50 to 1000 m in Basel. They are particularly deep in the "Mulde von St-Jakob-Tüllingen", along the Eastern limit of the Graben until Reinach in the South as well as West of the Allschwil fault.

The upper Tertiary (Oligocene age) is divided into the Niederroedern formation and the Froidefontaine formation. They have been heterogeneously characterized in the past on borehole logs (GeORG project team, 2013). The Niederroedern formation can itself be divided into the Tüllinger layers (marls and limestone) with a maximum thickness of 200 m, only present in the Mulde of St-Jakob Tüllingen, and the Molasse alsacienne, made of marl, siltstone, sandstone and limestone of
freshwater origin. They are also covering an area centered on the Mulde von St-Jakob Tüllingen but with a larger extent to the North, the South and the West (see Steimen et al., 2003) with a maximum thickness of 300 m. The Froidefontaine formation (up to 400 m thickness) is made of the formerly called "Séries grises" (Cyrenenmergel, Septarienton – formerly known as Meleta layers or Blauer Letten – Fischschiefer, Foraminifermergel and Meeresand), seawater sediments of Rupelian age, mostly made of mudstone (Septarienton formation). It extends to the whole Graben. It should be noticed that the Fischschiefer and the Foraminifermergel constitute clear reflectors on reflection seismic data but they are relatively thin (Berger et al., 2005; GeORG project team, 2013).

Below the Froidefontaine formation, the lower Tertiary is constituted by the Pechelbronn formation and the Schliengen formation. The Pechelbronn formation (formerly called Sannoisian, Bunte Mergel or Haustein) is a marl of Priabonian age (max. thickness 300 m) and the Schliengen formation (formerly called Siderolithikum, max. thickness 30 m) a limestone of Eocene age. The roof of the Pechelbronn formation has been considered as the geophysical bedrock since the first studies on Basel. However, we show in section 3.3 that the geophysical bedrock is located at the roof of the Mesozoic units.

The upper Mesozoic layers in the graben are made of massive limestone of Oxfordian age.

![Zonation model based on geology built for the project](image)

![E-W geological cross-section of the Basel area through sites Otterbach (OTTER) and Riehen (SRHE). The geophysical bedrock and its depth are displayed in red. Modified from Häring (2006)](image)
3.2 New strong motion network (see map in Fig. 5)

In the frame of this project, the Swiss Strong Motion Network (SSMNet) in Basel has been densified with the installation of 6 new state-of-art free field strong motion stations (SBAM2, SBAJ2, SBAV, SBAW, SRHE, SRHH). Four are located close to school buildings (SBAV, SBAW, SRHE, SRHH) and the two others were replacement of existing old stations. Moreover, 5 additional stations have been replaced since 2010 in Basel in the frame of the SSMNet Renewal project phases 1 and 2 (SBEG, SRER, SBAK, SBIF2, SSCN), SBAK being also located close to a school building. In addition, 5 temporary stations (CHBA, CHBJ, CHBK, CHBM, CHBRI) were installed and ran from August 2014 to September 2015 (Fig. 8). They were fully integrated in the network including the routine and real-time computations (Shakemap, localization of events, empirical spectral modeling (ESM) – see section 3.4). In 2006/2007 a temporary network had also been installed and operated to follow the seismicity induced by the Deep Heat Mining Project. This data has also been made recently available in the SED main archive.

The permanent stations all benefitted from site characterization (e.g. retrieval of 1D velocity profiles) using at least one ambient vibration array measurement (see section 3.3). The information related to the SSMNet stations in Basel, and especially site characterization, is stored in the SED database for a better access and archiving. A site characterization report is available for each station.

The temporary stations recorded 2 to 13 events with a sufficient signal to noise ratio (as defined by Edwards et al., 2013). Station CHBK (Kannenfeld) was located in the noisiest area of the city and therefore recorded only 2 events, CHBM (Schützenmatte) recorded 4, CHBJ (St-Jakob) 6, CHBA (Allschwil) 9 and CHBRI (Riehen) 13. As a comparison, the permanent stations located in the city-centre recorded 2 to 3 of these events. Basically, the network recorded all magnitude 3 events and above, located within the area monitored by SED as listed in Table 2 (events further than 200 km are not considered). For this recording period, the seismicity is considered low, limiting also the significance of the derived amplification functions of the temporary and newly installed stations.
3.3 New knowledge from site characterization in Basel

The SED site characterization database (Fig. 9) contains a large amount of data in the Basel area from single station or array measurements as well as active geophysical measurements from the different projects performed in Basel. Havenith et al. (2007) summarized the knowledge on the geophysical properties in Basel from site characterization performed before and during the microzonation project in 2006. Since then, new data has been gathered after the 2006 geothermal event and during the current project. Site characterization reports are available for each newly installed station on the website http://stations.seismo.ethz.ch. The details of the computations are therefore not developed here, but this section summarizes the major findings about the site properties in Basel. Moreover, nearly all the single stations recordings available in Basel were homogeneously reprocessed using the H/V method (2200 points). All points in the city-center were re-picked including second peaks, when existing, in order to be able to interpret them.
Otterbach site and new reference rock

Besides the newly installed stations, the site Otterbach (SSMNet station OTTER and borehole stations OTER1 and OTER2) has been reassessed in detail. A separate report (Michel et al., 2015) is available on http://stations.seismo.ethz.ch for station OTTER. At this site a sonic logging has been made available by the DHM project.

The reanalysis of these data showed that the results used by Kind (2002) about the bedrock location and velocity, based on a first sonic logging experiment, have to be revised. Moreover, the geophysical bedrock, producing the fundamental frequency between 0.4 and 1 Hz in the basin, is most probably the Mesozoic limestone and not the Sannoisian (Tertiary) marls as assumed up to now. This has been shown by computing the SH transfer function of the velocity profile obtained from the sonic logging experiment, successfully compared to the amplification function derived from earthquake recordings (see section 3.4). Rock layers have been removed gradually from the bottom of the profile. This computation showed that the interface between Tertiary sediments and Mesozoic limestone is responsible for the fundamental frequency of resonance. This is also supported at other sites (e.g. Reinach SRER) by the impossibility to reproduce the fundamental frequency peak with the Sannoisian marl as bedrock. This analysis also permitted to propose a rock velocity model for the 3D model of Basel, similar to the analysis of Fäh et al. (2008) but that has not been implemented yet (Fig. 10). This rock model can be used in future studies on Basel and for improvements of a 3D model, though this measurement is punctual and variations at depth are expected and would need further investigations.
H/V ratios

Nearly all the existing single station recordings in Basel from the database have been reprocessed homogeneously using 5 different codes for the H/V method that were implemented at SED. There are more than 2200 data points (this includes the array points that may be very close to one another) including points that had never been processed. A part has been newly picked, searching for secondary peaks. The results were included in the SED database.

Fäh et al. (2006c) interpreted the H/V ratios in the Rhine Graben as a fundamental frequency corresponding to the geophysical bedrock – assumed to be the Sannoisian marl – with variations of the peak quality attributed to the varying velocity contrast between the sediments (mostly the Meletta layers) and the bedrock. Secondary peaks have been attributed to the interface between the Quaternary cover and the Tertiary "rock", which varying stiffness should have been controlling the amplitude (or the existence) of the peak (Fäh et al., 2006c). The stiffer Molasse Alsacienne (Niederroedern formation), present close to the surface in the eastern part of the Graben and western of the Allschwil fault in the zone Basel-West, but not in the central part, would therefore favor the presence of a second peak. They observed this second peak in the zone Basel West and in a part of Rheingraben West. Fäh et al. (2006c) also studied the second peak outside of the Rhine Graben, in the zone Basel East and in the Ergolz valley.

Our analysis of the H/V ratios shows a complex behavior in the range of the fundamental frequency. Fig. 11 shows the fundamental frequency picked on the H/V curves versus the Mesozoic bedrock depth from the latest available geological model (GeORG project team, 2013). It shows on one hand a good correlation, but on another hand a large variability. It may be due to the variability of the velocity in the Tertiary layers or to uncertainties in the picking of $f_0$ values. Fig. 11 seems to show two lines of points shifted by 200 m depth. This plot should be used in future works to reinterpret homogeneously the H/V data. Though some measurements show a sharp clear peak, most of the results show a broad peak, possibly with small sub-peaks. Fig. 12 shows the average H/V spectral ratios for 4 test stations recording during one week relatively close one another. The H/V ratios are very similar and observed sub-peaks consistent from one measurement to another so that they probably correspond to actual geological features. However, waves at the fundamental frequency have about a 2 km wavelength and therefore average the geological structures over a large volume. The variations of the depth of the Tertiary at these sites, ranging from 730 to 810 m over 750 m horizontal distance is therefore not clearly mapped by the $f_0$ value. The 1D hypothesis fails in this case where the 3D geometry plays a role. A refined analysis and interpretation of the H/V ratios would therefore require a 3D model and another picking strategy as for example using templates.

Similarly, Fig. 13 shows a comparison between the H/V curves at stations SBAW, SBAV, SBAM2, OTTER and SBAJ2, all located in the Rhine Graben with a depth to bedrock of 800, 750, 650, 500 and 425 m. Except the later that is clearly shifted toward higher frequencies, these depth differences are not obvious in the overall H/V ratios. Although the right flank seems to be a good indi-
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cator, the example of OTTER site showed that different measurements at close locations but at different times could give a sharp or a broad peak independently of the computation method (Fig. 14). The right flank and the center of the fundamental peak are different for the 2 measurements. While the location was not exactly the same, such difference could also be due to a change in the composition of the wavefield. In this particular case, the sharp peak corresponds to the observed peak in the ESM amplification function at the same site OTTER, but not the center of the broad peak observed in the close-by array. The common strategy of picking the fundamental frequency in the center of a broad peak appears in this example to be inaccurate, although no better solution can be proposed yet. Anyhow, an uncertainty for $f_0$ has to be defined based on such observations, and this uncertainty has to be taken into account in the analyses.

Fig. 11 Relationship between the fundamental frequency of resonance and the thickness of the Tertiary layers. a) Map view; b) All data; c) data grouped by zones from the microzonation displayed in d). The dashed lines correspond to constant velocities $V_s$ of 400, 800, 1200, 1600 and 2000 m/s assuming $f_0=V_s/(4H)$.

Fig. 12 Comparison between the average H/V ratios (left) over 1 week recording at 4 test stations XBA51 to XBA54 located at close distances (see map on the right). The peak at 1.13 Hz corresponds to a well-known anthropogenic source.
Fig. 13 Comparison between the H/V ratios at the centre of 5 arrays in the Rhine Graben located close to stations SBAW, SBAV, SBAM2, OTTER and SBAJ2, respectively, ordered by decreasing depth to bedrock (from 800 m in SBAW up to 425 m in SBAJ2).

Fig. 14 Comparison between the H/V ratios at the station OTTER in 2005 (red), at the site of the closest array – distance 30 m - in 2004 (blue) and ESM amplification function for station OTTER (black).

Fig. 15 shows the second peak in the H/V curves that has been re-picked for the project. Some high-frequency second peaks (above 10 Hz, white to blue) can be recognized at various places in the basin, with a high variability. They correspond to few meters of ground with a lower S-wave velocity, sometimes anthropogenic infill, for instance around SBAJ2 station in the St Johann park. Consistent lower frequency of second peaks (below 10 Hz, white to red) are found only in the Loess hills in the South of Basel, which can justify the split of the Rhine Graben into 2 zones. The lower frequency is about 2 Hz but is difficult to precisely map because the peak is small and does not clearly appear everywhere. Contrary to Fäh et al. (2006c), we can see these peaks in the whole area, also when the Tertiary layers is composed of Meletta layers. Their frequency value is not correlated with the thickness of the Quaternary as shown on Fig. 16. Moreover, the array analyses available in Basel do not show any noticeable velocity contrast between Tertiary and Quaternary layers.

The shallowest observed contrasts in the velocity profiles from site characterization correspond to few meters of loose soil and another observed contrast occurs in general at about 50 m depth within the Meletta layers between weathered and sound rock (Havenith et al., 2007). Reprocessing the array close to SBIS2, showing this second peak at about 2 Hz showed similar profiles with the same kind of contrast. At this location, the upper 35 m show the lowest velocities in Basel but remain at about 400 m/s (note that the Quaternary is about 7 m thick at this site). The importance of these second peaks is developed in section 3.4.

As a conclusion, the H/V ratios are a powerful tool but, in the case of the Rhine Graben, are still difficult to interpret in terms of depth of layers and velocity contrasts because they are related to a 3D geometry. At the edge and outside of the Graben where the fundamental frequencies increase, the H/V curves are much easier to interpret. The large dataset of single station measurements in Basel is therefore still a source of exciting new developments in this field.
The interpretation of several array measurements (new and existing) has been made difficult by the fact that Love waves dispersion curves were found at a much higher velocity than predicted by simple models. This feature is present at sites SBAV, SBAW, SBAM2, SRER, Kannenfeld (CHBK) or SBIF2. Fäh et al. (2008) proposed a shallow layer (gravels at 15 m depth) with high velocities (1200 m/s) and a velocity inversion (weathered Meletta layer) below this layer to explain the mismatch between Love and Rayleigh curves at site Kannenfeld. However, since this feature can be observed on a large area, it might not be linked to the very variable surface sediments, but to deeper structures (in the order of hundred meters) or to characteristics of the wavefield.

It can be noted that the quality of the results on the transverse (Love) component is generally low. For the old measurements, it is partly due to a too short recording time. Another hypothesis would therefore be a misinterpretation of Love dispersion and mode mixing. In several cases in Basel, it has been observed that the fundamental modes, particularly of Love waves, were not excited under ambient vibrations at low frequencies.

Though this constitutes the most likely hypothesis, no satisfying results were found when the retrieved Love mode was assumed as first higher mode. Love dispersion curves had to be generally discarded in its largest part. As proposed by Fäh et al. (2008), more complex velocity profiles including low velocity zones may help explaining the mismatch. The issue becomes then that the measurements are not sufficiently constraining such low velocity zones.

Finally, another hypothesis can be mentioned: anisotropy in the Froidefontaine formation. Transverse-isotropy (TIV) (difference of the horizontal and vertical wave velocities) is associated with layered media (deposited by gravity) such as shale (Opalinus clay is a well known example in...
Switzerland). In such media, the horizontal velocity is generally faster than the vertical. This difference can reach 10-25% percent, and could explain the observed difference. If this issue is of great theoretical interest, practically the effect of the differences in the velocity profiles on the amplification is limited.

**Changes for the 3D model**

In the frame of the project, no numerical modeling was performed. However, we propose in the following, improvements to implement in the 3D numerical model of Basel. Fäh et al. (2008) noticed different issues with the numerical model that we consider here. They wondered about the deep velocity model and suggest to add layers at depth, which is confirmed by our study. This work should be based on the interpreted Otterbach sonic logging.

In the sediments, they wondered about the velocity of the lower part of the Meletta layers. Actually, 2 strong velocity contrasts are consistently retrieved in the Rhine Graben in the upper part of the model: at 50/100 meters (“weathered” Meletta layers) and the geophysical bedrock (roof of the Mesozoic rock). None of these layers are present in the 3D model and they should be introduced. The interface with the Mesozoic has been well defined (GeORG project team, 2013), the interface within the Meletta layers has to be interpolated from the velocity profiles, which may be more difficult. A detailed correlation of retrieved velocity profiles should be performed in order to update the velocity model accordingly.

Fäh et al (2008) also propose a better mapping of the surface sediments. Although the depth of the Quaternary is now well defined (GeORG project team, 2013), loose sediments creating high-frequency resonances are not correlated with the geology in the Rhine Graben. This would need further investigation in particular in combination with the composition of the Quaternary soils. The opposite is true outside of the Rhine Graben where $f_0$ correlates with layer thickness. The modeling could therefore be greatly improved using these newly available data.

### 3.4 New knowledge from earthquake recordings in Basel

**Empirical Spectral Modeling (ESM)**

SED is routinely modeling the Fourier spectra of recorded earthquakes at each station of the network. All signals with sufficient signal to noise ratio are then used to retrieve information on the source (Moment Magnitude, stress drop). The residuals of the modeling are used to retrieve the elastic amplification function of each site (Edwards et al., 2013). In addition, the anelastic attenuation $\kappa$ is retrieved as well so that the amplification function includes the anelastic attenuation. The reference of these amplification functions is the Swiss Reference rock model (Poggi et al., 2011).

Edwards et al. (2013) showed that the ESM functions were reliable. However, when stations have few recordings only, the results may be biased: the shape of the function is well retrieved with few events, but the absolute value of the amplitude (hereafter called DC) may be uncertain if the recorded events are not well distributed. This uncertainty is not taken into account in the standard deviation. During the estimation procedure, a trade-off with $\kappa$ makes the results sometimes unstable if only few events are available. As a result, the high-frequency part, dominated by $\kappa$, may be particularly biased in this case.

The ESM amplification functions in response spectra are routinely computed from the Fourier ESM functions with anelastic attenuation using Random Vibration theory (RVT) for a scenario of magnitude 5 at 50 km. Tests showed that this computation was not much sensitive to the scenario used.

