Guidelines and strategies for seismic microzonation in Switzerland

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Introduction

1. Seismic microzonation – an overview

One of the most severe modifications of the earthquake signal comes from the effect of the uppermost few hundred meters of the earth structure, where the larger variability of the geological and geophysical conditions have a considerable impact on the amplitude and duration of the ground motion, potentially increasing the severity of the earthquake's effects on environment and population.

Different phenomena can contribute to the increase of the seismic hazard at local scale. Trapping and constructive interference of seismic waves in sedimentary basins is the most important phenomenon responsible for large amplifications during an earthquake. Complex topography and buried geometrical structures can contribute as well through focusing/defocusing effects and the generation of specific phenomena, such as edge generated surface waves. The local environment is also vulnerable to earthquake-induced phenomena such as ground failure (subsidence, liquefaction, landslides) and earthquake-triggered flooding (natural dams, tsunamis) (e.g. Fäh et al., 2012). However, not all of these phenomena contribute equally to the increase in hazard potential, and large variability can be expected over relatively short distances.

Characterization of local conditions is recognized to be an important procedure in the definition of site-specific ground shacking hazard. Site variability has a direct impact on seismic hazard at local scale and can considerably contribute to the increase of damage level during an event. Therefore, an accurate assessment of the local geophysical/geomechanical properties at the site together with a reliable quantification of the local seismic site-response is of fundamental importance also in seismic risk analysis.

Within this framework, *seismic microzonation* is defined as a set of well-structured procedures aimed to the discrimination of areas (zones) susceptible to specific local effects and of possibly uniform seismic hazard potential. Such assessment is essential for the use in building codes, loss mitigation, land and urban planning and emergency response in case of catastrophic events.

Main criteria for the definition of the micro-zones are the variability of the predicted or observed *seismic site-response* and the vulnerability of the physical environment through *earthquake-induced effects*. The size of derived micro-zones is then commensurate with:

- required level of accuracy,
- monetary investment for the direct investigations,
- type of local effects being examined.

These aspects have also a direct impact on the model uncertainty, which has to be quantified. An insufficient number of investigations lead to a low level of knowledge and consequently to a high epistemic uncertainty.

2. Site-response analysis in engineering practice

Probabilistic or deterministic seismic hazard assessment provides an estimation of the expected ground motion at standard reference conditions (the engineering rock reference). For a sediment site or for different rock conditions, such estimate has to be then corrected for the local seismic site-response, or amplification spectrum, relative to such reference. In fact, during earthquakes, the effect of local amplification can be comparable with the differences in the hazard estimate between distant regions.

The definition of a site-specific hazard is therefore needed for two complementary reasons: firstly because it allows the assessment of more realistic ground-motion estimates for the calibration of the predictive model. Secondly, fine-tuning the hazard prediction at a local scale ultimately leads to a decrease in the epistemic uncertainty of the prediction, which is of fundamental importance in engineering hazard and risk analysis.

It is well accepted that without the knowledge of the local soil condition, ground motion recordings and derived attenuation models are not used in effective ways. In order to reduce uncertainties in regional ground-motion prediction equations (GMPEs, which are essential components of probabilistic seismic hazard assessment), recordings at seismic stations need to be related to a corresponding measured shear-wave velocity profiles. Such assessment allows an appropriate comparison of the ground motion recorded at different ground conditions and the subsequent proper scaling to the regional rock reference. The subsequent calibration of site-specific GMPEs allows then producing ground motion estimates properly adjusted to the specific response of a site of given characteristics (e.g. Edwards and Fäh, 2014), which ultimately helps reducing the level of uncertainty of the prediction and the resulting level of conservatism to be applied (often highly subjective).

3. Goals and organization of this document

The aim is of this document is to present an up-to-date overview of best practice in seismic site-response analysis and microzonation. The different consecutive steps of a microzonation are presented, and the main implementation procedures explained using progressively increasing details, starting from the general concepts to the practical recommendations for the end-user.

Although microzonation analysis is here presented in direct relation to their possible effect on structures and population (risk-oriented analysis), no explicit reference to normative or design criteria is included. The primary target of this document is to summarize and promote state of the art in microzonation practice for hazard and risk studies in Switzerland targeted to scientists, practitioners and decision makers.