The DC value (absolute amplification level) of Basel stations SBAP, OTTER, SBAT, SRHB and SMZW are fixed in the ESM code and are part of the definition of the Swiss Reference Model. However, they have been fixed based on earlier versions of the code and sometimes on few events only. Therefore, we computed the *a posteriori* ln(DC) correction for each event and fixed stations...
in Basel to check the fixed values. This correction is uncertain and may depend on the epicentral distance of the earthquake as well (modeling bias). Moreover, in Basel, many recorded events are part of the induced seismicity of the Deep Heat Mining Project (geothermal project) and are therefore recorded in very near field (about 5 km from the rupture). Events of this sequence are at about the same location with about the same source mechanism and therefore similar radiation of energy at a given station. This breaks the assumption of the computation of the ESM amplification that does not include source radiation pattern and assumes a homogeneous distribution of events to ensure no bias due to source effects.

Fig. 17 shows the obtained ln(DC) corrections in order to detect possible biases. They are normally distributed but show several outliers that affect the mean. These outliers are for intermediate epicentral distances and correspond to imprecisely modeled events (effect of a fixed geometrical spreading term for the entire Swiss foreland). Mean and median are closer when outliers are removed (ln(DC)>0.91). The only station with non-normally distributed ln(DC) is OTTER, located at the epicenter of the geothermal events, showing that the radiation patterns of the induced events has an impact for this station. Therefore, only the natural events are used in the following to define the correction values. Finally, the correction values (bias) computed for the amplification at these stations are the mean of all natural events without outliers. The obtained correction factors to DC are summarized in Tab. 3.

<table>
<thead>
<tr>
<th>Station</th>
<th>DC Correction</th>
</tr>
</thead>
<tbody>
<tr>
<td>SBAP</td>
<td>0.8829</td>
</tr>
<tr>
<td>OTTER</td>
<td>1.362</td>
</tr>
<tr>
<td>SBAT</td>
<td>1.0418</td>
</tr>
<tr>
<td>SRHB</td>
<td>1.0697</td>
</tr>
<tr>
<td>SMZW</td>
<td>1.0231</td>
</tr>
</tbody>
</table>

The only large correction (~35%) is for station OTTER, the closest to the geothermal events. Station SBAP shows a correction of more than 10%, which is not a large value when considering the uncertainties and variability of the amplification function. The others are 5% or less indicating that the values were correctly set in Poggi et al. (2011).
Fig. 17 DC correction factors for fixed stations in the ESM computation. Top: correction factors as a function of distance; Bottom: distribution of correction factors for natural and induced (geothermal) events.

In order to validate the ESM amplification functions, they are compared to standard spectral ratios (SSR in Fourier spectra) with respect to station SRHB inside the Rhine Graben (Fig. 18) and SKAF outside (Fig. 19). It should be noticed that less events are available for the couples of stations used to computed the SSR than for the ESM of single stations. Reliable SSRs cannot be computed for all stations and only stations with enough events are displayed. The match is very good for many stations, including those not presented here. The DC correction proposed above (applied here) improves the results, especially for OTTER station. For some stations (e.g. OTTER), although signal to noise ratio is carefully checked, the SSR may include some noise at low frequency, inducing a bias compared to the ESM function. Comparing only with the larger magnitude events leads to a convergence between ESM and SSR at low frequency (not presented).

It should be noticed that the SSRs between stations outside of the Rhine Graben and station SKAF do not reach 1 below the fundamental frequency but are approximately 2 at stations SRHH, SBEG or SFRA for example. It could mean that the rock profile these stations is different. Therefore, there can be differences in the amplification due to the characteristics of the underlying rock, which could be investigated in further studies.

The SSRs and the ESM cannot be easily compared in response spectrum since, at low frequency, the response spectrum of earthquake recordings is influenced by noise at even lower frequencies. However, above 1 Hz where this effect can be neglected, such comparison also shows a very good agreement (not presented here).

As a conclusion, the ESM functions, even though sometimes based on few earthquakes, are reliably expressing the amplification at each SSMNet station. Biases due to the distribution of events can however still be present in both ESM and SSR and will be identified with an increased number of recordings.
Fig. 18 Comparison of standard spectral ratios (SSR, grey lines and grey shading) and ESM functions (black lines), with their respective standard deviations, for stations in the Rhine Graben with respect to SRHB. The stations are: CHBAL, OTTER, SBAF, SBAP, SBAT, SBIS2 and SRER (from left to right and top to bottom).

Fig. 19 Comparison of standard spectral ratios and ESM functions, with their respective standard deviations, for stations outside of the Rhine Graben with respect to SKAF. The stations are: CHBDO, CHBMU, CHBPF, CHBRI, SAUR, SBEG, SFRA, SMZW and SRHH (from left to right and top to bottom).

Verification of the microzonation

Fäh and Havenith (2006) derived amplification functions in response spectra for each zone of the microzonation study. The reference of these curves has been changed to the Swiss Reference rock profile (Poggi et al., 2011) by computing the correction from the impedance contrast between the
Basel 2006 and the Swiss reference velocity profiles expressed as the ratio of their quarter-wavelength velocities. This procedure does not account for differences in the anelastic attenuation. These curves are compared to the ESM amplification function of stations located in each zone, if any (in the Rhine Graben in Fig. 20 and outside in Fig. 21.). It should be reminded that the microzonation aims at defining the envelope of possible amplifications in a given zone.

West of the Allschwil fault, the zone Basel West (Loess hills) shows a relatively flat amplification function in the microzonation with an amplitude of 3 (Fig. 20). Station CHBAL shows similar amplitudes though the effect of the second peak in the H/V ratios at 4 Hz induces a peak that is not reproduced in the amplification function proposed for the microzonation. The zone Basel Nord is hosting a large number of SSMNet stations (Fig. 20). Its amplification function shows a peak around 1 Hz. The difference between the subzones Holocene and Pleistocene occurs above 2 Hz and is limited. Although the amplitudes match with the observations, a large variability is observed. The sub-zones are not reproducing an observed difference and are probably not relevant. Station CHBZ2 is considered as unreliable (only one recording).

In the zone Rheingraben West, the amplification of the microzonation is well matching with the ESM function of station SBIS2 (Fig. 20). The second peak at 2 Hz is reproduced. The amplitude of station CHBBO is very high and therefore not covered by the microzonation. We showed that this high values have been really observed during the geothermal sequence, but they may have been influenced by the source radiation of the induced earthquakes and is therefore not considered as reliable. The secondary peak amplification (2-10 Hz) of CHBA2 is also not reproduced. The eastern part of the Rhine Graben (zone Rheingraben Ost) shows an amplification function in the microzonation with amplitudes around 3 (Fig. 20). The difference in the amplification between the subzones Holocene and Pleistocene is not significant as for the observed ESM functions. Most of the stations in the zone show similar values, decreasing with frequency, with little variability. Stations CHBNM and CHBZ1 (located in a 4-story building) were only operation in 2006/07 and are considered as unreliable: they recorded only geothermal events and are therefore most probably influenced by the source radiation of the induced earthquakes. Station SBIF2, located at the edge of the basin, shows a slightly different behavior with lower amplification over a broad frequency range.

Outside of the Rhine Graben, the zone Flexur Nord is represented by an amplification function increasing with frequency, with a resonance around 4 Hz (Fig. 21). The subzone Loess shows higher amplitude around 1 Hz. Although the variability in this zone is particularly large, the observations tend to confirm the function chosen in the microzonation. The zone Flexur Sued is represented by an amplification function with a ramp up to 1.25 Hz and then a plateau (Fig. 21). The observed ESM are much more variable in terms of resonance frequency and amplitude; however, again, these stations were in place only in 2006/07. Considering the number and nature of recorded events, stations CHBMU and CHBPF are therefore considered as unreliable.

The zone Basel Ost displays an amplification function that matches particularly well up to 4 Hz with the observations at the different stations, though the peak value is underestimated (Fig. 21). In reality a large variability in the thickness of the Quaternary layers let us expect a large variability of amplification. The rock subzone Basel Ost HF (Fig. 21) overestimates the amplification.

In the zone Ergolz Nord, variability is also expected so that the amplification from the microzonation does not match the observation at the available station (Fig. 21). Station SFRA is installed in a transformer house that may influence the recorded ground-motion at high frequency.

As a conclusion, the microzonation is well representing the observed amplifications, particularly in the Rhine Graben where they are relatively homogeneous. Some high-frequency amplifications, for instance in the Loess hills in the South of Basel, are however not reproduced. Outside of the Rhine Graben, a larger variability is expected and observed, so that the microzonation is not expected to model the amplification of a particular site. In this area, a single amplification per zone is not enough for our purpose due to the variability of the fundamental frequency of resonance. Today, the number of recordings at some of the strong-motion stations is not sufficient to produce reliable statistics; this will however improve with time.
Fig. 20 Comparison of the PSA amplification in each zone of the 2006 microzonation inside the Rhine Graben and ESM functions of station located in the corresponding zones. Subscripts P (Pleistocene), H (Holocene) and L (Loess) denote the subzones.

Fig. 21 Comparison of the PSA amplification in each zone of the 2006 microzonation outside of the Rhine Graben and ESM functions of station located in the corresponding zones. Subscripts P (Pleistocene), H (Holocene) and L (Loess) denote the subzones. All available stations are displayed including those with amplification functions regarded as unreliable (see text).

Explaining amplification with 1D S-wave velocity profiles from site characterization

This section summarizes how much we understand from the observed amplification using the site characterization tools as detailed in section 3.3. We distinguish in the following the Rhine Graben with very deep sediments and many stations and outside of the Rhine Graben, with alluvial deposits as well as weathered rock on hill sites with strong lateral heterogeneities.

The amplification in the Rhine Graben is mostly controlled by the deep sediments and varies smoothly. The overall match between 1D SH transfer function from site characterization and ESM amplification is fair in the Rhine Graben (Fig. 22 – SRHH is not located in the Rhine Graben). It is
however clear that the 1D transfer function is too simplistic: a 3D model is necessary to improve the match taking into account local surface waves, focusing and defocusing effects. The depth of the sediments, increasing to the East (trough of St-Jakob Tüllingen), controls the fundamental frequency, between 0.4 and 1 Hz, though the ESM function is not available below 0.6 Hz due to a lack of recordings of large earthquakes. This resonance frequency plays anyway a marginal role in the amplification at the frequencies of interest in our project (we are interested in the range of the building frequencies), except at the edges of the basin. At 1 Hz (Fig. 23), the observed amplification shows no correlation with the resonance frequency obtained from H/V ratios. The ESM amplification is smoother than the 1D SH transfer function probably indicating 2D or 3D effects (Michel et al., 2014b). A manual grouping of the station amplifications (Fig. 24) shows significantly lower amplifications below 2 Hz in the North-Western part of Basel including the Rhine-Wiese confluence and the Kannenfeld area compared to the rest of the Rhine Graben. Between 2 and 6 Hz, the area with the lowest amplification is the Kannenfeld area. These small spatial variations are hardly reproduced by the 1D velocity profiles from site characterization and therefore not explained.

A noticeable exception is observed at site Bruderholz (SBIS2) that shows large amplifications with a second peak at 2 Hz and to a lesser extent at sites CHBAL and CHBA2, also located on the Loess hills. This peak can be clearly observed in the H/V as a second peak in the South of Basel (Loess hill - see previous section). It is reproduced in the microzonation by the 2D model only (envelope, not mean) and not by the 1D SH transfer function. The site characterization shows lower S-wave velocity values in the first 35 m at site SBIS2 compared to the North of Basel but the difference remains limited (at 30 m depth: 430 m/s at SBIS2, 500 m/s at SBAM2, SBAV, SBAW, SBAP, 600 m/s at OTTER). The second peak at site CHBAL is also not reproduced by 1D site characterization (site Allschwil 1, Havenith et al., 2007). Without a more reliable 3D model, we have therefore to rely on the empirical observation that the amplification is larger in the Loess hills, based on homogeneous H/V curves and ESM. We therefore separated the Loess hills from the rest of the Rhine Graben. However, the behavior of the Loess hills remains poorly controlled compared to the rest of the Graben.

High-frequency resonances (10 Hz and above) can be found on the ESM and are generally captured by the site characterization. Their spatial variation is however hardly mapped even with single station measurements.

Fig. 22 ESM amplification of stations installed during the project compared to the SH transfer function from site characterization. From left to right and top to bottom: SBAJ2, SBAM2, SBAV, SBAW, SRHE and SRHH.
Out of the Rhine Graben, the depth of sediments (Quaternary) is generally limited though the resonance frequency can go down to 1 Hz in some areas (Fig. 25). The assumption of a single layer over the bedrock can be made there so that 1D site characterization is generally well explaining the amplification (available only at SBEG – Michel et al., 2014 –, SMZW and SRHH, Fig. 22). Site-response is then controlled by the thickness of the sediments and their S-wave velocity. Three types are recognized and mapped from Swisstopo geological maps and sub-zone maps of the microzonation: Pleistocene, corresponding to alluvial terraces with relatively stiff material, Holocene, corresponding to young and loose alluvial sediments and Loess corresponding to very soft sediments deposited by wind (Fig. 5). This categorization is an oversimplification of the reality. Moreover, in our study, weathered hillsides were considered as rock.

Cadet et al. (2012) proposed a generic model called SAPE for amplification functions based on $f_0$, $V_s$ or any combination of these parameters, based on Japanese sites. We compared their model using $f_0$ only to our observations from ESM. For that purpose, we normalized the frequency in the ESM functions by the fundamental frequency of the site (Fig. 26). Another source of data are the amplification curves from the microzonation for zones outside of the Rhine Graben, corrected for the Swiss reference. Since $f_0$ is not considered for these zones, the first peak in the amplification function has been picked and considered as the fundamental frequency. When normalized by the fundamental frequency, the shapes of the obtained functions are comparable.
Amplification of ground motion in Basel Earthquake Risk Mitigation

The peak value of the ESM functions is variable though not clearly linked to any other observable such as the peak amplitude in the H/V ratios or $f_0$. A dependency with the sediment type and underlying rock could be developed in future studies. The microzonation and the SAPE function show lower amplitudes than the ESM and a smoother function. The SAPE model depends on $f_0$ with decreasing amplitude at the peak with increasing $f_0$, while we observe the opposite. The amplitudes after the peak are generally lower for the microzonation than for the ESM functions. In the ESM function, the amplification after the peak is controlled by the anelastic attenuation $\kappa$. Tab. 4 shows the observed values of $\kappa$ using ESM. They are negative indicating a lower attenuation compared to the Swiss Reference rock that corresponds therefore to amplification as expected in shallow sediments. Conversely, $\kappa$ values in the Rhine Graben are generally positive and correspond to attenuation with respect to the reference. However, the uncertainty has the same order of magnitude as the mean (Edwards et al., 2015b). The observed variability in the ESM function after the peak is therefore probably more related to this uncertainty than on an actual physical behavior.

**Tab. 4** $\kappa$ values retrieved from ESM outside of the Rhine Graben

<table>
<thead>
<tr>
<th>Station</th>
<th>$\kappa$</th>
<th>Uncertainty</th>
</tr>
</thead>
<tbody>
<tr>
<td>SBEG</td>
<td>-0.030</td>
<td>0.023</td>
</tr>
<tr>
<td>SAUR</td>
<td>-0.008</td>
<td>0.016</td>
</tr>
<tr>
<td>SMZW</td>
<td>-0.017</td>
<td>0.017</td>
</tr>
<tr>
<td>SFRA</td>
<td>-0.018</td>
<td>0.021</td>
</tr>
<tr>
<td>SRHH</td>
<td>-0.036</td>
<td>0.030</td>
</tr>
</tbody>
</table>
A “generic amplification function” based on the average of the available ESM amplification functions outside of the Rhine Graben is therefore proposed (Fig. 26). It is extended at low frequency using the curve from the microzonation. This function can be used in combination with the fundamental frequency of the site to predict the amplification at any frequency (see next section).

### 3.5 Computation of amplification maps

In order to retrieve the amplification on a dense grid of points, three types of techniques are available and can be combined: zoning, interpolation and modeling. Interpolation allows to densify the number of points with an amplification value, while modeling allows to transform a set of data into other parameters and ultimately into amplification. The observed amplification (ESM) in Basel is available for a limited number of points (typically 20). A smoothly varying process such as the amplification at low frequency in the Rhine Graben can therefore be directly interpolated from these observations. Other types of available data are the H/V curves (in the order of 1000 points) and velocity profiles from site characterization (about 20 sites). For the 2006 microzonation, these profiles combined with geological data have been used to build (interpolate) a 3D velocity model, later used to compute (model) the amplification. However, the final results were based on zoning (one curve per zone) and no interpolation. For the present project, we proposed another combination of these techniques in order to take the most of the available data.

As explained before, the objective is to compute the amplification functions in PSA at the frequencies of the structures for the school buildings on one hand and for the whole city and Shakemap software on the other hand.

The amplification functions developed in the frame of the microzonation study are still considered to be valid and therefore used as an alternative during the project. Producing different amplification maps and using them for risk assessment allows to account for epistemic uncertainties. We changed the reference of these curves to the Swiss Reference Rock profile (Poggi et al., 2011) as described above. It should be noticed that the microzonation is available in a limited area, for instance not outside the Swiss border, where a default value of 1 is displayed (Fig. 27).