In *chapter I*, the basic concepts for seismic microzonation at different scales are first provided. Three different levels of analysis are discussed and a summary description of the corresponding implementation procedures given. These topics will be developed in more detail in the subsequent chapters. In *chapter II*, a selection of most significant phenomena causing site-effect analysis are presented, while *chapter III* provides a summary of the most common site-investigation techniques relevant for the characterization of such phenomena. In *chapter IV* an overview on ground motion modelling techniques is given. Finally, in *chapter V* we discuss the use of ground motion observations for the verification of the seismic response model.

Chapter I

Seismic microzonation

1. State of the art and national normative

Microzonation is nowadays a common practice in seismic hazard analysis. It has been applied to many risk-prone areas all over the world. Although a considerable amount of application examples can be found in literature, both from research institutes and private contractors, a general lack of homogeneity is evident in analysis procedures and implementation criteria used. Microzonation output is also not always comparable between different studies: in some cases an exhaustive numerical output is provided (e.g. amplification and f_0 maps, landslide and liquefaction maps), whereas in other cases only qualitative interpretation is given.

The major reason of this variability is that only few countries provide clear guidelines and recommendation for practical implementation of microzonation; more importantly none of these countries - at least in our knowledge - have implemented any normative, clearly regulating the criteria for microzonation in risk analysis, which often remain a pure academic exercise. Among countries with guidelines it is worth to mention Italy (ICMS, 2008), France (AFPS, 1995), Japan (ISSMGE, 1999), California (California Division of Mines and Geology, 1992, 1997) and Turkey (Studer & Ansal, 2004). Italian civil protection, in particular, provides a manual about best engineering practice, including a set of recommendations on implementation criteria and some case-study examples. As well, the French Ministry of Environment offers a short but rather comprehensive document with guidelines, which is likely to be revised in the near future within the new *Seismic Risk Prevention Plan* (PPRS). Surprisingly in other countries, such as Germany, the research field is advanced at academic level, but no national recommendations seem to be available. In Switzerland, a first tentative of microzonation guidelines was done by Mayer-Rosa & Jimenez (2000); this report provides a comprehensive overview on microzonation analysis and a useful selection of examples, but it has to be revised in light of new technological advances in the field.

Present national building codes (e.g. European EUROCODE8 and Swiss SIA261) include a section related to soil type definition used to select the design spectrum that should account for local site response. Even though the adopted level of conservatism of the type 1 design spectrum is quite high, in particular at long periods of ground motion, such approach is not sufficient to describe the complex interaction of the ground motion with the local geology and does not give any measure of uncertainty. A microzonation, on the contrary, can provide a more realistic representation of the local seismic response for design of buildings, with a potential to decrease the conservatism in the design. In Switzerland, site-specific or microzonation studies can replace the spectra in the building code; however, implementation criteria are not yet established at national scale, so that each canton (or even building owner) has the possibility to independently decide which action to undertake for defining design spectra (Cartier, 2007). This is related to the legal issues and liability as discussed in *SIA-Documentation D 0227* (2008), however never explicitly addressed in the context of microzonation in Switzerland. There are plans to improve the situation by adding rules for microzonation in the building codes.

2. Levels of site-response analysis for hazard studies

Effects of local geology on ground motion can be assessed at different scales and resolution. In general, it is possible to define at least three distinct levels: *level I* (**national**), *level II* (**regional**) and *level III* (**local**). Each scale is characterized by a different detail of investigation and consequently different resolution of the result. This affects the related uncertainties of the seismic site-response analysis, which is then very high at national scale and progressively reduces at the other levels.

It should be stressed that the three levels are not independent. A clear separation cannot be uniquely defined and the levels are in some cases overlapping. For example, results from a level I zonation are preparatory to level II for the identification of areas of potentially similar characteristics in terms of site-effect potential. In turn, a level II study cannot be accomplished without performing a sufficient number of investigations of level III. Complementary, output of the level I itself is calibrated on results from previous level II and III studies. This circular dependency highlights also the fact that site-response analysis is not a static procedure, but is in fact a continuously evolving process also driven by new studies, on-going progress in scientific research and observations during future earthquakes; existing methods and results must be regularly updated with the availability of new data and further technological advances.

2.1. Level I, national scale

The national scale is aimed to the subdivision of the national territory into large zones (*domains*) of comparable seismic response. Zone separation is done on the base of geological (sedimentology, lithology and tectonic), geotechnical and geomorphological considerations, which define a number of discrete proxies for the zonation (the use of any continuous proxies is impractical at this resolution, see Appendix A). Each domain is associated with an average seismic response and a model for the *aleatory and epistemic uncertainties*. One of the goals in land-use planning is the identification of areas where important ground motion alteration is expected. Based also on risk considerations some of those areas would need a level II study to reduce epistemic uncertainties.

Defined proxies are calibrated over a large number of punctual observations and express an average seismic behaviour and the associated uncertainties. Even if they incorporate information about different site-specific wave phenomena, the selection criteria should be the expected level of amplification.

Calibration of proxies should be primarily done on the base of empirical ground motion observations, such as residual analysis of observed macroseismic intensities or ground motion amplifications. Results from numerical modelling can also be considered in particular cases (e.g. from microzonation studies in level II). In all cases, amplification values have to be referenced to a common reference-rock, for which a geophysical description must be provided. Output of a level I zonation can be a single parameter, as in the case of macroseismic intensity residuals and amplification in terms of PGA, PGV, CAV or Arias intensity, or more advanced as a function of spectral ordinates. In all cases, epistemic and aleatory uncertainties need to be defined with reproducible procedures.

A useful application of the level I zonation is in combination with regional probabilistic seismic hazard maps for risk calculation at national scale, where a layer of qualitative location modifiers (in PGA, CAV, Arias intensity or spectral ordinates) can be superimposed to the ground motion estimates at reference-rock condition. Such approach is useful to provide a first-order representation of the variability of seismic hazard due to the soil effects. An important *caveat*, however, is that results from the national scale zonation cannot be used for the evaluation of local amplification at a specific site or region without special assessments. Due to the large variability of the ground motion averaged in the process of defining the spatial proxies, any attempt of extrapolation from national to local scale might lead to significant bias in the prediction and must be avoided. It should be clear to the end user that a reliable local response might not be necessarily represented by the use of national location modifiers or proxies without additional work at level II. Introducing low resolution in maps at level I is a recommended action to prevent misuse of such maps (e.g. ShakeMap implemented at SED or aggregation of results in soil classification schemes).

On a national scale, macroseismic data can be used for the compilation of the EMS intensity amplification map (Kästli and Fäh, 2006). The amplification factors are defined relative to the ECOS (Earthquake Catalogue of Switzerland) macroseismic attenuation relation (Fäh et al., 2003) and calibrated on geological soil classes as documented in Appendix D in Fäh et al. (2011). This macroseismic amplification map was implemented in the ShakeMap tool at SED, and tested in combination with the Swiss stochastic ground-motion prediction equation, in the sense, that we can reproduce the general shape of observed macroseismic fields of the strong historical earthquakes in Switzerland (Cauzzi et al., 2015). An application is given in Figure 1.