### Interpolation methods

The available spatial interpolation methods are: nearest neighbor method, inverse distance weighting (IDW), spline interpolation technique and geostatistics. Spline interpolation requires a large number of data points smoothly varying and is therefore not adapted to this problem. Geostatistics requires also a large number of data points to infer their statistical distribution. It is not adapted to the interpolation of observed amplification, but could be used in future projects for H/V curves. The nearest neighbor method is a simple but robust method that separates the space into Voronoi cells around the known points and assigns one value per cell. IDW allows a simple and smooth interpolation based on the values at a given number on neighbors weighted by their distance. However, depending on the weighting function and the distribution of available data, it may create unrealistic artifacts. Anisotropy of the weights can be introduced to account for the geology (for a given point, the weight of stations located parallel to the direction of the Graben could be larger than those located perpendicularly). The power p used to weight with respect to distance can be modulated. A large value (16) simulates the nearest neighbor method. A value of 2 is used here to avoid having too much weight from points located too far.

### Interpolation in the Rhine Graben

We showed in the previous sections that the average amplification was not changing much in the Rhine Graben where many stations are available. Therefore, we performed a spatial interpolation of the station amplification in the Rhine Graben. We tried to interpolate using nearest-neighbor and IDW (with and without anisotropy) techniques. We finally did not use anisotropy because evidences of such anisotropy in the direction of the Rhine Graben are lacking. IDW was also rejected because it was creating artefacts in the map and we decided for the robust nearest neighbor
method. In order to keep flexibility in the code, the nearest neighbor method is implemented using IDW with large value of $p$.

We also showed that the Loess hills in the South and the West of Basel were constituting an exception (secondary peaks, higher amplification) so that this zone is treated separately. Only 3 stations with very different amplification functions are available in this zone so the interpolation is very uncertain. The characterization of this area should be improved in the future. Concerning the scenarios, only the 5 buildings of school Bruderholz are located in this area close to station SBIS2, with a resonance frequency of the buildings of 2 Hz. They will therefore experience a noticeably higher ground motion than the other school buildings.

**Interpolation outside of the Rhine Graben**

Only two school buildings are located outside of the Rhine Graben, each of them is very close to a SSMNet station: the Zur Hoffnung school, where the station SRHH is installed and the Bettingen school, located 200 m away from the SBEG station, on the same geology. Therefore, for the scenarios, the ESM functions of these stations are used directly as proxy for the amplification at the site.

However, a more general amplification model is needed for the Shakemap calculation and the extension of the scenarios to the whole building stock. We therefore used the generic amplification function developed in section 3.4 together with the available $f_0$ values in zones with sediments outside of the Rhine Graben. The obtained amplification value at each available $f_0$ value is interpolated over the other points of interest using the IDW algorithm.

The final amplification maps are displayed in Fig. 27. The amplification at 0.3 s is large outside of the Rhine Graben and low inside and vice versa at 1 s. They show similar orders of magnitude compared to the microzonation. The Loess hills in Bruderholz have a larger amplitude at 0.3 Hz. The Rhine valley outside of the Rhine Graben shows larger values than the microzonation at 0.3 s but lower at 1 s. The interpolated map extends also to the right riverside of the Rhine (Germany) where data is also available. Towards France, however, the results are extrapolated.

![Amplification maps in Basel in terms of spectral acceleration at 0.3 s (left) and 1 s (right) based on the 2006 microzonation (top) and the new interpolation (bottom). The color scales are identical.](image-url)
Towards a detailed Shakemap for Basel (written with C. Cauzzi)

Shakemap is a software provided by USGS to compute in near-realtime the ground motion produced by an earthquake (Wald et al., 1999; Worden et al., 2010). Cauzzi et al. (2015) detailed the computation framework for Switzerland, based on the Swiss stochastic model of Edwards and Fäh (2013) and an amplification map based on macroseismic intensity. In the framework of the Basel project, a high-resolution instance of the software focused on the Basel area is proposed. The amplification map is computed following the strategies explained above for a 40 m grid size. Fig. 28 shows the high-resolution predictive scenario for the Basel 1356 (Mw6.6) event generated using ShakeMap. The shaking parameter shown in the picture is macroseismic intensity converted from 5%-damped pseudo-spectral acceleration PSA(T=0.3s) based on Faenza and Michelini (2011). PSA levels in the city area on rock reference conditions (Poggi et al., 2011) are those given by the predictive model of Edwards and Fäh (2013) for the Swiss foreland region (Cauzzi et al., 2015). Amplification at T = 0.3 s due to local site condition was derived in this study and was applied in the scenario as a multiplicative factor of the PSA values on rock. Predicted intensity reaches degree IX as reported in Fäh et al. (2009).

The conversion from PSA to Intensity of Faenza and Michelini (2010,2011) is available at 0, 0.3, 1 and 2 s. However, for small events (typically magnitude 3), the corner frequency is rather large so that the spectral acceleration of the ground motion at 1 or 2 s is not representative of the macroseismic intensity. Even though 0.3 s may also be too long for the smallest earthquakes, this period is the best compromise for this computation.

The GMICE can be written as follows (Faenza and Michellini, 2011):

\[
I = 1.24 + 2.47 \times \log_{10}(100 \times Sa(0.3s))
\]  

Then, the intensity increment can be computed based on the amplification of the spectral acceleration a 0.3 s (\(Amp(0.3s)\)) as follows:

\[
\Delta I = 2.47 \times \log_{10}\left(\frac{Sa(0.3s)}{Sa_{ref}(0.3s)}\right) = 2.47 \times \log_{10}(Amp(0.3s))
\]  

Fig. 29 shows the obtained amplification with the new interpolation compared to the currently used map, based on macroseismic data and surface geology (Cauzzi et al., 2015). The amplification on rock is significantly reduced and the Rhine Graben is more homogeneous compared to the current map.
Conclusions

As a conclusion, the recordings from the strong motion network in Basel allowed to improve our understanding of the ground-motion amplification in Basel. Still only few ground-motion recordings exist for some of the strong-motion stations, but it will improve with time and new recordings. We pointed out the most important geological features that play a role in this phenomenon: in the Rhine Graben the interface with the Mesozoic bedrock, the interface above 100 m depth between “weathered” and compacted Tertiary layers and 2D/3D effects; outside of the Rhine Graben, the depth of the Quaternary sediments and their material properties. Compared to the 2006 microzonation, more extreme values have been identified and spatial variability of the amplification, not investigated in the microzonation, have been evidenced. Hence, we showed that installing strong motion stations is a need to validate a microzonation study. In Basel, the spatial coverage of stations is very dense in the city centre but could be largely improved on the Loess Hills in the South. Modern techniques to analyze ambient-vibration array data might also help to fill this gap in future studies. We produced amplification maps for use in the scenario computation and in a high resolution Shakemap that can work in real-time.
4. Derivation of fragility curves

For a given set of damage grades (DG), fragility curves are defined as the probability of exceeding each DG as a function of a given Intensity Measure (IM). An IM is a single parameter characterizing the ground motion such as the Peak Ground Acceleration (PGA) or the spectral acceleration at a given period (SA(T)). In several other projects, instead of using fragility curves defined by IMs, researchers were using Engineering Demand Parameters (EDP), characterizing the building response – not directly the ground motion (e.g. HAZUS, Risk-UE project). It was therefore necessary to compute first the buildings response from the ground motion before being able to compute the damage distribution through the fragility curves. Other researchers pointed out that this method was unpractical for risk computations and therefore suggested to include the computation of the building response in the fragility curve derivation (e.g. Michel et al., 2009; Crowley, 2014), resulting in fragility curves as a function of IMs. This standard is followed in the GEM project and therefore in Openquake software.

In this project, the building response is computed using different non-linear static procedures (no time-history analysis is performed). The behavior of buildings is modeled by its so-called “capacity curve”, a force-displacement model (see Resonance, 2015).

In the following, the basics about the capacity curves are recalled, the damage scale is defined and two different methods to derive fragility curves are detailed: a simple one based on the Risk-UE LM2 method (Lagomarsino and Giovinazzi, 2006) and a new method developed during the project, allowing to propagate the uncertainties. The results are then presented and the methods compared. They are further analyzed in section 5. These methods have been implemented in a Matlab toolbox to handle fragility curves.

4.1 Capacity curves

The capacity curves are representing the force-displacement behavior of the structure. It can conveniently be represented in the spectral acceleration/displacement ($S_e/S_d$) plane by simply dividing the y-axis (the force) by the equivalent mass of the system $m^*$. They are computed by Resonance for the different building types (Resonance, 2015). In this project, the capacity curves are supposed to be pure elastoplastic models, i.e. made of 2 straight lines: the elastic part until the yield point with a slope related to the elastic period $T$ and a horizontal line at the yield acceleration of the structure until its ultimate point. Therefore, one curve is defined by 3 parameters: the elastic period $T$, the yield displacement $d_y$ and the ultimate displacement $d_u$. The yield acceleration is defined as $a_y = d_y * 4 * \pi^2 / T^2$. Besides purely mechanical considerations, the elastic period has been further constrained in the project by ambient vibration measurements in buildings (not presented in this report).

The response spectrum of the input motion can also be represented in the $S_e/S_d$ plane (ADRS format), which is further used to compute the buildings response.

4.2 Damage scale

The EMS98 damage scale is used in this project (Grünthal et al., 1998). It has 5 grades: slight, moderate, severe, partial collapse and complete collapse, defined from the description of building damage. The corresponding limit states of the model have been defined by Resonance in displacement capacity from the capacity curve (Resonance, 2015): DG1 corresponds to a displacement exceeding $0.7 * d_y$, DG2 to $1.5 * d_y$, DG3 to $0.5 * (1.5 * d_y + d_u)$ and DG4 to $d_u$. These limit states implicitly define a 6th damage state: DG0 (no damage). This corresponds to the limits defined by Lagomarsino and Giovinazzi (2006) except for DG3 that is set in the centre between DG2 and DG4 instead of the centre between $d_y$ and $d_u$.

In order to avoid double counting of uncertainties, it should be noticed that these values are definitions and therefore not subjected to uncertainties, though the corresponding transcription to observed damages or loss ratios is subjected to a large uncertainty.
4.3 Computation of the structural response

In order to derive fragility curves, the set of displacement thresholds (plastic displacements of the structure) corresponding to the set of damage grades should be related to the chosen intensity measure (IM) of the ground motion. The forward problem is here to compute the response of a structure having a given capacity curve to a given ground motion. The inverse problem to solve is to compute for which level of ground motion each damage grade is exceeded. Several methods exist to solve the direct problem. Non-linear time history analysis can be used once the capacity curve is turned into a hysteretic model that provides the force/displacement relationship in the dynamic case (whereas the capacity curve is static). Other methods are called linearization methods because they relate the non-linear system to an equivalent linear system. Michel et al. (2014) review the different families of linearization methods. Two different linearization methods are used in the following (N2 method, Fajfar, 2000, and Lin and Miranda, 2008, method).

Using such a non-linear static procedure makes natural the choice of an IM based on the pseudo spectral acceleration (or displacement). Other IMs such as the PGV, the Arias Intensity etc. are rather inconvenient because they are not used in the model. Moreover, the chosen IM should preferably be provided by the used GMPEs. In this project, the pseudo spectral acceleration at the elastic period of the structure $SA(T)$ for a 5% damping is used, though alternatives exist in the literature (such as an average of the PSA over a given period range, also called Housner Intensity). One can notice that the 5% damping is the usual value, although it has no solid basis.

4.4 Simple derivation method based on Risk-UE LM2 method (N2 method)

The first method used in the project, was proposed by Lagomarsino and Giovinazzi (2006) and is known as Risk-UE LM2 method. It uses the EC8 linearization method (or N2 method, Fajfar, 2000; CEN, 2004) to compute the building response (Fig. 30). This method assumes that long period structures (Fig. 30a) have the same maximal displacement in their inelastic range of behavior $d^*_{el}$ as the one of an elastic system with the same period $T^*$ ($d^*_{el}$). This non obvious principle is called the equal displacement principle and was proposed by Newmark in the 1960s. For short period structures (Fig. 30b), however, the maximum displacement in the inelastic range $d^*_y$ is larger than the one of the equivalent elastic system $d^*_y$ that is, by definition, the spectral displacement of the input ground motion). An empirical relationship has been derived and is used in the EC8 (CEN, 2004) to compute this maximum displacement, knowing the displacement of the equivalent elastic system. The threshold between short and long period systems is set at the corner period of the Newmark spectrum $T_c$ used in the design codes.

$$\begin{align*}
\forall T^* \geq T_c \quad & d^*_y = d^*_{el} \\
\forall T^* < T_c \quad & d^*_y = \frac{d^*_{el}}{R} \left(1 + (R - 1) \frac{T_c}{T^*}\right)
\end{align*}$$

with $R$ the so-called reduction factor that measures the level of non-linearity of the system and $d^*_y$ the yield displacement from the capacity curve.

$\textbf{Fig. 30}$ Computation of the response ($d^*_y$) of a building with a given capacity curve to an event with a given elastic spectral displacement ($d^*_{el}$) following the EC8 method. For a long period system (a), the inelastic and elastic maximal displacements ($d^*_y$, and $d^*_{el}$ respectively) are assumed to be equal (equal displacement principle). For a short period system (b), the equivalent elastic...
system has the displacement $d^*_m$ that is smaller than the inelastic one $d^*_e$. The computation of $d^*_e$, as a function of $d^*_m$, is done through an empirical formula.

Using this simple empirical relationship, the inverse problem is easily solved by reversing the equation. We assume that the fragility curve median $\mu_i$ for each damage grade $i$ (from 1 to 4 or 5) in terms of spectral displacement at the period of the structure $S_d(T)$ corresponds to the deterministic displacement corresponding to each damage grade. They are therefore directly computed using this model as follows:

$$\forall i, \begin{cases} T^* \geq T_e \cup d^*_i \leq d^*_y \quad \mu_i[S_d(T)] = d^*_i = d^*_{et} = d^*_{DGi} \\ T^* < T_e \cap d^*_i > d^*_y \quad \mu_i[S_d(T)] = d^*_i = d^*_i\left(1 + \left(\frac{d^*_{DGi}}{d^*_y} - 1\right)\frac{T^*}{T_e}\right) \end{cases}$$

with $d^*_{DGi}$ the displacement corresponding to damage grade $i$, defined in the capacity curve. The first case in the equation groups all the cases where the displacement did not reach the non-linear domain (damage grades 1 and 2) and the cases of long period systems for all damage grades, whereas the second case is for damage grades 3, 4 and 5 of short period systems.

The only parameter depending on the ground motion in this model is $T_e$. $T_e$ depends mostly on the magnitude of the earthquake and the soil conditions. According to the microzonation of Basel, this value should be between 0.2 and 0.4 s. The latter has been chosen through this report since it is a standard value also in EC8.

In order to derive a distribution (not only a median value) of the spectral acceleration or displacement for which each damage grade is exceeded, Lagomarsino and Giovinazzi (2006) proposed to use a lognormal distribution with a lognormal standard deviation of 0.4 ln(μ) (where $\mu = d_e/d_s$ is the ductility).

Following this method, the fragility curves for each type of buildings are computed based on the median capacity curve provided by Resonance (2015).

This method is simple and has been used for several applications in Europe. However, the uncertainties are not transparent and one cannot know what is covered by the chosen value. There is therefore a need to better control the uncertainties in the process. The other drawback of the method is the need to use standard Newmark spectra that may not be realistic for a given site or magnitude event. Finally, Michel et al. (2014) showed that the N2 method was not accurate to compute the response of structures for a large level of non-linearity ($R>4$). Damage grades 3 and 4 happen to correspond to very large reduction factors, up to 30 (though at this level of non-linearity, this concept is probably not valid anymore). There is therefore a need for the project to develop a new method coping with these drawbacks.

### 4.5 New derivation method including the uncertainties

The new method has been developed from the work of H. Crowley (personal communication). It uses the linearization method of Lin and Miranda (2008). In this method, in the non-linear range of behavior, the structure is assumed to have an equivalent period $T_e$ and damping $\xi_e$ that are computed based on the elastic period $T$ and the reduction factor $R$ (see above) as follows:

$$T_e = T \cdot \left(1 + \frac{0.026}{T^{0.87}} \cdot (R^{1.8} - 1)\right)$$

$$\xi_e = \xi + \frac{0.016}{T^{0.84}} \cdot (R - 1)$$

These equations correspond to a pure-elastoplastic capacity curve (see Lin and Miranda (2008) for further cases). The inelastic response of the structure is then supposed to be the same as the response of a linear structure with these equivalent properties. The next step is therefore to compute the spectral acceleration corresponding to the system having a period $T_e$ and a damping $\xi_e$. 

Derivation of fragility curves Basel Earthquake Risk Mitigation 01.03.2016
For that purpose, an input ground motion in the form of a response spectrum has to be given. Since there is not constraint on this spectrum, it is possible to provide a realistic spectrum based on a ground motion prediction equation. The spectrum for other damping ratios has to be also available for the computation. Therefore, we used spectra computed using the Akkar et al. (2014a) GMPE (later called ASB) that also provides a correction value (later called DSF) for different damping values (Akkar et al., 2014b). Alternatively, the EC8 correction factor could be used (CEN, 2004). We used ASB GMPE because it is currently the only one providing response spectra at all periods and damping ratios and inter-period correlation (needed later in this section). Although ASB may not be the most appropriate GMPE for Switzerland, only the shape of the spectra is of importance for the fragility computation. The inelastic displacement for a given capacity curves of period T and at reduction factor R is therefore:

\[ d^*_{\text{t}} = \text{ASB}(T, M, R_{JB}, V_{s30}, \text{SoF}) \times \text{DSF}(T, \xi_e, M, R_{JB}, V_{s30}, \text{SoF}) \]  

(7)

with \( M \) the magnitude of the scenario and \( R_{JB} \) the distance from the source. \( V_{s30} \) is the proxy used by this GMPE to account for site effects. It has been set to 450 m/s as an average from values in the Rhine Graben in Basel. Finally, \( \text{SoF} \) is the style of faulting, set to normal for the case of Basel.