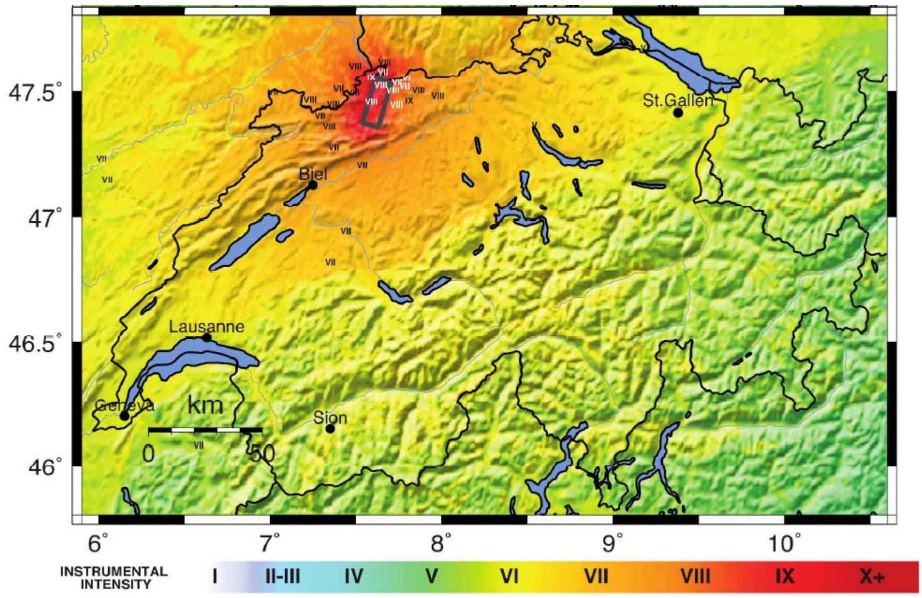


Figure 1. Shakemap for an earthquake scenario similar to the historical Basel Mw 6.6 event in 1356 (modified from Cauzzi et al., 2015). The local soil condition is taken into account using an amplification map based on observed macroseismic intensities. Resolution is very limited to prevent over-interpretation. This scenario modelling was used for an exercise of civil defence in 2012 (SEISMO12).

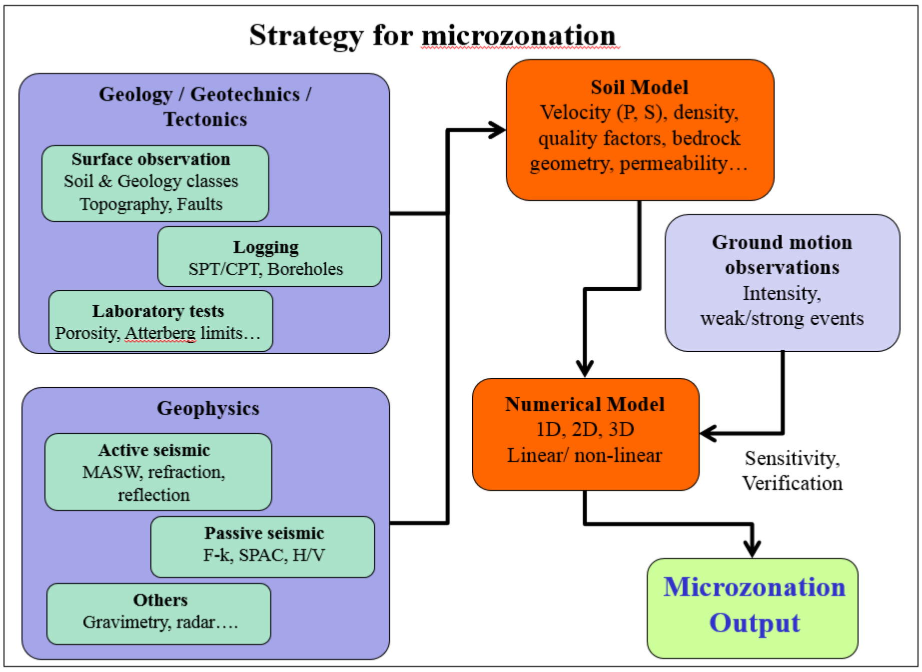
2.2. Level II, regional scale

The regional scale involves more complex and time-consuming investigations. This level is usually considered as "*microzonation*" in most national normative. It effectively subdivides the study area in sub-zones of (supposedly) homogenous seismic response. The size of the target area can be quite different, ranging from a canton to the size of a small city; the dimension of micro-zones is better commensurate with the level of detail necessary to describe all the major features of the local site-response, including earthquake-induced phenomena. Additional factors can be considered in the zonation scheme, when available, as in the case of evidences of active faults with surface expression (see Chapter III).

A level II study is performed in four consecutive steps:

- Data mining and direct investigations
- Scenario definition and seismic site-response analysis
- Model verification with ground motion recordings
- Final interpretation and zonation

The workflow is however not linear; often initial assessments undergo some revision and are updated depending on the outcome of the subsequent steps. In the following, we provide a short description of each step, whereas a detailed discussion is provided in a separate section.



2.2.1. Data mining and direct investigations

In the first step, all available information for the target area is collected, with particular focus on existing geological, hydrological, geotechnical and geophysical studies and datasets. Reports from cantonal offices and municipalities should be obtained, as well as any accessible data and reports owned by private companies. An overview of available data in Switzerland is given in Appendix A. Historical information should also be analysed to identify any evidence of significant site-effects during past events (including damage distribution and reports related to induced phenomena such as ground cracking, liquefaction and triggered landslides). This first phase is preparatory to the identification of areas of potentially high site-related hazards, and useful for the planning of the subsequent steps.

In a second step, a number of punctual invasive and non-invasive investigations (level III studies) are performed to obtain information on the subsoil structure at selected "strategic" sites. Relevant parameters to be determined are primarily the fundamental frequencies of resonance (f_0) and characteristic shear-wave velocity profiles (Vs) or the elastic moduli. Density, shear resistance, porosity and permeability, average cohesion and friction angle for the geological units are other parameters that might play a role. Beside material properties, additional information should be assessed such as topography/geomorphology, the geometry of the major geophysical interfaces (e.g. soil-bedrock) and of the ground-water table levels.

Due to the high investment costs of this step, the number and distribution of geotechnical and geophysical investigations is generally restricted to a selected number of sites considered to be representative of the most significant geological conditions of the area. Extrapolation to areas not covered by direct measurement should be done using appropriate discrete or continuous proxies, such as geological/geotechnical classification, combined with additional punctual investigations such as horizontal-to-vertical (H/V) spectral ratios (see chapter III). An important aspect of the extrapolation is a transparent assessment of the epistemic uncertainties.

2.2.2. Scenario definition and seismic site-response modelling

The collected information is summarized in an interpretative geotechnical/geophysical model of the study area. The model can be used for the evaluation of the seismic site-response using analytical or numerical ground-motion simulation techniques (see chapter IV). When the level of confidence in the available calibration data is low (large uncertainties on model parameters) or when not sufficient information is collected, different models corresponding to different interpretations have to be proposed to represent the associated epistemic uncertainty.

Basically two main types of numerical analysis are relevant in microzonation: *wave propagation simulations* and *geomechanical modelling*. The goal of the first category is to directly reproduce the interaction of the incident seismic wave-field with the local structure, in order to quantify the ground motion amplification (in amplitude, duration and polarization) and its variability over the study area. In turn, geomechanical models are used to verify the possibility of earthquake-induced effects under dynamic loading. This includes the analysis of slope instabilities and the assessment of liquefaction potential, lateral spreading and ground subsidence. The two modelling schemes are however not independent; geomechanical models often rely on the results from wave propagation simulation for the definition of the input ground motion.

The complexity of the numerical model is defined by the characteristics of the study area and the data available. In the case of seismic site-response analysis, one-dimensional (1D) models are an acceptable simplification in cases of basins with a large width/depth ratio and where the existence of geometrical effects such as edge-generated surface waves can reasonably be excluded; smooth variations of the soil properties within the basin are also an essential requirement. Note that the feasibility of using a one-dimensional model has to be demonstrated prior to the implementation, for instance by means of mapping the variability of the soil properties using geological information and H/V spectral ratios. A two-dimensional modelling approach can be used in cases of narrow elongated basins, such as alpine valleys, where seismic response is strongly controlled by the interface between sediments and bedrock. Finally, a three-dimensional model is to be used in all other cases. Other factors affecting the result of numerical modelling are the definition of the incident wave-field, vertical or oblique incidence versus a more realistic incidence by simulation of the seismic sources. Selection and use of several methods and the related influence on the result is part of the assessment of epistemic uncertainties.

Numerical simulations are also bound to the definition of a particular earthquake scenario, representing one or a set of events whose characteristics (e.g. magnitude, peak ground motion, style of faulting, distance and depth) are likely to occur within a given return period. Criteria for the selection and implementation of a scenario are discussed in the Appendix A and are mainly based on the de-aggregation of the probabilistic seismic hazard at the defined return period. Finally, numerical results in terms of amplification obtained from different models and scenarios are collected, validated and analysed to map the variability of the ground motion at particular sites and over the study area. Aleatory and epistemic uncertainties have to be defined and documented.

2.2.3. Model verification with ground motion recordings

Comparison of the modelling results with direct ground motion observations is an important step that allows the validation of the models and the reliability of the numerical simulations, and the weighting related to the epistemic uncertainties. This can be done in different ways and with different levels of confidence, which is then however affecting largely the final epistemic uncertainties that need to be taken into account in the hazard and risk calculation.

With the advent of modern technologies, temporary networks are probably the most cost-effective way for model verification. If a sufficient number of reports are available for a well-studied event, numerical results can be validated with macroseismic observations. This approach is mandatory if sufficient data is available, however rather qualitative, as it makes use of empirical relations to convert ground motion to intensity estimates. A comparison of modelled amplifications with earthquake recordings from permanent or temporary seismic stations is preferred and highly recommended. In turn, the limited duration of a temporary installation might limit the number of recorded events, especially in low seismicity regions.