The structural response depends therefore on the scenario, so that one set of fragility curve per scenario (magnitude, distance) is produced.

In order to account for the uncertainty, a set of 1000 capacity curves corresponding to possible behavior of the studied type of buildings provided by Resonance has been used. Each of the curves has its period T. Moreover, in order to account for the uncertainty in the demand spectra and the correlation between the initial period and the equivalent period, Crowley (personal communication) proposed a method to generate response spectra compatible with the Akkar et al. (2014) GMPE, conditioned on a given value of the spectral acceleration at the initial period \( \text{SA}(T) \). The inverse problem is therefore solved here by Monte Carlo (or Latin Hypercube) sampling: for each \( \text{SA}(T) \), a large number of conditioned spectra are generated, allowing to compute the distribution of the response of the structures as described above and therefore the probability of exceeding the defined damage limits is computed. The obtained fragility curves are smoothed using a moving average filter of order 7 applied in forward and reverse directions to avoid any shifting of the curves.

Alternatively to this complete method, when a single capacity curve is available, the method is applied similarly to derive the median values \( \mu_i \) of each damage grade. Then, a log-normal cumulative density function with a standard deviation of \( 0.4 \ln(\mu) \), where \( \mu \) is the ductility, is assumed (Lagomarsino and Giovinazzi, 2006).

One should notice that Lin and Miranda (2008) validated their method up to \( R=6 \). Vidic et al. (1994) who provided the base for the N2 method made computations for \( R \) up to 10. However, the performed computations to retrieve fragility curves exceed this level for the largest values of the intensity measure. Preliminary computations based on Michel et al. (2014) showed that the method of Lin and Miranda (2008) was still providing realistic results up to very large \( R \) values (25-50), whereas the N2 method was proven to provide too low values even at \( R=4 \) (Michel et al., 2014). However, depending on the structure, unrealistic behavior of Lin and Miranda (2008) was observed at large ground motion, where the computed response may be decreasing with increasing intensity (above 2 g). Parts of the curves showing these characteristics were discarded.

### 4.6 Derivation of damage grade 5

Mechanical methods do not provide in general fragility curves for complete collapse (damage grade 5) since it is impossible to differentiate it with damage grade 4 (partial collapse) in the model. The difference in the loss ratios between these two grades is however significant (especially in terms of fatalities). In order to separate them, Lagomarsino and Giovinazzi (2006) as-
sume that the share of buildings in damage grade 5 follows a binomial distribution as observed from field data. They proposed an approximation of the binomial distribution to compute the probability of exceedance of damage grade 5 as follows:

\[ p_{S5} = 0.09 \sinh(0.06 \mu_{DS}) p_{S4} \]

with \( \mu_{DS} = \sum_{k=1}^{5} k p_{sk} \) the mean damage and where the \( p_{sk} \) are the probabilities of being in the damage grade \( k \) (not the exceedance probability of the fragility curves).

However, it should be noticed that this approximation diverges from the true binomial distribution for high mean damages (above 3), leading to a limit of 50% of DG4 / 50% of DG5 for infinite ground motion, instead of 100% DG5.

Alternatively, we propose to compute the exact solution from the binomial distribution. For each damage grade \( k \), the probability of being in this damage grade assuming a binomial distribution is the following (Lagomarsino and Giovinazzi, 2006):

\[ p_{sk} = \frac{5!}{k!(5-k)!} \left( \frac{\mu_D}{5} \right)^k \left( 1 - \frac{\mu_D}{5} \right)^{5-k} \]  

(8)

Therefore, one can compute the share of DG5 in the probability of exceedance of DG4:

\[
\frac{p_{S5}}{p_{S4} + p_{S5}} = \frac{\left( \frac{\mu_D}{5} \right)^5}{\frac{4!}{5!} \left( 1 - \frac{\mu_D}{5} \right)^1 + \left( \frac{\mu_D}{5} \right)^5} = \frac{1}{\frac{5 \left( \frac{5}{\mu_D} - 1 \right)}{5} + 1} = \frac{1}{\frac{25}{\mu_D} - 4}
\]

(9)

It should be noticed that \( \mu_D \) is the mean damage grade computed using five damage grades and is therefore different from \( \mu_{DS} \) computed only using the first four damage grades. They are related by the following equation:

\[
\mu_D = \sum_{k=1}^{4} k p_{sk} + 4 \cdot p_{S4} + 5 \cdot p_{S5} = \sum_{k=1}^{4} k p_{sk} + 4 \cdot (p_{S4} + p_{S5}) + p_{S5} = \mu_{DS} + \left( \frac{\mu_D}{5} \right)^5
\]

(10)

\( \mu_D \) is therefore the root between 0 and 5 of the equation \( \left( \frac{\mu_D}{5} \right)^5 - \mu_D + \mu_{DS} = 0 \) that can be numerically solved for a given \( \mu_{DS} \).

The difference with the Lagomarsino and Giovinazzi (2006) method is displayed in Fig. 31. It shows a clear divergence with their method for damage grades greater than 3.5. Moreover, the difference is reduced if the mean damage grade from DG1 to DG5 is used in their method instead of the mean damage grade from DG1 to DG4, although this value is normally not available at this stage. This improvement is not important for scenario computation themselves since these high mean damage grades should not be reached, but is relevant for the comparison of fragility curves from different origins.

![Fig. 31 Share of DG5 from DG4 for increasing mean damage grade (based on DG1 to DG4) assuming a binomial distribution. Blue: approximate equation of Lagomarsino and Giovinazzi (2006), orange: same equation but using the damage grade computed using the five damage grades, red: new proposition without approximation.](image-url)
4.7 Results

Fig. 32 is presenting the fragility curves obtained with the N2 method and the new derivation method for a scenario of magnitude 5.7 located at 5 km for a selection of masonry and RC types.

The obtained curves with the new method are not following a lognormal cumulative density function as commonly assumed. They account for the fact that the uncertainty at large ground motion is larger due to the non-linear processes involved in the structural response. Moreover, they do not cross each other as lognormal cumulative density functions with different standard deviations would do. They are therefore more realistic than the curves obtained with the LM2 method and the uncertainty is controlled. The curves of the different types cannot be directly compared in terms of vulnerability because the used IM on the x-axis is different so that these curves can hardly be interpreted. Their interpretation is performed in section 5.

The effect of the magnitude of the scenario earthquake is investigated in Fig. 33: it shows that this parameter has a strong influence on the fragility curves. The N2 method is based on the Newmark spectrum and therefore corresponds to large magnitude earthquakes so that the two methods match for the M=6.6 scenario. However, for lower magnitude scenarios, the frequency content of the input motions imposes a large shift of the fragility curves. This is important because it shows that fragility curves developed using the N2 method or more generally using input corresponding to large magnitude events should not be used for low magnitude event such as geothermal events. The mechanical fragility curves are therefore scenario dependent.

On the contrary, the effect of distance to the source is limited (Fig. 34) and is neglected in the following (distance of 5 km is used).

The resulting fragility curves have been rounded at $10^{-3}$ in order to avoid damage from very low intensity events. Though such damage is possible in practice, our models do not provide control on the tails of the distributions and therefore the low probabilities.
Fig. 32 Fragility curves for types M_H_FF, M_H_RF, RC_W_NS and RC_F (from left to right and top to bottom) obtained with the complete method for a scenario of M=5.7 at 5 km distance and compared to the Risk-UE LM2 method based on the N2 method. The IMs (x-axis) are different so that the plots cannot be directly compared.

Fig. 33 Fragility curves for type M_H_RF obtained with the complete method for a scenario of M=5.0 (left), M=5.7 (centre) and M=6.6 (right) at 5 km distance and compared to the Risk-UE LM2 method based on the N2 method (identical from left to right).

Fig. 34 Fragility curves for type M_H_RF obtained with the complete method for a scenario of M=5.7 at 5 (left), 10 (centre) and 20 km (right) distance and compared to the Risk-UE LM2 method based on the N2 method (identical from left to right).
5. Verification of fragility curves

State-of-the-art seismic loss assessment is performed using mechanical methods (Crowley, 2014). They are preferred over empirical methods, i.e. using macroseismic intensity, because these physical parameters enable the quantitative modeling of phenomena that have not been observed, through analytical modeling (ground motion and its effects on structures). However, macroseismic intensity and its related studies still have benefits and cannot be completely ruled out. Major historical events generally occurred before the instrumental era and have therefore been studied with the macroseismic tools. Modern scenario-based risk assessment is often verified against these historical events. They provided a large amount of damage data that were used to calibrate the first Damage Probability Matrices (Whitman, 1973). They are still broadly used today for first-level analysis, especially by the insurance industry, since data is generally lacking to build reliable mechanical models. Mignan et al. (2015) used them to reproduce the damage from the 2006 geo-thermal event in Basel.

Besides, macroseismic Intensity is regaining interest in the last years through “citizen science” on the Web: people are asked to provide their observations after earthquakes (“Did You Feel It?”). Together with seismological recordings, these data can be finally converted into macroseismic intensity and displayed for example as Shakemaps (Wald et al., 1999). The macroseismic methods are based on observed data and are therefore expected to be accurate, whereas mechanical methods account for the behavior of each structure and are therefore expected to be precise, but eventually biased.

EMS98 (Grünthal et al., 1998) is the latest macroseismic scale and is using fuzzy definitions of vulnerability in order to extract the ground motion intensity from the damage field, based on numerous observations and expert judgment. Lagomarsino and Giovinazzi (2006) completed this method assuming a binomial distribution in order to derive fragility curves as a function of macroseismic intensity and later improved their model (Bernardini et al., 2010). They also first proposed a calibration of a mechanical method on their empirical fragility curves (Lagomarsino and Giovinazzi, 2006; Lagomarsino and Cattari, 2014).

We propose here a new method to verify the developed mechanical fragility curves with respect to standard empirical fragility curves. The limitations of this comparison are also highlighted. The idea used in the new method presented in section 4 to derive fragility curves can also be used to convert fragility curves derived using a given IM (e.g. SA(T)) to another IM (e.g. PGA). Therefore, comparisons between different curves from other mechanical methods can also be performed. We present here the comparison with the mechanical curves of Lagomarsino and Giovinazzi (2006), and with the curves developed within the project “risque sismique du bâti existant” of the Federal office for Environment (Karbassi and Lestuzzi, 2014).

5.1 Empirical fragility curves (Risk-UE LM1 method)

Our comparison is based on the empirical fragility curves of Lagomarsino and Giovinazzi (2006), also known as Risk-UE LM1 method, updated by Bernardini et al. (2010). Their work is based on an interpolation of the fuzzy definition of EMS-98 that gives for each macroseismic intensity the distribution of damage for buildings of a given vulnerability class. Lagomarsino and Giovinazzi (2006) defined a vulnerability index \( V \) between 0 (low vulnerability) and 1 (high vulnerability) and assigned values and uncertainties for different building classes in Europe. The fragility curves are defined by the single parameter \( \mu_p \) (mean damage grade, see section 4.6). For a given vulnerability index \( V \), \( \mu_p \) is computed as a function of the EMS98 macroseismic Intensity as follows (Fig. 3 left):

\[
\mu_p = 2.5 \times \left[ 1 + \tanh \left( \frac{I + 25V - 110}{23} \right) \right] \quad \text{Lagomarsino and Giovinazzi (2006)}
\]

\[
\begin{align*}
\forall I > VII, \quad \mu_p &= 2.5 \times \left[ 1 + 1.2 \times \tanh \left( \frac{I + 25V - 127}{3} \right) \right] \\
\forall I \leq VII, \quad \mu_p &= e^{-\frac{I - 7}{2}} \times 2.5 \times \left[ 1 + 1.2 \times \tanh \left( \frac{I + 25V - 127}{3} \right) \right] \quad \text{Bernardini et al. (2010)}
\end{align*}
\]

A difference of 0.02 in the vulnerability index \( V \) for a given class exists between the two publications. Moreover, Lagomarsino and Cattari (2014) dropped the exception for \( I < 7 \).
Then, assuming the distribution of damage grades is binomial, the marginal probabilities of being in each damage grade is computed as:

\[ p_{sk} = \frac{5^k}{k! \cdot (5 - k)!} \left( \frac{\mu_D}{5} \right)^k \left( 1 - \frac{\mu_D}{5} \right)^{5-k} \]  

(13)

It allows the computation of the probability of exceedance of each damage grade as a function of macroseismic intensity (fragility curves).

It should be noticed that Bernardini et al. (2010) show a slight shift towards higher damage for the same intensity level compared to Lagomarsino et al. (2006): for intensity IX, according to EMS-98, many buildings (20-50%) of class B suffer damage grade 4: the 2006 version is giving 36% (center of the interval) whereas the new version provides 44% (Fig. 35, right). However, Bernardini et al. (2010) correct formerly unrealistic damage probabilities at low intensities (I < VI). The formulation of Bernardini et al. (2010) is used in the following.

**Fig. 35** Comparison of available macroseismic methods (example of vulnerability class B / V=0.72). Left: Mean damage grade; Right: corresponding fragility curves.

The vulnerability index V values are derived by Resonance (2015) for each building so that average values can be computed for each building type. The V values are relatively homogeneous and correspond to EMS98 vulnerability classes B and C.

### 5.2 Ground Motion to Intensity Conversion Equations (GMICEs)

In order to compare fragility curves as a function of Intensity with curves as a function of SA(T), with T the resonance period of the structure/type, a so-called Ground Motion to Intensity Conversion Equation (GMICE) is needed. Cauzzi et al. (2015) showed that the GMICE of Faenza and Michellini (2010) in terms of PGV (PGA also available), derived from Italian data, was coherent with the Swiss Stochastic model and the historical Swiss catalogue. Moreover, they calibrated an additive coefficient to account for the Swiss rock reference (\( \Delta I = 0.47 \)). Faenza and Michellini (2011) published a version of their GMICE in terms of spectral acceleration at periods T= 0.3, 1 and 2 s. Since they used orthogonal regression, their relationships are revertible into Intensity to Ground Motion Conversion Equations (IGMCEs), which is in general not the case for other GMICEs. The equations of Faenza and Michellini (2010, 2011) have the following form (Tab. 5 for coefficients):

\[ I = b + a \times \log_{10}(SA(T)[cm/s^2]) \]  

(14)

**Tab. 5** Coefficients of Faenza and Michellini (2010, 2011) GMICEs

<table>
<thead>
<tr>
<th>T [s]</th>
<th>a</th>
<th>b</th>
<th>σ</th>
</tr>
</thead>
<tbody>
<tr>
<td>0 (PGA)</td>
<td>2.58</td>
<td>1.68</td>
<td>0.35</td>
</tr>
<tr>
<td>0.3</td>
<td>2.47</td>
<td>1.24</td>
<td>0.53</td>
</tr>
<tr>
<td>1</td>
<td>2.05</td>
<td>3.12</td>
<td>0.36</td>
</tr>
<tr>
<td>2</td>
<td>4.31</td>
<td>0.29</td>
<td></td>
</tr>
</tbody>
</table>

PGA=Peak Ground Acceleration; σ standard deviation
GMICEs are subjected to several limitations. First of all, a large uncertainty is associated to such conversion, which should be accounted for. Another limitation is that all physical IM are not good measures for observed damage, depending on the magnitude: for low magnitude events, Intensity VI may be reached due to non-structural elements caused by large high-frequency ground motion, although low frequency parameters (SA(1s) for instance) may be particularly low.

Lagomarsino and Giovinazzi (2006) first proposed a validation of mechanical fragility curves against their empirical curves. They consider PGA=0.15 g as being I=VIII whereas in the Faenza and Michellini (2010) GMICE, it would correspond to I=VII which is a significant difference. Other GMICEs are providing a value of VI. Moreover, Lagomarsino and Cattari (2014) performed a comparison of empirical and mechanical fragility curves, through the conversion of the empirical curves into PGA using an IGMCE, where we propose to do the opposite. They did not account for the uncertainty in the conversion and used the Murphy and O’Brien (1977) IGMCE. It has to be noted that the Murphy and O’Brien (1977) relation is not based on EMS-98 but on the MMI macroseismic scale, a scale without any intrinsic use of statistical damage distributions, but mostly based on observed maximum damage. Their comparison with the GMICE of Faccioli and Cauzzi (2006) cannot be considered since they used a wrong coefficient and their GMICE should not be reverted into IGMCE.

The used IGMCE is much steeper than the Faenza and Michellini (2010) IGMCE (Fig. 36), causing a shift of the highest damage grades to lower intensities. Intensity IX corresponds to a PGA of 3.2 m/s² for Murphy and O’Brien (1977) against 6.9 m/s² for Faenza and Michellini (2010). Though the use of Murphy and O’Brien (1977) can hardly be justified for this study, Faenza and Michellini (2010) has been calibrated with limited data for Intensities greater than VIII and the results should be taken with care above this value.