The comparison can be done on the base of ground-motion measures such as PGA, PGV, Fourier spectra and waveforms, but only if the modelling scenario corresponds to the recorded event. More commonly a “relative” comparison is performed, where a particular ground motion estimate (generally the Fourier spectrum, but in few cases also peak ground values, response spectral values) is defined in terms of amplification to the corresponding value at the rock-reference, a site assumed free of site-effects. Such approach effectively removes the effect of the seismic source from the observed ground motion and long-distance propagation, and highlights the effect of the site. Moreover, being the relative comparison independent from the selection of a specific scenario, a large number of events of different size can be compared. This assures a robust statistic and a better representation of the aleatory variability of the site-response estimate. Examples of such techniques and criteria for the definition of a proper local and regional reference will be provided in Chapter V.

2.2.4. *Final interpretation and zonation*

Zone subdivision is an expert task, which might require a certain amount of subjective decisions. Such zonation also depends on the goal of the final zonation and uncertainties affect these decisions. No unique or general rule can be provided, but a number of recommendations are useful to guide the final choice. Microzonation studies are generally performed to provide site-specific design spectra related to the building codes. They often contain conservative decisions. Decisions and related data therefore need to be documented in a reproducible manner.

In order to be useful for seismic risk computation several criteria need to be fulfilled in level II studies. Important criteria for the zonation are the average amplification level (observed or predicted) together with the uncertainties, the possible occurrence of non-linear phenomena and the potential for induced effects. While the amplification can be represented quantitatively, earthquake induced phenomena are more difficult to describe. Each phenomenon can be mapped separately over the study area, e.g. by using an appropriate thematic map or *layer***,** also assigning addition information that might be useful for the end-users.

In the final zonation, average amplification is combined with the regional seismic hazard, whereas double counting of aleatory variability has to be avoided. Moreover, microzonation products/results should also allow for the simulation of specific deterministic scenarios, in particular when applied for risk calculations. A probabilistic or deterministic calculation of induced effects would require additional elements that relate ground motion to strain, and strain to non-linear soil response, to liquefaction and landslide triggering. Such elements are generally not included in level II studies, but might become more quantitative in the future as an outcome of on-going research.

2.2.5. *Some studies performed in Switzerland*

Several site response studies are available from SED. For other studies, e.g. by private companies, it is often not clear if the relevant information and data were derived and are accessible. The level of detail of these studies is different, ranging from complete regional microzonation (e.g. Basel area in Figure 2) to local models.

Among these, it is worth to mention:

- Microzonation of Basel (Fäh & Huggenberger, 2006)
- Multi-hazard analysis of the Visp area within project COGEAR (Burjanek et al., 2012, Fäh et al., 2012)
- Preliminary microzonation analysis of Lucerne (Poggi et al., 2012)
- 3D model for the region of Sion (Roten et al., 2008)
- Several microzonation studies based on engineering methods (e.g. studies of Resonance SA for the cantons VS and VD)

Microzonation studies are generally performed to provide site-specific design spectra related to the building codes. They often contain conservative decisions and the results cannot be directly used for site response analyses. The problem with such studies might be that intermediate results are not stored, and that expert assessment is used instead of geophysical measurements. A prerequisite for microzonation studies to be useful would be the compliance to a set of rules, the elaboration of intermediate results and the verification with earthquake recordings.

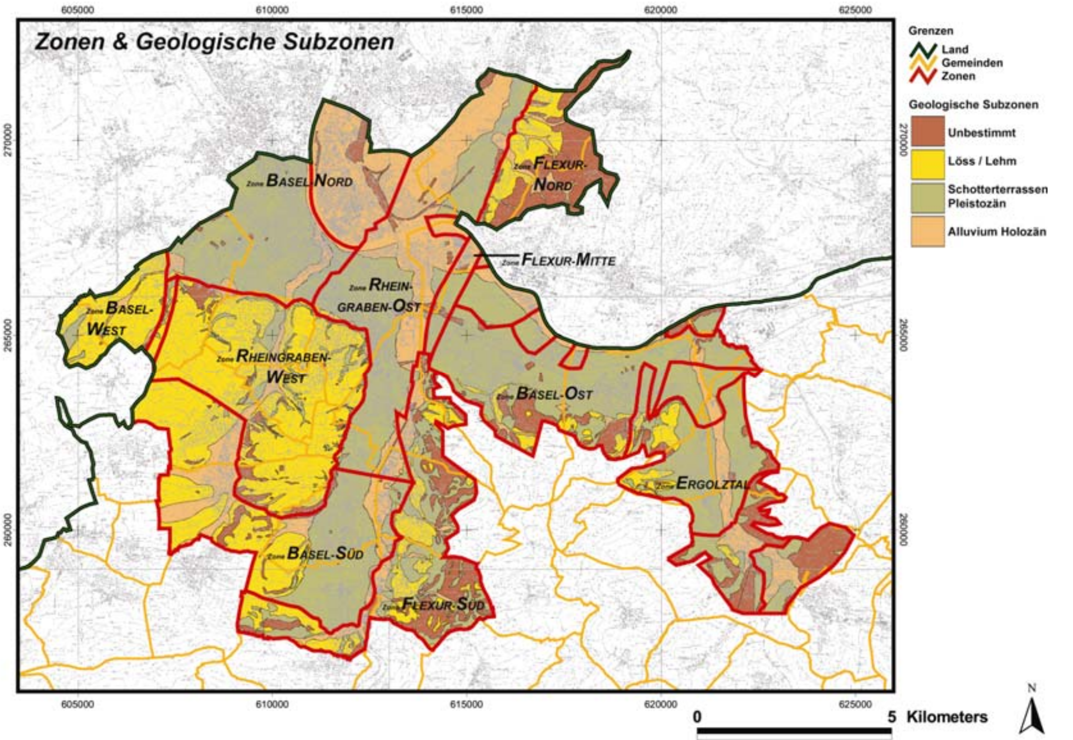


Figure 2. Example of a microzonation map for the wider Basel area (Fäh & Huggenberger, 2006). For each zone a specific design spectrum is proposed based on numerical modelling and observed ground-motion amplification (Fäh & Wenk, 2009).

2.3. Level III, local scale

Level III implies detailed investigations at a specific site in a limited area around the site, usually in the order of few tens to hundreds of square meters. This kind of analysis is generally performed complementary to a level II study for the calibration of the model, and can be necessary for those areas of expected high amplification and areas susceptible of earthquake-induced phenomena, such as soil liquefaction, subsidence or slope instability. A detailed level III investigation is essential in case of lifeline structures or critical facilities (e.g. dams, nuclear power plants). Methods and recommendations for level III studies are outlined in Chapter III.

The goal of a local scale study is to provide a continuous log of the geological, geophysical and geomechanical properties of the soil by means of invasive (borehole measurements, sampling and laboratory tests) and non-invasive investigations (mostly seismic methods, as well as resistivity sounding, and gravimetric methods). Output is in most cases a set of one-dimensional vertical profiles (defining epistemic uncertainty) of a set of properties, but local 2D/3D continuous models are also required in case that site-response is considered to be complex. Resolution and maximum depth need to be defined in advance, and are restricted by the selected methods and available financial resources. As discussed for level II studies, such investigations should preferably be complemented with ground motion modelling, and to reduce epistemic uncertainties with the installation of seismic stations to measure empirical site-specific amplification.

3. Definition of engineering output

Working with response spectra appears necessary in light of the requirements from the engineering community, but is actually not a convenient approach in site-response evaluation. This is due to the intrinsic non-linearity in the computation of response spectra, in particular at high frequencies. Instead, the use of *Fourier spectra* is recommended for the different steps of the study, including the numerical modelling, empirical observation and model calibration. An output in response spectra can subsequently be derived by means of stochastic simulations and random vibration theory.

When dealing with response spectra, however, an earthquake scenario needs to be defined. This is a task of the *de-aggregation of the seismic hazard* for the specific return period of interest. Microzonation output can then be computed for a given specific scenario or as average over different scenarios spanning a range of event parameters (e.g. magnitude, distance, fault style and stress drop), different ground motion parameters (PGA, PGV, spectral ordinates) and/or return periods. We have to mention that, even though the first approach is more appropriate, response-spectra amplification factors are generally rather insensitive to small changes in magnitude and distance of the earthquake scenarios, and therefore average estimates might represent a useful simplification in those cases where a unique input scenario cannot be defined.