Though a large uncertainty is related to this procedure, the conversion to Intensity is the only way of comparing empirical and mechanical methods. The results should therefore not be over-interpreted.

![Fig. 36 Comparison of Faenza and Michellini (2010) and Murphy and O’Brien (1977) IGMCE plus/minus one standard deviation.](image)

### 5.3 Conversion accounting for uncertainties

For this computation, a lognormal function is first fitted to the fragility curves developed in section 4, which allows an easy handling of uncertainties. Most likely, the period T of interest for a given set of fragility curves is not one of the periods where the GMICE is available (0, 0.3, 1 and 2 s). Therefore, we used Akkar et al. (2014) GMPE to predict the ratios Ra between SA(T) and SA(0.3-1-2s) and their uncertainty, for a given scenario (M,R,Vs₃₃ₙ,SoF) as detailed in section 4.5. It can be noticed that the Newmark spectral shape of Eurocode 8 would be an alternative way of computing the ratios Ra but no uncertainties are provided and the EC8 spectral shapes are not considered as realistic. For the present comparison, we used a scenario of M=6.6 at 5 km. Although the computation should not depend on the chosen scenario, we observed that it has an impact on the results and therefore chose a relatively large magnitude, likely better corresponding to the empirical data.
Finally, Intensity is computed based on the GMICE following:

\[ I = b + a \times \log_{10}(SA(0.3 - 1 - 2s)) = b + a \times \log_{10}(Ra \times SA(T)) \]  

(15)

The associated uncertainty given by the authors is added (L2 norm since these uncertainties are independent) to the lognormal standard deviation of the above computed ratio, converted to the Intensity scale (i.e. multiplied by \( b/\log(10) \)). The lognormal standard deviation of the fragility curves is added the same way. It is assumed the EMS-98 damage scale is valid for all studied methods, though mechanical proxies are used to define the damage grades in the mechanical methods (section 4.2). However, it was not accounted for here using an additional uncertainty.

5.4 Results and discussion

The comparison between the LM1 fragility curves with the ones used in this project, converted to Intensity accounting for uncertainties are presented in Fig. 3. This conversion to Intensity allows a direct comparison of the buildings’ vulnerability.

The first version of the delivered fragility curves, based on a similar approach to LM2 method (Lagomarsino and Giovinazzi, 2006), were showing a large shift for DG4 (and therefore DG5) towards left, meaning that the curves were too pessimistic about the seismic resistance of the buildings (conservativeness). The low damage grades, conversely, were well matching. On the contrary, the conclusion of Lagomarsino and Cattari (2014), using the GMICE of Murphy and O’Brien (1977), is that macroseismic and mechanical curves match. After these first computations, Resonance modified their hypotheses and delivered new sets of curves, accounting for uncertainties as developed in section 4. The modified hypotheses are detailed in Resonance (2015) and include the resonance period, the failure mode and the displacement capacity.

The goal of the procedure was not to obtain a perfect match of the curves since it is clear that the LM1 results are not a target, but a benchmark for verification. Moreover, the binomial assumption in the LM1 method fixes the distance between the damage grades so that a complex behavior cannot be modeled with such empirical method. It should also be noticed that the European LM1/2 types cover a larger range of buildings than the Basel types that are restricted to the school buildings found in Basel. Therefore, we expect the LM1/LM2 methods to show a larger dispersion. This was not particularly observed, which means that the developed curves are not too certain.

Most of the building types show a good match in Fig. 37 (e.g. M_H_FF, M_H_RF or RC_W_S). One of the largest differences comes from the RC shear wall (squat) buildings (RC_W_NS) where our mechanical curves show a significantly lower vulnerability compared to the empirical data. This is realistic since well-built RC shear walls exhibit a good seismic behavior although few damage data are available for this type of buildings. Its vulnerability is therefore probably overestimated in the empirical method, maybe due to a lack of damage data concerning this building type.

Another interesting feature is the behavior of types like RC_P (particular buildings with nearly no laterally resisting system) or MRC_MI_SS (soft-story) with very close damage grades. This kind of fragile behavior cannot be reproduced with an empirical method based on a vulnerability index because the distance between the damage grades is fixed by the used probability distribution. These types are particularly vulnerable according to our computations that should be confirmed with additional data.
Fig. 37 Comparison of fragility curves converted to Intensity with the LM1 method (Lagomarsino and Giovinazzi, 2006) for the following building types: a) M_H_FF; b) M_H_RF; c) MRC_MI_SS; d) RC_P; e) RC_W_NS; f) RC_W_S.

Furthermore, we compared the developed curves with the mechanical curves of Lagomarsino and Giovinazzi (2006) for the corresponding building types (damage grades 1 to 4 only), after conversion to the same IM (Fig. 38). Masonry types show in general a good agreement, although the slight damage occurs earlier for our curves. However, a large difference occurs for the RC types for which collapse occurs at much higher ground motion for our curves. Though this effect corresponds to an expected behavior, the difference is extreme.
Fig. 38 Comparison of fragility curves to the mechanical curves of Lagomarsino and Giovinazzi (2006) (LM2 method) for the following building types: a) M_H_FF; b) M_H_RF; c) M_L_RF; d) MRC_MI_SS; e) RC_W_NS; f) RC_W_S.

In addition, we compare the fragility curves of some of the building types to the curves developed in the project “risque sismique” of the Federal Office for environment (Karbassi and Lestuzzi, 2014). These curves have been developed for single buildings, typical of the Swiss building stock, using non-linear time history analysis (large magnitude events) and 3D non-linear models. Lognormal functions with the same standard deviation were used. The results (Fig. 39) show that slight damages for masonry structures occur earlier in our curves compared to Karbassi and Lestuzzi (2014), though the partial collapse and collapse are comparable. The opposite is true for RC buildings where the first damage grades tend to match whereas collapse occurs at higher ground motion in our project. However, the curves of Karbassi and Lestuzzi (2014) have been developed for single buildings, with significantly more stories for the RC types (11 and 6, respectively).
Fig. 39 Comparison of fragility curves to the curves of Karbassi and Lestuzzi (2014) for the following building types: a) M_H_FF; b) M_H_RF; c) M_L_RF; d) RC_W_S; e) RC_F.

As a conclusion, we verified that the produced fragility curves for the project were reasonable compared to empirical methods, i.e. to observed damage data, using a new method that we developed for the project. Even if this stage cannot be called a validation, it constitutes a necessary verification for any risk analysis.
6. Derivation of vulnerability curves

6.1 Procedure

In order to derive vulnerability curves, i.e. the probabilistic distribution of loss ratios (fatalities, injured and financial) as a function of the chosen IM, fragility curves are combined with loss ratios. Each DG corresponds to a probabilistic distribution of loss ratios (Fig. 40). In this project, loss ratios are assumed to follow a uniform distribution between two plausible bounds obtained from the literature (constant probability density). The combination of fragility curves and loss ratios is performed by Monte Carlo analysis: a large number of random samples in the damage distribution (as defined by the fragility curves) are multiplied by (independent) random values from the loss ratio in order to determine the distribution of the losses. This distribution is defined by its mean and coefficient of variation (standard deviation divided by the mean) as requested by Openquake that considers it as lognormal. It can be noticed that this was the only distribution available in Openquake for the vulnerability curves at the time of the project, although it has some drawbacks: the 0 value cannot be reached, although no fatality for instance is common in our model. Latest versions also allow the use of a beta distribution or a discrete probability density function. They should be considered for future projects.

In the following, a literature review is detailed for the choice of the loss ratios.

Fig. 40 Density distributions of financial loss ratios (see section 6.3) following a uniform distribution for damage grades 1 to 4 (dark green, light green, yellow and red, respectively).

6.2 Casualty rates

Coburn and Spence (2002) showed that, excluding secondary effects (landslides, tsunami etc), 90% of the deaths during earthquakes are due to building collapse. Other types of victims such as heart attacks can hardly be predicted. It justifies the use of so-called lethality rates, first introduced by Coburn et al. (1992) defined "as the ratio of the number of people killed to the number of occupants present in collapsed buildings of that class." They can be extended to injured people (casualty rate) and, besides the building class, depend on its functions, occupancy, collapse mechanism and extent, ground motion characteristics, occupant behavior, effectiveness of rescue teams, etc.

Coburn and Spence (2002) recognize 5 M-factors that influence the casualty rate. M1 is the population per building that is known in the Basel project. M2 is the occupancy by the time of the earthquake. In the Basel project, it has been decided to consider full occupancy (worst-case scenario) that corresponds approximately to a normal school day. It should be however noticed that an extension to the whole city of Basel should account for variable occupancy rates. According to their definition, M1 and M2 are however not part of the casualty rate himself.

M3 is the proportion of occupants trapped by collapse. It is strongly influenced by the building type, depends on the building height (more trapped people in higher buildings) and on the shaking intensity and frequency content (escape is more difficult for stronger shaking). It varies from 60% in high-rise RC building down to a small fraction for 1-story houses. M4 is the distribution of injuries for collapsed structures that also strongly depends on the building type. Finally, the M5 factor covers the post-collapse mortality that depends on the effectiveness of the rescue teams and the
building type (progressing in a collapsed masonry building may be easier for the rescue teams than in RC buildings).

Based on these concepts, casualty models have been proposed in the literature for different countries and calibrated with observed data. Four to five levels of casualty are generally recognized, from slight injuries needing a limited medical aid to death. We will consider only 2 levels here: injury needing medical aid (including slight injuries) and death. Casualty rates are given by building type in the first place, but could be also modulated by damage grade, number of stories, etc. Since only structural collapse is considered, only DG4 and 5 EMS98 are generally associated to casualty ratios, though collapse of non-structural elements, occurring at DG1-3 can also be associated to injuries or even deaths. This can happen to the occupants of a building or even outdoor. Payany (1983) indicates that "many" of the 46 victims of the 1909 Lambesc (South of France) earthquake have been hit in the street, sometimes by a single stone. The HAZUS manual (FEMA, 2012) recognizes the importance of non-structural elements and outdoor casualties. For the current project, outdoor casualties are not considered since a full occupancy is considered, nor non-structural elements.

In the frame of the LessLoss project, Spence (2007) proposed a new set of casualty rates based on data from earthquakes in Kobe, Chichi, Kocaeli, Northridge and Athens. The distribution of injuries (no, slight, moderate, serious, critical, death) for different types of buildings (Timber, Masonry, RC and steel) and different height classes (1, 2-3 and >4 stories) are given for structures in DG5.

So et al. (2013) used the values proposed by Spence (2007) for DG5 and since no values for DG4 were proposed there, they suggest the use of 25% of the DG5 values.

Zuccaro and Cacace (2011) proposed a casualty model for Italy calibrated from previous studies. The fatality ratios are set to 0 for DG0-3.

Jaiswal et al. (2011a) also propose a fatality model (no injury rates) for collapsed buildings (DG5) of different types (origin unclear) and suggest to use 25% of these values for partial collapse (DG4) (Jaiswal et al., 2011b). The application targets a worldwide loss model (PAGER system). The proposed values are in general lower than the other studies.

So and Pomonis (2012) derived fatality rates for collapsed masonry structures worldwide. They notice that masonry structures with pitched tile roofs supported by a wooden truss, as they are common in Europe, were much less lethal (10-15%) than other types of roofs. Based on the L'Aquila (2009) event, the fatality rate of low to mid-rise stone masonry structures with wooden floors was 9-12%, whereas in China, the masonry structures with concrete floors led to a rate of 10-18%.

Moreover, the HAZUS software of the Federal Emergency Management Agency is providing fatality and injury rates for all damage grades for the building types present in the US based on the available data (FEMA, 2012). In addition, a model for casualty rates for outdoor is proposed (not used here). It should be noticed that the damage scale used is however not the EMS98 damage scale, though similarities can be found.

We assembled 4 sets of casualty ratios for the types present in Basel based on Spence (2007), complemented by So et al. (2013), Zuccaro and Cacace (2011), Jaiswal et al. (2011a) and FEMA (2012) and propose in the following a comparison and a selection of these data. We selected the values for DG1-3 given by FEMA (2012) since this is the only study accounting for casualties at these damage grades. For DG4, the FEMA (2012) study shows completely different results compared to the other authors due to the damage scale used, for which DG4 does not correspond to partial collapse as for EMS98, and is therefore discarded. The values of the other studies are comparable with 12 to 20% injuries and 1.5 to 8% fatalities. Finally, for DG5, the 3 studies provide comparable results for injuries: 50 to 80%, but more variable results for fatalities 6 to
30%. The discrepancy comes only from the RC buildings where the value of 30% is reached in Spence (2007) and Zuccaro and Cacace (2011) studies, whereas FEMA (2012) or Jaiswal et al. (2011a) propose 10%. This large value corresponds most probably to badly designed RC frames subjected to pancake collapses that were responsible of numerous fatalities in Turkey or even Italy but are rare in Switzerland. Therefore, these high values have been discarded here and a maximum value of 15% is kept.

Tab. 6 Summary values (%) for the casualty rates (Injured I and Dead D) for each building class in Basel and each damage grade with uncertainty (minimum and maximum of the uniform distribution).

<table>
<thead>
<tr>
<th>Classes</th>
<th>Injury (I) and lethality (D) rates (%) for each damage grade</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>DG1</td>
</tr>
<tr>
<td>M_H_RF / M_H_FF</td>
<td>I 0.05</td>
</tr>
<tr>
<td>M_L_RF / M_L_FF</td>
<td>I 0.05</td>
</tr>
<tr>
<td>RC_F / RC_FW</td>
<td>I 0.05</td>
</tr>
<tr>
<td>RC_W_NS / RC_W_S</td>
<td>I 0.05</td>
</tr>
<tr>
<td>RC_P</td>
<td>I 0.05</td>
</tr>
<tr>
<td>MRC_MI_OOP</td>
<td>I 0.05</td>
</tr>
<tr>
<td>MRC_MI_SS</td>
<td>I 0.05</td>
</tr>
<tr>
<td>OTH</td>
<td>I 0.05</td>
</tr>
</tbody>
</table>

6.3 Property loss

In order to estimate the property loss, one uses the repair cost ratio, defined as the ratio between the cost of repair and the replacement cost. This ratio depends on the damage grade, but also on the economic condition of the considered country since countries in a difficult economic condition may prefer not to repair slight damages. In most recent studies (FEMA, 2012), the repair cost is divided into structural, non-structural and content damage. In such cases, the repair cost can also be given as a function of the occupancy type, due to a different replacement value for different use of buildings, and not to different repair cost for a given damage state. Non-structural damage may even be sub-divided into acceleration-sensitive and drift-sensitive elements. In the frame of the Basel project, for the sake of simplicity, a single value, including structural, non-structural and content damage is given for each damage grade. These values cannot therefore be directly compared to the FEMA (2012) values for structural damage that count only for a share of the repair cost.

The studies reviewed here are apparently based on expert judgment only. Computational analyses exist in the literature but can be used only for the buildings they have been derived for (e.g. Kappos et al., 2006). Kircher et al. (1997) proposed for damage grades Slight, Moderate, Extensive, Complete (US scale) the values 2-10-50-100%. D’Ayala et al. (1997) gave numbers for European historical cities, based on EMS92 damage scale of 5-20-50-80-100%. Tyagunov et al. (2004) proposed for Germany, based on the EMS98 damage scale, the values 0.5-10-40-80-100% with uncertainty bounds. Silva et al. (2015) proposed for Portugal to use 10-30-60-100% for the Slight/Moderate/Extensive/Complete damage states. Moreover, in Switzerland, the working group SIA269 proposed the values 1-40-80-100-100% (Jamali and Köll, 2015). These values are gener-
ally slightly higher than what is proposed elsewhere and may not account for the cost of the content that can be a large share of the replacement cost. For this project, DG1 is chosen between 0 and 2%, which matches the range observed in the literature and the average of the SIA group. For DG2 and DG3, the SIA choice is kept as a maximum bound and the lowest values are discarded. DG4 and DG5 are chosen as 100% loss as suggested by most of the authors.

**Tab. 7 Summary values (%) for repair cost ratios taken as minimum and maximum bounds.**

<table>
<thead>
<tr>
<th>DG1</th>
<th>DG2</th>
<th>DG3</th>
<th>DG4</th>
<th>DG5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Min</td>
<td>Max</td>
<td>Min</td>
<td>Max</td>
<td>Min</td>
</tr>
<tr>
<td>0</td>
<td>2</td>
<td>10</td>
<td>40</td>
<td>40</td>
</tr>
</tbody>
</table>

### 6.4 Results

The resulting vulnerability curves for some types of the project are presented in Fig. 41 for the human consequences and Fig. 42 for the financial consequences. The new method to develop fragility curves depends on the considered scenario (section 4.5), so do the vulnerability curves. The difference in the vulnerability curves for different scenarios is comparable to the one of the fragility curves since they are only multiplied by a loss ratio. Again, the vulnerability curves are not directly comparable since the used IMs are different, except types M_H_FF and M_H_RF that have the same. The uncertainties are increasing with increasing ground motion due to more uncertain damage distribution and loss ratios. M_H_FF (flexible floors) shows a slightly larger vulnerability compared to M_H_RF (rigid floors) as expected. However, for the most vulnerable types such as MRC_MI_SS, larger uncertainties at intermediate ground motion than for high ground motion may be present. In this case, the choice of the log-normal distribution may cause more losses for these intermediate ground motions that for the largest ones.

**Fig. 41** Vulnerability curves in terms of human loss ratio (fatalities and injured) for types M_H_FF, M_H_RF, RC_W_NS and RC_F for a scenario M=5.7 at 7.5 km. Error bars correspond to mean plus/minus one standard deviation (lognormal distribution).
Fig. 42 Vulnerability curves in terms of financial loss ratio for types M_H_FF, M_H_RF, RC_W_NS and RC_F for a scenario M=5.7 at 7.5 km. Error bars correspond to mean plus/minus one standard deviation (lognormal distribution).
7. Computation of scenarios

This section is describing the practical implementation of the scenarios in Openquake software and the results. Section 7.1 is describing the general workflow, section 7.2 the chosen scenarios with their characteristics, section 7.3 the content of the exposure, fragility and vulnerability files. The following sections are describing the results and sensitivity analysis.

7.1 Workflow of the computation and propagation of uncertainties

In the project, we decided to use the Openquake software (Silva et al., 2014), first because the recent Swiss Hazard Model 2015 used this software and therefore made the specific tools and data for Switzerland available in the software, and second as a test for future risk studies in Switzerland. Though empirical methods could be included in the software, it is designed for mechanical-based methods and based on ground motion prediction equations and fragility/vulnerability curves that should be function of the predicted Intensity Measure. This framework strongly limits the degrees of freedom of the exercise and imposes an extensive work of pre-processing. However, within this framework, a comprehensive consideration of the uncertainties can be done: the tool is designed to be fully probabilistic.

In Openquake, hazard and vulnerability computations are fully decoupled. For scenario and risk computations, hazard is represented by a set of Ground Motion Fields (GMFs). A GMF is a set of sites (latitude/longitude) for which an intensity measure value is provided. GMFs may group sets of sites with different Intensity Measures (e.g. PGA, SA(1s) etc.) but all the IMs are given for all the sites, increasing quickly the size of the computation. A large number of GMFs is used to sample the uncertainty in the ground motion (aleatory – epsilon from the GMPEs, and epistemic – different models). The losses are computed using the fragility and vulnerability curves for each of the GMF (Monte Carlo sampling) to retrieve the probabilistic losses. The uncertainty in the vulnerability should therefore be fully included in the fragility and vulnerability curves (see section 4 and 6). Additional features were developed in house, especially the derivation of fragility and vulnerability curves. For the Basel project, another important additional feature was the inclusion of site amplification. The general workflow of the computation is displayed on Fig. 43 and detailed in the following.

Fig. 43 Workflow of the scenario computation. Blue arrows denote computations performed using Openquake.
Computation of Ground Motion Fields

The Hazard calculators offer a large range of possibilities to compute the ground motion fields, particularly for Probabilistic Seismic Hazard Assessment (PSHA), not used here. The GMFs are computed at all the points defined in the exposure XML file (list of considered buildings) or alternatively, a regular grid can be defined.

For scenarios, where the source is pre-determined, only one calculator is available. The input parameters are the geometry of the fault (polygons) that can be complex, the position of the hypocenter, the rake (direction of slip on the fault) and the magnitude. These parameters are stored in the fault rupture XML file. It should be noticed that the geometry parameters (fault plane, hypocenter) are used only for the computation of distance for the GMPE (type of distance depending on the GMPE), the rake may also be accounted for in GMPEs related to the style of faulting. However, complex properties of the fault (slip distribution, directivity...) cannot be accounted for with a GMPE approach.

The second set of input parameters are relative to the choice of the GMPE(s). A $V_{s30}$ reference value or map has to be provided as input for the GMPEs. Within the project, $V_{s30}$ is not the used parameter to map amplification because it is too simplistic. The GMPE are therefore computed for the Swiss Reference Rock Model with $V_{s30}=1100$ m/s (Poggi et al., 2011). Moreover, the list of requested Intensity Measures (in our case SA at different periods, corresponding to the different building types) has to be provided. The GMPE has to be provided as well. The use of a logic tree XML file for GMPEs has been implemented in Openquake v1.6, made available Mid-November 2015, i.e. too late for our project. However, the logic tree file is already available from the Swiss Hazard Model 2015. The GMPE of Edwards and Fäh (2013), tailored for Swiss Foreland, is mostly used in the following. It provides single-station sigma (site variability has been removed), which is necessary. Several stress drop values are available. Cauzzi et al. (2015) showed that 60 bars was a standard value that could be used. Without the logic tree functionality, the uncertainty on this value can however not be accounted for. To show the importance of this epistemic uncertainty, computations have also been performed using the adjusted Akkar and Bommer (2010) GMPE.

Finally, a spatial correlation model can be used for the GMF computation. The model of Jarayam and Baker (2009), the only available in Openquake, is used. As developed in section 2.4, accounting for the spatial correlation of the ground motion residuals (uncertainties) is necessary to predict more realistic ground motion fields. It does not affect the mean value of ground motion but the uncertainties (see Appendix A). In Appendix B, some results towards a Swiss spatial correlation model are presented. In Openquake, spatial correlation can only be used for GMPEs providing inter- and intra-event standard deviations. This is currently not the case for all the GMPEs adjusted for Switzerland. However, this can be relatively easily modified in the code for the GMPE of Edwards and Fäh (2013), what we did in the project.

The critical issue is the implementation of the amplification due to the surface geology. For that purpose, the GMFs are exported from Openquake to files. A Python code, generating the amplification on the fly based on the procedure detailed in section 3.5, is modifying the ground motion before importing it back to Openquake. The amplification is dynamically computed, based on the latest available ESM functions. Two strategies have been selected (2006 microzonation and interpolation). Both are presently applied with the same weight to account for the epistemic uncertainties. We computed 500 GMFs for each amplification model yielding a total of 1’000 GMFs.

Computation of Losses

For the loss computations, Openquake offers two calculators: the scenario damage and scenario risk calculators. The first calculator is taking the GMFs and fragility curves as input and computes the distribution of the damage as output (mean and standard deviation). The standard deviation is not adapted to the description of uncertainties for this parameter so that only the mean can be used further. This calculator is used to estimate the distribution of unusable buildings, at least partially collapsed buildings and completely collapsed buildings. The first category is needed to compute the number of homeless people (in our case of children without school
building) and therefore by the crisis authority to plan the number of temporary shelters or school buildings. The at least partially collapsed buildings are the buildings where victims are expected. Finally, the completely collapsed buildings need search and rescue actions.

The second calculator is using GMFs and vulnerability curves as input and outputs the distribution of losses. The calculator can handle occupants’ vulnerability and financial losses, but in order to compute injured and fatalities, it has to be run twice. The only possible additional input parameter is the asset correlation. This number accounts for the correlation of the distributions of the vulnerability among the buildings. By default, this parameter has not been used but the sensitivity on this parameter has been tested. Different occupancy models (e.g. night and day) could be accounted for but this was not used within the project as explained in section 7.3.

It is important to notice that the computation of the losses for each single building is straightforward. However, since lognormal distributions are generally assumed (easy to multiply not to sum), the expected aggregated loss is not the sum of the median of the individual losses as it would be done for a deterministic computation and the standard deviation cannot be computed easily. The simulation of random scenarios (Monte Carlo analysis) has therefore to be performed from the source until the aggregated losses, not only until the damage distribution.

**Loss assessment in the literature**

Mechanical-based loss assessment in the literature is mostly related to the use or the development of a dedicated software. HAZUS (FEMA, 2012), ELER (Hancilar et al., 2010) and SELENA (Molina et al., 2010) are all based on the same methodology: capacity curves are selected from a database or developed using provided tools. Fragility curves are developed based on the response of the structure to design spectra (capacity spectrum method) and independent inter-story drift limit values proposed by the software. HAZUS is a commercial software that has been used for instance for loss assessment of the city of Montreal (Yu et al, 2015) or the Koecali region in Turkey (Spence et al., 2003).

Contrarily to these programs, Openquake (Silva et al., 2014) does not stick to a single vulnerability assessment method since it uses as input only fragility curves. The drawback is that the user has to develop himself these curves, or use existing ones.

In the frame of the Risk-UE project, capacity curves and displacement limits for fragility curves have been proposed by different research groups for various types of European buildings (LM2 method) (Lagomarsino and Giovinazzi, 2006; Kappos et al., 2006). A major difference with HAZUS is the use of the simpler N2 method instead of the capacity spectrum method. It allows a more direct derivation of fragility curves for sets of European building types. The Risk-UE dataset has been used for various loss assessment including the city of Barcelona (Barbat et al., 2010) or the Azores (Veludo et al., 2013).

Depending on the building type, different simplified structural models, including those developed during Risk-UE project, have been proposed in Europe. Silva et al. (2013) proposed an extension of one of these method (DBELA) to also compute the fragility curves based on a probabilistic distribution of capacity curves and the capacity spectrum method. The curves derived following this method have been used to compute loss scenarios for mainland Portugal (Silva et al., 2015), the district of Florence (Weatherill et al., 2015), Istanbul (Bal et al., 2008) or Kathmandu (Chaulagain et al., 2016). Some of these studies have been performed using Openquake.

At this point, one can remark that all the methods are using a design spectrum for the hazard, although the hazard is later computed using a ground motion prediction equation afterwards. Most of the publications relate to broad building types that can be used only for a large scale study. Moreover, the developed fragility curves, except the study of Silva et al. (2013), are log-normal distributions with standard deviation values fixed from others studies. The control on the uncertainty is therefore very limited. Our study goes therefore beyond what has been proposed up to now, though for a limited number of buildings (121).
### 7.2 Scenario events

The scenario earthquakes have been selected out of the historical damaging events and the disaggregation of the Swiss Hazard model 2015 (Wiemer et al., 2015). We now shortly describe the seismicity in the area and the selected events for the scenarios, including the needed parameters (magnitude, fault location etc.).

The first known event that struck the area of Basel in history may have occurred in 250 AD and damaged the roman city of Augusta Raurica located 10 km East of Basel (Fäh et al., 2006b). Its magnitude is assumed to be Mw=6.0 and the main fault in the area is described from geology and represented in purple in Fig. 44 (Fäh et al., 2006b).

Basel’s strongest historical earthquake is the Mw=6.6 event that occurred on October 18th 1356. This event is the largest known North of the Alps. Ferry et al. (2005) reconstructed sections of the Reinach fault that may have ruptured in 1356 (in blue in Fig. 44). The length of the investigated sections of the fault is too short to correspond to a Mw=6.6 event but the extent to the South is not important for our scenarios since the distance between the rupture and Basel only depends on the Northern end. This end is however debatable as well. Moreover, Cauzzi et al. (2015) used a simplified geometry for the fault (dark green in Fig. 44) that is also further used in this work. The Reinach fault is located very close to the city and the distance to the rupture is mostly related to the assumed depth. Though the rupture reached the surface (Ferry et al., 2005), we cannot assume that the fault released important parts of the energy close to the surface because the asperities generating the largest amount of energy are generally located at depth. The assumption of a shallow depth of 2 km is adopted though it is difficult to justify. Moreover, Cauzzi et al. (2015) showed that the ground motion generated with the Edwards and Fäh (2013) model with a stress-drop of 60 bars was well fitting the reconstructed macroseismic field.

The Swiss Hazard Model 2015 (Wiemer et al., 2015) used all the available seismicity information and the most up to date ground motion prediction models to estimate the probability of occurrence of each ground motion intensity (e.g. Peak Ground Acceleration PGA) for all sites in Switzerland and for the theoretical Swiss reference rock model (Poggi et al., 2011). For instance, the model is providing the probability of exceedance of each PGA value in the city-centre of Basel. It is also providing the disaggregation of the results for a given probability value. We consider here the probability of occurrence of 10% in 50 years, corresponding to a return period of 475 years. It corresponds to the probability of occurrence used as reference in the current design code SIA261 for normal buildings. For this particular value, the PGA value is 0.1 g in Basel (at the reference rock) and the distribution of the events contributing to this estimated PGA (so called disaggregation) is represented on Fig. 45. The most likely value in Fig. 45 corresponds to an event of magnitude Mw of 5.6 to 5.7 located at 5 to 10 km distance. Therefore, we selected a scenario of Mw=5.7 located on a fault oriented parallel to the Reinach fault at a distance of 7.5 km of the buildings of interest and with a length compatible with the size of the event (orange line on Fig. 44). It can be seen on Fig. 44 that other magnitude-distance couples have a high probability in the disaggregation. Among these values, we picked up an event of Mw=5.0 at 5 km distance (light green color on Fig. 44) that we used as another possible scenario.

Finally, we included the geothermal event with Mw=3.2 of December 8th 2006 (Ripperger et al., 2009; Edwards et al., 2015). The location of the fault is very accurate compared to the previous events (red dot on Fig. 44). The stress-drop has been estimated to 35 bars (Edwards et al., 2015) but since this value was not available in Openquake, the value 30 bars has been used. No structural damage occurred during this event.

When no other information was available the standard value of stress drop of 60 bars has been used.
7.3 Exposure and vulnerability (before and after retrofit)

The considered elements at risk are the individual school building (121 elements). Each of them is associated to a building type, a replacement cost and a number of pupils. At the first order, the occupancy in a school building is either total when there is class, or zero outside of school time. This could be refined considering administration employees and an average presence rate that is probably below 100%, but for the project we only assumed full occupancy, corresponding to an event occurring during school time. Moreover, a long term retrofitting project of the school buildings (project HarmoS), not only related to earthquake safety, started to improve the buildings. Only 33 buildings have been subjected to significant retrofitting (Resonance, 2015). One goal of the present project is to quantify the global gain of earthquake safety due to the retrofitting. Therefore, two exposure configurations have been determined: before and after retrofitting, i.e. before the retrofitting project started and at the end of the retrofitting works. The current status is somewhere in-between these two states since only part of the measures has been already implemented. The type of structure is the only different parameter before and after retrofit. This simple approach does not allow to quantify finely the gain of the retrofitting for each individual structure but provides an idea for the building stock as a whole. It has been estimated that the occupancy would stay the same and the value would increase by maximum 10% by the end of the project. It has been finally decided to keep the same values.
Resonance (2015) defined 12 building types for which a set of fragility and vulnerability curves has been determined (sections 4 and 6). These curves are function of the elastic spectral acceleration at the fundamental frequency of the type. We computed one set of curves per magnitude/distance scenario (see section 4). Weatherill et al. (2015) showed that taking the inter-period correlation was important for the scenario computation and when it is not available, as in our case, they suggest to use a single intensity measure. Therefore, a computation has also been performed converting all the curves to the same intensity measure, in this case SA(0.5s), for the sensitivity analysis.

### 7.4 Results

**Ground motion fields**

Fig. 46 shows the median ground motion fields for the following scenarios: the repeat of the 1356 event (Mw=6.6 at 2 km), the Mw=5.7 at 7.5 km and the Mw=5.0 at 5 km. The ground motion level is clearly increasing with the magnitude. The effect of the site amplification can also be clearly observed. It should however be noticed that, to produce these maps, the amplification outside of the Rhine Graben has been simplified compared to section 3 because it was not useful for the scenarios. The difference between PSA(1s), corresponding to tall flexible buildings (RC_P and MRC_MI_SS in our scenarios), and PSA(0.3s) is also noticeable in terms of spatial distribution due site amplification and source properties. Such a complex interaction between source and building can only be reproduced with mechanical scenarios, but not with empirical ones. Fig. 46 corresponds however to the median ground motion that does not correspond to a realistic spatial distribution of ground motion: Fig. 47 shows an example of ground motion field (1000 are used for one scenario) including spatial correlation that is actually used for the computations. One cannot recognize a ground motion "level" from a single GMF.

![Fig. 46 Median ground motion field (spectral acceleration) for the scenarios of a hypothetical Mw=6.6 event, the Mw=5.7 and the Mw=5.0 events (from left to right), at 0.3 s (top) and 1 s (bottom) periods.](image-url)
The results before/after retrofitting are discussed in section 7.6.

### Losses

Table 8 summarizes the results of the 5 scenarios, after the retrofitting project. It should be noticed that the uncertainty concerning the number of unusable and at least partly collapsed buildings (and further the number of homeless) cannot be extracted from Openquake. The provided values are rough estimates. For the scenarios from the disaggregation (475 years return period), no total collapse is expected, leading to a much lower amount of victims compared to an event similar to the 1356 earthquake. This event would be a catastrophe: nearly all the school buildings would be unusable and a large number of fatalities (several hundreds) and injured would be resulting. This scenario has however a low probability to occur. In the two scenarios from the disaggregation, ten (M=5) or several tens (M=5.7) of fatalities may still occur, but building collapse, even partial, is not certain. One should however notice that the uncertainties are nearly as large as the mean values so that only the order of magnitude should be considered. A large part of the building stock would however not be usable right after the event.

An event similar to the 250 Augusta Raurica earthquake (Mw=6.0 at 10 km) would cause approximately as much damage to the Basel schools as the M=5.7 scenario (similar amplitude of the ground motion).

Finally, the Mw=3.2 event from 2006 has been tested. It shows no significant damage and victims and a financial loss with a much larger uncertainty than its mean value. Our model is not able to predict accurately the consequences of such small event since we are only focusing on structural damage.

The results before/after retrofitting are discussed in section 7.6.

### Tab. 8 Results from the scenarios after retrofitting

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Basel 1356</th>
<th>M=<a href="mailto:5.7@7.5km">5.7@7.5km</a></th>
<th>M=5@5km</th>
<th>Augusta 250</th>
<th>DHM Basel 2006</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Out of 121 buildings</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>not usable (DG≥2)</td>
<td>102±48 (84%)</td>
<td>61±50 (50%)</td>
<td>44±39 (36%)</td>
<td>60±49 (50%)</td>
<td>0±1 (&lt;1%)</td>
</tr>
<tr>
<td>At least partial collapse (DG≥4)</td>
<td>35±6 (29%)</td>
<td>6±3 (5%)</td>
<td>2±2 (2%)</td>
<td>6±3 (5%)</td>
<td>0±1 (&lt;1%)</td>
</tr>
<tr>
<td>Total collapse (DG5)</td>
<td>12±4 (10%)</td>
<td>1±1 (1%)</td>
<td>0±1 (&lt;1%)</td>
<td>1±1 (1%)</td>
<td>0±1 (&lt;1%)</td>
</tr>
<tr>
<td><strong>Out of 16960 pupils</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fatalities</td>
<td>305±130 (2%)</td>
<td>33±30 (&lt;1%)</td>
<td>10±13 (&lt;1%)</td>
<td>34±32 (&lt;1%)</td>
<td>0±1 (&lt;1%)</td>
</tr>
<tr>
<td>Injured</td>
<td>1814±709 (11%)</td>
<td>242±189 (1%)</td>
<td>88±82 (1%)</td>
<td>244±199 (1%)</td>
<td>0±1 (&lt;1%)</td>
</tr>
<tr>
<td>Homeless (DG≥2)</td>
<td>13129 (77%)</td>
<td>8809 (52%)</td>
<td>6311 (37%)</td>
<td>8651 (51%)</td>
<td>1 (&lt;1%)</td>
</tr>
<tr>
<td>Total value 1143 MCHF</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Total losses (MCHF)</td>
<td>566±122 (50%)</td>
<td>210±94 (18%)</td>
<td>126±49 (11%)</td>
<td>202±73 (18%)</td>
<td>0.11±1 (&lt;1%)</td>
</tr>
</tbody>
</table>
The contribution of each type has been investigated. Fig. 48 shows the damage distribution of each building grouped by type for the scenarios 1356, M=5.7 and M=5.0. The catastrophic nature of the 1356 scenario is clear to see on this figure where a lot buildings collapse. The other scenarios show only a small number of buildings with DG3 or larger. Large numbers of victims logically occur in the most vulnerable building types with a large number of pupils. The large majority of fatalities occur in masonry buildings (about 70% for the 1356 and 5.7 scenarios). The share of the financial losses in masonry structures is lower but still dominant (about 60%).

The fatality rates per type (number of fatalities divided by number of pupils) range from 0.3% to 6% for the repeat of the 1356 event and from 0% (considering our precision) to 3.5% for the Mw=5.7 scenario (Fig. 49; Tab. 9). This parameter depends at the first place on the scenario (larger rates for stronger earthquakes) and on the vulnerability. The largest fatality rate occurs for the MRC_MI_SS building type (Tab. 9). Masonry building types all have a similar fatality rate (about 3% for the 1356 scenario and 0.5% for the 5.7 scenario), significantly larger than the reinforced concrete structures (Tab. 9). The RC_P building type is an exception for the RC structures: it shows fatality rates comparable or even larger than masonry types for the 1356 event (4.4%) as well as the 5.7 scenario (0.4%).

For a given scenario, the ground motions are relatively homogeneous over the considered area (most of the schools are in the Rhine Graben) so that variability in ground motion does not explain differences in the fatality rates. The two school buildings outside the Rhine Graben show among the lowest fatality rates.

It is also important to notice that the mean damage grades are low for all the buildings: on average, they are not expected to collapse and induce fatalities. For the M=5.7 scenario, the maximum mean damage grade is 2.6. Building collapse and fatalities occur due to uncertainties in the final damage grade of the building, so that they cannot be captured by deterministic computations.

<table>
<thead>
<tr>
<th>Type</th>
<th>Basel 1356</th>
<th>M=<a href="mailto:5.7@7.5km">5.7@7.5km</a></th>
<th>M=5@5km</th>
</tr>
</thead>
<tbody>
<tr>
<td>M_H_FF</td>
<td>2.90%</td>
<td>0.44%</td>
<td>0.13%</td>
</tr>
<tr>
<td>M_H_RF</td>
<td>2.14%</td>
<td>0.19%</td>
<td>0.05%</td>
</tr>
<tr>
<td>M_L_FF</td>
<td>4.23%</td>
<td>0.61%</td>
<td>0.17%</td>
</tr>
<tr>
<td>M_L_RF</td>
<td>3.51%</td>
<td>0.61%</td>
<td>0.24%</td>
</tr>
<tr>
<td>MRC_MI_OOP</td>
<td>3.15%</td>
<td>0.15%</td>
<td>0.02%</td>
</tr>
<tr>
<td>MRC_MI_SS</td>
<td>6.12%</td>
<td>3.49%</td>
<td>0.55%</td>
</tr>
<tr>
<td>OTH</td>
<td>2.02%</td>
<td>0.34%</td>
<td>0.23%</td>
</tr>
<tr>
<td>RC_F</td>
<td>1.28%</td>
<td>0.02%</td>
<td>0.00%</td>
</tr>
<tr>
<td>RC_FW</td>
<td>1.37%</td>
<td>0.03%</td>
<td>0.00%</td>
</tr>
<tr>
<td>RC_P</td>
<td>4.40%</td>
<td>0.38%</td>
<td>0.02%</td>
</tr>
<tr>
<td>RC_W_NS</td>
<td>0.25%</td>
<td>0.01%</td>
<td>0.00%</td>
</tr>
</tbody>
</table>
Fig. 48 Average damage distribution of the buildings per type for the M=5 (top), M=5.7 (centre) and 1356 (bottom) scenario events after retrofitting.
We tested the convergence of the computations by using 2000 GMFs instead of 1000 for the Mw=5.7 scenario. The largest difference reaches 2% that can be considered as acceptable. The chosen number of GMFs is therefore adequate. Table 10 shows the results of the 5.7 scenario without uncertainty in one component of the computation in order to determine the sources of uncertainty in the results. The ground motion is from far the most uncertain component: a scenario using the median ground motion field only (no uncertainty) is ten times less uncertain regarding fatalities, six times less regarding injured and three times less regarding financial losses. Therefore, the uncertainty in the vulnerability is several times smaller than that in the ground motion but it depends on the scenario and the investigated parameter. Moreover, the mean losses are significantly different whether uncertainties are considered or not. This can be explained by the fact that accounting for the uncertainties in the ground motion let us consider extreme cases with a lot of fatalities. However, depending on the scenario, this can lead to larger or lower mean losses: in the case of the M=5.7 event, fatalities are lower but financial losses larger if no uncertainty in the ground motion is considered. The effect of spatial correlation on the uncertainty of the results is also clear: the uncertainty increases by 20% if spatial correlation is considered in this case (no effect on the mean value as explained in Appendix A). The effect of the uncertainty in the site amplification is limited: both used strategies (microzonation and interpolation) lead to results with insignificant differences. The effect of uncertainty in the loss ratio is in the same order as that of the spatial correlation.

We tested the convergence of the computations by using 2000 GMFs instead of 1000 for the Mw=5.7 scenario. The largest difference reaches 2% that can be considered as acceptable. The chosen number of GMFs is therefore adequate. Table 10 shows the results of the 5.7 scenario without uncertainty in one component of the computation in order to determine the sources of uncertainty in the results. The ground motion is from far the most uncertain component: a scenario using the median ground motion field only (no uncertainty) is ten times less uncertain regarding fatalities, six times less regarding injured and three times less regarding financial losses. Therefore, the uncertainty in the vulnerability is several times smaller than that in the ground motion but it depends on the scenario and the investigated parameter. Moreover, the mean losses are significantly different whether uncertainties are considered or not. This can be explained by the fact that accounting for the uncertainties in the ground motion let us consider extreme cases with a lot of fatalities. However, depending on the scenario, this can lead to larger or lower mean losses: in the case of the M=5.7 event, fatalities are lower but financial losses larger if no uncertainty in the ground motion is considered. The effect of spatial correlation on the uncertainty of the results is also clear: the uncertainty increases by 20% if spatial correlation is considered in this case (no effect on the mean value as explained in Appendix A). The effect of the uncertainty in the site amplification is limited: both used strategies (microzonation and interpolation) lead to results with insignificant differences. The effect of uncertainty in the loss ratio is in the same order as that of the spatial correlation.

**Fig. 49** Fatality rate per type for scenario M=5.7 after retrofitting.

<table>
<thead>
<tr>
<th>Tab. 10</th>
<th>Sensitivity study on scenario M=5.7 after retrofitting</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>reference</td>
</tr>
<tr>
<td>Out of 121 buildings</td>
<td></td>
</tr>
<tr>
<td>not usable (DG≥2)</td>
<td>61 (50%)</td>
</tr>
<tr>
<td>At least partial collapse (DG≥4)</td>
<td>6 (5%)</td>
</tr>
<tr>
<td>Total collapse (DG5)</td>
<td>1±1 (1%)</td>
</tr>
<tr>
<td>Out of 16960 pupils</td>
<td></td>
</tr>
<tr>
<td>Fatalities</td>
<td>33±30</td>
</tr>
<tr>
<td>Injured</td>
<td>242±189 (1%)</td>
</tr>
<tr>
<td>Homeless (DG≥2)</td>
<td>8809 (52%)</td>
</tr>
<tr>
<td>Total value</td>
<td>1143</td>
</tr>
<tr>
<td>MCHF</td>
<td>210±94 (18%)</td>
</tr>
</tbody>
</table>
In Table 11, we present the results for the same scenario using alternative modelling hypotheses that have not been finally selected, in order to understand their effects. The tested effect with the largest impact is the chosen GMPE. This is not a surprise since the Swiss hazard already showed that epistemic uncertainties on ground motion models were playing a major role (Wiemer et al., 2015). Using the Akkar and Bommer (2010) GMPE, adjusted for the Swiss reference (Vs/kappa adjustment), leads to an increase of 50% of the financial losses and the fatalities are more than doubling. However, the importance of this uncertainty in scenario modelling, for which the magnitude and location are anyway fixed, can be relativized.

The absolute effect of site amplification is also very important: the fatalities would be 1/8 of what they are if the city was built on the theoretical Swiss reference rock (corresponding approximately to weathered molassic rock of the Swiss plateau), and the financial losses about 1/3.

We also tested the sensitivity of the results to the used intensity measure. We converted all the fragility and vulnerability curves to a common intensity measure: SA(0.5s). The results are significantly different (50% more fatalities, 15% more financial losses) but more importantly, the uncertainties are noticeably larger, multiplied by 1.4 to 4 depending on the considered parameters. The increase in the mean values is related to the increase in the uncertainties in the vulnerability. Finally, we tried to use an asset correlation coefficient of 0.9 (large losses for one building compared to the mean value will lead to large losses for all the buildings and vice versa) that also strongly increase the uncertainty in the losses (nearly a factor of 2). However, such correlation lacks data to be justified.

In Tab. 12, we compare the results of the repeat of the 1356 event with two different fault geometries. They have the same strike and magnitude, a similar position, they are if the city was built on the theoretical Swiss reference rock (c
c

<table>
<thead>
<tr>
<th>Tab. 11 Alternative modelling hypotheses on scenario M=5.7 after retrofitting</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Out of 121 buildings</strong></td>
</tr>
<tr>
<td>not usable (DG≥2)</td>
</tr>
<tr>
<td>GMPE AK10 (w.o. correlation)</td>
</tr>
<tr>
<td>w.o. amplification</td>
</tr>
<tr>
<td>common IM</td>
</tr>
<tr>
<td>with asset correlation</td>
</tr>
<tr>
<td>---</td>
</tr>
<tr>
<td>61 (50%)</td>
</tr>
<tr>
<td>At least partial collapse (DG≥4)</td>
</tr>
<tr>
<td>6 (5%)</td>
</tr>
<tr>
<td>Total collapse (DG5)</td>
</tr>
<tr>
<td>1±1 (1%)</td>
</tr>
<tr>
<td><strong>Out of 16960 pupils</strong></td>
</tr>
<tr>
<td>Fatalities</td>
</tr>
<tr>
<td>33±30</td>
</tr>
<tr>
<td>Injured</td>
</tr>
<tr>
<td>242±189 (1%)</td>
</tr>
<tr>
<td>Homeless (DG≥2)</td>
</tr>
<tr>
<td>8809±52 (52%)</td>
</tr>
<tr>
<td><strong>Total value 1143 MCHF</strong></td>
</tr>
<tr>
<td>Total losses (MCHF)</td>
</tr>
<tr>
<td>210±94 (18%)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Tab. 12 Sensitivity on the choice of the fault for a repeat of the 1356 event (after retrofitting)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Out of 121 buildings</strong></td>
</tr>
<tr>
<td>not usable (DG≥2)</td>
</tr>
<tr>
<td>Reference</td>
</tr>
<tr>
<td>102 (84%)</td>
</tr>
<tr>
<td>Cauzzi et al. (2015) fault</td>
</tr>
<tr>
<td>At least partial collapse (DG≥4)</td>
</tr>
<tr>
<td>35 (29%)</td>
</tr>
<tr>
<td>Total collapse (DG5)</td>
</tr>
<tr>
<td>12±4 (10%)</td>
</tr>
<tr>
<td><strong>Out of 16960 pupils</strong></td>
</tr>
<tr>
<td>Fatalities</td>
</tr>
<tr>
<td>305±130 (2%)</td>
</tr>
</tbody>
</table>
Injured \( \pm 709 \) (11%) \hspace{1cm} 2509\( \pm 780 \) (15%)

Homeless (DG\( \geq 2 \)) \hspace{1cm} 13129 (77%) \hspace{1cm} 13056 (77%)

Total value 1143 MCHF

Total losses (MCHF) \hspace{1cm} 566\( \pm 122 \) (50%) \hspace{1cm} 688\( \pm 119 \) (60%)

As a conclusion of the sensitivity tests, we showed that the uncertainty in the ground motion is strongly dominating our results. Epistemic uncertainty in the GMPE is still a key issue that should be tackled: although future versions of the software will allow to account for it in the computation, it will remain a major source of uncertainties for risk assessment. For scenarios modelling, its importance can be relativized since magnitude and location of the event are anyway fixed. Spatial correlation and the choice of the intensity measure play also a critical role in the magnitude of the uncertainties. Finally, the uncertainty due to the effect of amplification from surface geology has been critically decreased thanks to the large measurement and instrumenting effort performed in Basel in the last 20 years.

7.5 Comparison with the empirical method

In order to cross-check the results with empirical data, we converted the GMFs to Macroseismic Intensity using Faenza and Michellini (2011) as described in section 5 and computed the damage distribution following Risk-UE LM1 method (Lagomarsino and Giovinazzi, 2006; Bernardini et al., 2010). Spectral acceleration at 0.3 s period was used. The vulnerability index for each building is provided in the Resonance (2015) report. All the values are however not provided (no values for types OTH and RC_P) so the statistic is based on 101 out of 121 buildings.

The converted intensity in the city centre corresponds roughly to Intensity 8.9 for the repeat of the 1356 event, 7.7 for the Mw=5.7 and 7.0 for the Mw=5.0. As a comparison, the Akkar and Bommer (2010) GMPE for the Mw=5.7 event leads to a converted intensity of 8.3 and the repeat of the 1356 event using the fault of Cauzzi et al. (2015) gives a value of 9.1.

The empirical and mechanical scenarios provide a coherent damage distribution, though differences naturally exist (Tab. 13). Assuming the same intensity for all the buildings (e.g. I=IX for the 1356 event) leads to similar results (slightly higher).

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Basel 1356</th>
<th>M=5.7</th>
<th>M=5</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Mechanical</td>
<td>Empirical</td>
<td>I=IX</td>
</tr>
<tr>
<td>not usable (DG( \geq 2 ))</td>
<td>84%</td>
<td>83%</td>
<td>87%</td>
</tr>
<tr>
<td>At least partial collapse (DG( \geq 4 ))</td>
<td>29%</td>
<td>28%</td>
<td>32%</td>
</tr>
<tr>
<td>Total collapse (DG5)</td>
<td>10%</td>
<td>7%</td>
<td>8%</td>
</tr>
</tbody>
</table>

As a comparison, during the 2009 L’Aquila (Mw=6.3) event, about 4% of the 1665 surveyed buildings in the city completely collapsed (DG5), 20% suffered at least partial collapse (DG\( \geq 4 \)) and 90% were not usable (DG\( \geq 2 \)) (recomputed from Tertulliani et al., 2011). Given the magnitude and distance to the city, intermediate between our 1356 and the M=5.7 scenario, these results are also coherent.

Lang and Bachmann (2004) computed the damage distribution for a ground motion corresponding to the design code at that time, i.e. 475 yrs. return period event. They found out that 45% of the unreinforced masonry buildings would at least partially collapse and that mixed buildings would behave even worse. Considering an EMS vulnerability class of B, this damage distribution corresponds to Intensity IX to X, although the PGA is only 1.3 m/s\(^2\), i.e. corresponding to Intensity VII according to Faenza and Michellini (2010). Other GMICEs may even give Intensity VI. By verifying the fragility curves in section 5, we tried in this project to avoid such an inconsistency between the computations and the past observations. For the 475 yrs. return period event in Basel, complete collapse should be unlikely. This is the result found from the scenarios based on the disaggrega-
Computation of scenarios as well as the empirical scenario (Tab. 13), which shows the consistency of our scenarios. This could be achieved only by taking out the conservativeness in the vulnerability assessment methods.

### 7.6 Reduction of potential losses (before and after retrofitting)

Table 14 shows the results before and after retrofitting for 3 scenarios. The differences are limited with respect to the uncertainties, but they are considerable in absolute value.

Before retrofitting, type M_H_FF is supporting alone 30% of total fatalities, while after retrofitting this number goes down to 10% because 15 buildings left this category. This is the most notable change concerning the distribution of the losses per type.

<table>
<thead>
<tr>
<th>Tab. 14 Results from three scenarios before and after retrofitting</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Retrofitting</strong></td>
</tr>
<tr>
<td></td>
</tr>
<tr>
<td>Out of 121 buildings</td>
</tr>
<tr>
<td>not usable (DG≥2)</td>
</tr>
<tr>
<td>At least partial collapse (DG≥4)</td>
</tr>
<tr>
<td>Total collapse (DG5)</td>
</tr>
<tr>
<td>Out of 16960 pupils</td>
</tr>
<tr>
<td>Fatalities</td>
</tr>
<tr>
<td>Injured</td>
</tr>
<tr>
<td>Homeless (DG2)</td>
</tr>
<tr>
<td>Total value 1143 MCHF</td>
</tr>
<tr>
<td>Total losses (MCHF)</td>
</tr>
</tbody>
</table>

The difference before/after retrofitting is computed in Tab. 15 for the 33 buildings out of 121 that have been or are planned to be retrofitted. Victims are approximately divided by 2 for all the events. The fatality rates of the retrofitted buildings remain below 0.6% for the M=5.7 event, compared to the largest values at about 3% it means that they are not among the most problematic structures anymore.

The financial gain is smaller, around 25%. It corresponds to direct losses of 13 MCHF for the 475 yrs. return period event. This number does not take into account the cost of fatalities, injured and homeless. This number could be compared to the investment of the city in the seismic retrofitting of the 33 buildings.

Fig. 50 shows the average distribution of occupants for all considered buildings in each damage grade before and after retrofitting. Pupils are mostly in buildings with DG2, which explains their large number without usable buildings right after the event. This type of damage can be in general repaired but the large number of buildings involved may be a challenge. Only few occupants are located in buildings with DG3 or larger. This number decreases slightly after retrofitting, which explains the lower number of fatalities and injured but since the structures are still damaged, the financial gain is more limited.

The retrofitting measures in project HarmoS are in general limited and target the loss of life that is indeed improved. It should however be noticed that these gains are mostly due to the change of two RC_P buildings, particularly deadly in our scenarios, into other building types: these two buildings explain alone 1/3 of the gain in terms of fatalities. The high vulnerability and the low uncertainty (steep fragility curves) on types RC_P and MRC_MI_SS should be investigated further to consolidate the results.
Tab. 15 Loss gain between before and after retrofitting for the 33 retrofitted buildings only

<table>
<thead>
<tr>
<th></th>
<th>Basel 1356</th>
<th>M=<a href="mailto:5.7@7.5km">5.7@7.5km</a></th>
<th>M=5@5km</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fatalities</td>
<td>46%</td>
<td>62%</td>
<td>59%</td>
</tr>
<tr>
<td>Injured</td>
<td>43%</td>
<td>54%</td>
<td>48%</td>
</tr>
<tr>
<td>Total losses</td>
<td>23%</td>
<td>25%</td>
<td>24%</td>
</tr>
</tbody>
</table>

Fig. 50 Expected distribution of occupants per damage grade for the M=5.7 scenario before (BR) and after (AR) retrofitting.

8. Conclusions

In this project, we performed a pilot study to compute loss scenarios in the city of Basel. A comprehensive estimation of the uncertainties has been performed in order to better understand the sources of uncertainty. The performed scenarios are more precise than previous studies and we also showed their robustness. We showed that the uncertainties are large, in the same order of magnitude as the mean values. In the future, the uncertainty in each individual part of the computation has to be decreased to improve the results.

The ground motion prediction equations remain the most uncertain part of the analysis though large improvements have been done in the last ten years, especially thanks to recordings of the Swiss strong motion network (Wiemer et al., 2015). There is still room for improvements through densification of the network and site characterization. The epistemic uncertainty in the ground motion modelling, not accounted for in our project, has to be relativized for scenario modelling since neither the location nor the magnitude of future events are known and they are therefore arbitrary fixed in our computations. The uncertainty in the choice of the ground motion model is therefore relative and could be balanced for instance by a small magnitude increment. The Swiss stochastic model (Edwards and Fäh, 2013) calibrated with Swiss data is currently the optimal choice for scenario modelling.

Spatial correlation of ground motion plays an important role in the uncertainty estimation and inter-period correlation should be introduced as well in the future.

We showed that site amplification has a critical impact on the loss scenarios (factor 8 on the fatalities for the 475 yrs. return period) but that the amplification in the city centre is relatively well known since the 2006 microzonation. Improvements have been however made in this project, particularly for areas outside of the Rhine Graben and the robustness of the model has been improved. These improvements are related to the recordings of the SSMNet in Basel despite the low seismicity in the last years. The amplification in the Loess hills in the South of the city, poorly understood, should however be a target for future studies. Although the impact on the amplification is not critical, the interpretation of dispersion and ellipticity of surface waves retrieved under ambient vibration remains a challenge. These issues could be solved with additional measurements in the first 100 m including anisotropy and improvements of the 3D model. Studies of the wavefield composition of earthquakes and ambient vibrations should also help improving these issues.

We proposed a new method to fully propagate the uncertainties in the capacity curves to the fragility curves for a given scenario event. We showed that this method was able to better represent the uncertainties, which are for instance larger at larger ground motions. Moreover, we showed that currently used seismic analysis methods were too conservative, i.e. biased towards higher
vulnerability, to be directly used for risk scenarios and that conservativeness needed to be first removed. However, the estimation of epistemic uncertainties for vulnerability models remains a challenge. The curves for some building types (e.g. RC_P and MRC_MI_SS) should be developed further to consolidate the results. We also recognized that the data on fatality rates in the literature were limited and that more studies would be necessary to improve them, including non-structural elements and fatalities outside of structures.

We performed a number of selected damage scenarios on the school buildings of Basel and evaluated for each the distribution of damage, the number of fatalities, injured and homeless pupils and the financial losses. The 475 yrs. return period event (M=5.7) is unlikely to cause the total collapse of a school building but tens of fatalities should be expected as well as large financial losses. A large amount of the buildings would be moderately damaged, meaning that they would need a quick analysis and repair, which can be challenging considering the large amount of affected structures.

The results are obviously very different depending on the scenario itself (magnitude, distance, depth...), but this information could be obtained shortly after an earthquake and the scenarios computed in near real-time.

Despite the large uncertainties, we showed that the retrofitting measures had a noticeable impact on the results (50% less fatalities for the group of retrofitted buildings). The strategy of retrofitting in priority the weakest structures with the largest number of occupants is proven to be efficient. However, retrofitting does not mean reducing the losses to 0, even for the 475 yrs. return period scenario. A more detailed analysis on the retrofitted buildings is however necessary to be able to precise this statement.

This model is a solid basis for extending loss scenarios to the whole city, eventually with automatic computations shortly after an event. For that purpose, the amplification model should be slightly improved, and more building types should be proposed together with a comprehensive inventory.

Aknowledgements

The authors thank the SED network that is distributing the data used in this study, Benjamin Edwards who designed and maintained the tool to compute the ESM function as well as Marco Pilz who took over this task. Data from GeORG project have been used (www.geopotenziale.eu). We also thank Pia Hannewald and Pierino Lestuzzi for the fruitful discussions and the review of parts of this report as well as Blaise Duvernay and Stephan Husen for the reviews. Finally, this report has been written with contributions from Benjamin Edwards (section 3), Carlo Cauzzi (section 3.5), Helen Crowley (section 4) and Simona Esposito (Appendix B).

References


Conclusions


Teilbericht 5: Bestimmung der zonenspezifischen Antwortspektren: Abschlussbericht: Teilbericht B


Appendix

Appendix A: Combined distribution of correlated variables

In order to describe several correlated variables following a normal distribution, one has to know the vector of their means ($\mu$) and the variance-covariance matrix [C] (variances on the diagonal, covariances in the other parts of the matrix)

The covariance between two variables with data vectors $X_1$ and $X_2$ is estimated using the equation:

\[
cov_{1,2} = \frac{1}{n^*} (X_1 - \mu_1)^T (X_2 - \mu_2)
\]

with $n^*$ being n the number of data points if the mean is known, n-1 if the mean is also estimated. $cov_{1,2}$ is varying between 0 (no covariance) and $\sigma_1 \sigma_2$ (full correlation)

In order to simulate a realization $x$ of such multi-variate distribution, one can use the following:

\[
x = \mu + L v
\]

with $C = LL^T$ the Cholesky factorization of the variance-covariance matrix and $v$ a vector of random values (normal distribution of mean 0 and variance 1, size of the vector corresponds to the number of considered variables).

Example with two variables

\[
C = \begin{pmatrix}
\sigma_1^2 & cov_{1,2} \\
cov_{1,2} & \sigma_2^2
\end{pmatrix} = LL^T \Rightarrow L = \begin{pmatrix}
\sigma_1 \\
cov_{1,2} & \sigma_2
\end{pmatrix} \begin{pmatrix}
0 \\
\sqrt{\sigma_2^2 - cov_{1,2}^2} / \sigma_2
\end{pmatrix}
\]

Application: If the value of the spectral displacement is known at period $T_1$, what is the expected distribution of the spectral displacement at period $T_2$?

\[
\begin{cases}
S_d(T_1) = \mu_1 + \sigma_1 v_1 \\
S_d(T_2) = \mu_2 + \frac{cov_{1,2}}{\sigma_1} v_1 + v_2 \sqrt{\sigma_2^2 - \frac{cov_{1,2}^2}{\sigma_1^2}} = \mu_2 + \frac{(S_d(T_1) - \mu_1)}{\sigma_1^2} cov_{1,2} + v_2 \sqrt{\sigma_2^2 - \frac{cov_{1,2}^2}{\sigma_1^2}}
\end{cases}
\]

So the distribution at period $T_2$ is a normal distribution with the following mean and standard deviation:

\[
\begin{cases}
\mu_2' = \mu_2 + \frac{(S_d(T_1) - \mu_1)}{\sigma_1^2} cov_{1,2} \\
\sigma_2' = \sqrt{\sigma_2^2 - \frac{cov_{1,2}^2}{\sigma_1^2}}
\end{cases}
\]

Example with more variables

We assume that N=10 buildings have individually an expected damage grade 3 with a standard deviation (uncertainty) of 1 damage grade. We want to estimate the expected mean damage for these 10 buildings considered together (mean of the individual distributions). We assume either no dependency between the N buildings (Normal distribution of mean 3 and standard deviation 1/N) or a high correlation (0.9) between the N buildings (if one has DG4, another one has more chances to have DG4 as well). Fig. 51 shows that the resulting damage distribution has a much larger spread: the correlation increases the uncertainty on the aggregated results.
**Fig. 51** Example of the effect of correlation: mean damage for N independent buildings without correlation (blue) or with a correlation of 0.9 (orange).
Appendix B: Towards a spatial correlation model for Switzerland

Following Jarayam and Baker (2009), we studied the spatial correlation of the intra-event residuals from the Swiss Stochastic model (Edwards and Fäh, 2013). The Swiss Stochastic model is fitting spectra of the recorded earthquakes in Switzerland based on a functional form defined by a simple model of source physics. For each considered frequency, the intra-event residuals are the difference (L2 norm) with the model for each studied earthquake. Events between the years 1999 and 2009 were used here (see Edwards and Fäh, 2013 for the event list).

The recording station pairs are first grouped into classes having similar inter-distances. The semi-variograms are computed for each frequency (Jarayam and Baker, 2009): they correspond grossly to the correlation of the residuals for each inter-distance class. For one event \( i \), the empirical semi-variogram is computed as follows:

\[
\gamma_i(h) = \frac{1}{2n_{h,i}} \sum_{h-\delta h < d(S_1,S_2) < h + \delta h} (r_i(S_1) - r_i(S_2))^2
\]

With \( S_1 \) and \( S_2 \) stations located within the inter-distance \( h \), \( n_h \) the number of station pairs within this inter-distance class and \( r(S_1) \) the residual at station \( S_1 \).

However, a large number of events \( i \) is available and one can assume that they represent the same process. It is also possible to normalize each event by its variance. The number of available data points should however be used to weight the semi-variance for each event. Therefore, we computed the semi-variogram as follows:

Either normalized \( \tilde{\gamma}(h) = \sum w_{h,i} \gamma_i(h) \) or not \( \gamma(h) = \sum w_{h,i} \gamma_i(h) \)

with \( w_{h,i} = \frac{n_{h,i}}{\Sigma n_{h,i}} \) and \( \sigma_i^2 \) the variance of the residuals for event \( i \). The number of pairs \( \Sigma n_{h,i} \) for one inter-distance \( h \) should ideally be larger than 50 because these pairs are not independent.

It can also be written:

\[
\gamma(h) = \frac{1}{2 \Sigma n_{h,i}} \Sigma \left[ \sum_{h-\delta h < d(S_1,S_2) < h + \delta h} (r_i(S_1) - r_i(S_2))^2 \right]
\]

In theory, the semi-variogram is starting at 0 for 0 inter-distance (perfect correlation) and increasing up to the total variance of the process \( \sigma_0^2 \) or "sill" (if it is stationary) for longer inter-distances. If the residuals are already normalized by the sill of each event, the value for large distances is 1. Several fitting models are commonly used to reproduce the behavior of the semi-variogram. As proposed by Jayaram and Baker (2009), we used the exponential model:

\[
\tilde{\gamma}(h) = \sigma_0^2 \left( 1 - e^{-\frac{h}{h_0}} \right)
\]

or \( \gamma(h) = 1 - e^{-\frac{h}{h_0}} \)

This model has only one free parameter \( h_0 \), the inter-distance above which the data are nearly uncorrelated (0.05% correlation), called "range". If the experimental semi-variogram was not normalized by the intra-event variance, we computed the sill as the total variance of all intra-event residuals. The inversion of the \( h_0 \) parameter is performed with least square fit (L2 norm). Weighting the least square with the number of points in each class is critical to stabilize the results.
Fig. 52 Raw (top) and normalized (centre, bottom) experimental semi-varioogram of the residuals (blue) and fitted exponential model (green) at 2Hz, 1Hz and 0.5Hz frequencies (from left to right) for $\delta h=10$km and 2.5km

The above figure is presenting the results for different frequencies and $\delta h$ of 10 and 2.5 km. Normalized and raw semi-variograms are similar showing that the variance for each event is very close to the total variance, although the normalized values provide better results. The so-called “nugget effect” in observed in the experimental semi-variogram (i.e. semi-variance is not decreasing to 0 at zero-distance) is due to the averaging over too large inter-distances values. For smaller values of $\delta h$, this effect disappears. For $\delta h=2.5$ km, there are not enough couples of stations at low inter-distances resulting in more scattered semi-variograms.

Finally, the range $h_0$ is plotted over the whole considered frequency band and compared to the results of Jayaram and Baker (2009) and Esposito and Iervolino (2012) in the figure below. This last model has been established using the European Strong-Motion Database. Though the uncertainty on these models is not provided, the standard deviation is in the order of 10 km. The model of Jayaram and Baker (2009) is based on 7 events only but also accounts for clustering of VS$_{30}$ values (expected for sedimentary basins) that has an effect at high frequencies. Such a clustering is expected for Switzerland.
These results show that the range is slightly decreasing over the whole period range with values between 30 and 50 km, indicating that a spatial correlation is measurable up to this inter-distance. The values are comparable to the model of Jayaram and Baker (2009) or Esposito and Iervolino (2012) within the uncertainties. The increase of the range with increasing period found by Jayaram and Baker (2009) and Esposito and Iervolino (2012) can be recognized with the smallest $\delta h$ values but we did not investigate long periods. However, the range at $T=2$ s is consistently large (70 km) compared to these models. Jayaram and Baker (2009) suggest a decrease of the range for low periods in case of clustering of $V_{s30}$ values. This feature is not clearly reproduced, the values of range seem stable at low periods. The models of Esposito and Iervolino (2012) and Jayaram and Baker (2009) without $V_{s30}$ clustering show on the contrary a rapid decrease of the range with increasing frequency. Our observations lie therefore in-between these two models.

This study shows that the intra-event residuals of the stochastic model for Switzerland are spatially correlated up to a distance of about 30-40 km. The existing models can be used for Switzerland or a new model could be proposed.

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