Chapter II

Seismic local effects

1. Introduction

In this chapter, the main phenomena that are relevant in local site-response evaluation and microzonation are discussed, and the criteria for their identification and quantification presented. The discussion is organized in three major blocks. As first, criteria for identification of faults with seismogenic potential are given. Subsequently, the effect of local geology on the ground motion is analysed, including an overview of the different phenomena. Finally, the seismic vulnerability of the environment is discussed in terms of earthquake-induced (or triggered secondary) phenomena. All phenomena are presented in light of a practical assessment in microzonation. Related characterization techniques are presented but not discussed in detail, as a more complete overview will be given in other chapters.

2. Faults

Identification of local seismogenic structures is an important part of microzonation. Local faults can induce large ground motion in the near-field, and large static (differential) displacement affecting lifelines and the stability of constructions and infrastructure. As a general rule, buildings on or nearby a known active structure should be avoided. A main issue remains the tagging of a fault to be active or not. This depends on the considered return period of the seismic hazard assessment and a definition of activity in clear guidelines.

In a first step, all existing evidences of exposed or buried faults should be collected from any possible source. Information on faults with a surface expression can be obtained from geological and structural/tectonic maps available for the area. Morphological analysis of aerial/satellite pictures and high-resolution digital elevation models (DEMS) is also useful. Shallow faults with large differential displacement can often be recognized by the presence of topographic discontinuities like scarps, offsets or by linear or curved morphological features in valleys, on ridges and riverbeds. Quarry, mines, tunnels, boreholes, trenches or any other sort of excavation can provide further direct evidences of shallow faults, often without direct surface expression. Faults at larger depths can only be identified with geophysical methods (e.g. interpretation of stratigraphic offsets in seismic reflection profiles in which however strike-slip events cannot be identified) or from the analysis of local seismicity (historical and measured through permanent or temporary networks), which can highlight particular "patterns" related to specific tectonic lineaments. In Switzerland, we are far from being able to map all active faults. Only few active faults have been identified so far, among these the Reinach (Figure 3), the Fribourg, the Visp, and the St.Gallen faults to provide some examples. Earthquake data combined with seismic reflection data are probably the most powerful tool to identify active faults. The increase in seismic station density and availability of interpreted seismic reflection data in deep geothermal mining projects or projects such as GEOMOL (in particular the Seismic Atlas of the Swiss Molasse Basin, Sommaruga et al., 2012) will allow improving our knowledge in the coming decades. In particular, active faults should be tracked by direct observation of seismic activity (local monitoring, reinterpretation of earthquakes recordings and of macroseismic fields) and in rare cases by measurement of surface displacement rates (e.g. trough extensometer, laser measure and GPS). Fault size, orientation as well as style of faulting and tectonic regime, if available, should be then specified.

Size and extension of the zone around a guessed or identified seismogenic structure should include the entire fault system, plus a buffer zone dimensioned in relation to the location uncertainty and the extension of the area interested by near-field deformation effects (function of the expected maximum magnitude and fault depth) and possible conjugate fault sets. However, specific guidelines for fault zonation still need to be established.

Figure 3. Fault controlled escarpment in the Birs Valley near Reinach, south of Basel, seen from the East.

3. Effects on ground motion

Local site conditions contribute significantly to the modification of ground motion during an earthquake and their characterization is a key-issue in microzonation. This is particularly important for narrow alpine valleys or sites with low seismic-velocity sediments overlying rigid bedrock. In such condition, one can observe amplifications of factor ten or more in particular frequency bands and an increase in duration of several tens of seconds. Additionally in case of non-linear soil response, the energy can be redistributed over different frequency bands of the spectrum (mostly transfer of energy to lower frequencies), with a chance of matching the dominant resonant frequencies of buildings and structures.

Understanding the way local geology interacts with the ground motion is the first step in site-response analysis. Examples of resulting amplification of ground motion are shown in Figures 3 and 4. Different phenomena can contribute differently to the complexity of the seismic response. Each phenomenon in controlled by a set of specific parameters, which can be quantified through focused analysis. In the following, we present an overview of the most important phenomena influencing the ground motion at the site. Most relevant model parameters are then discussed.

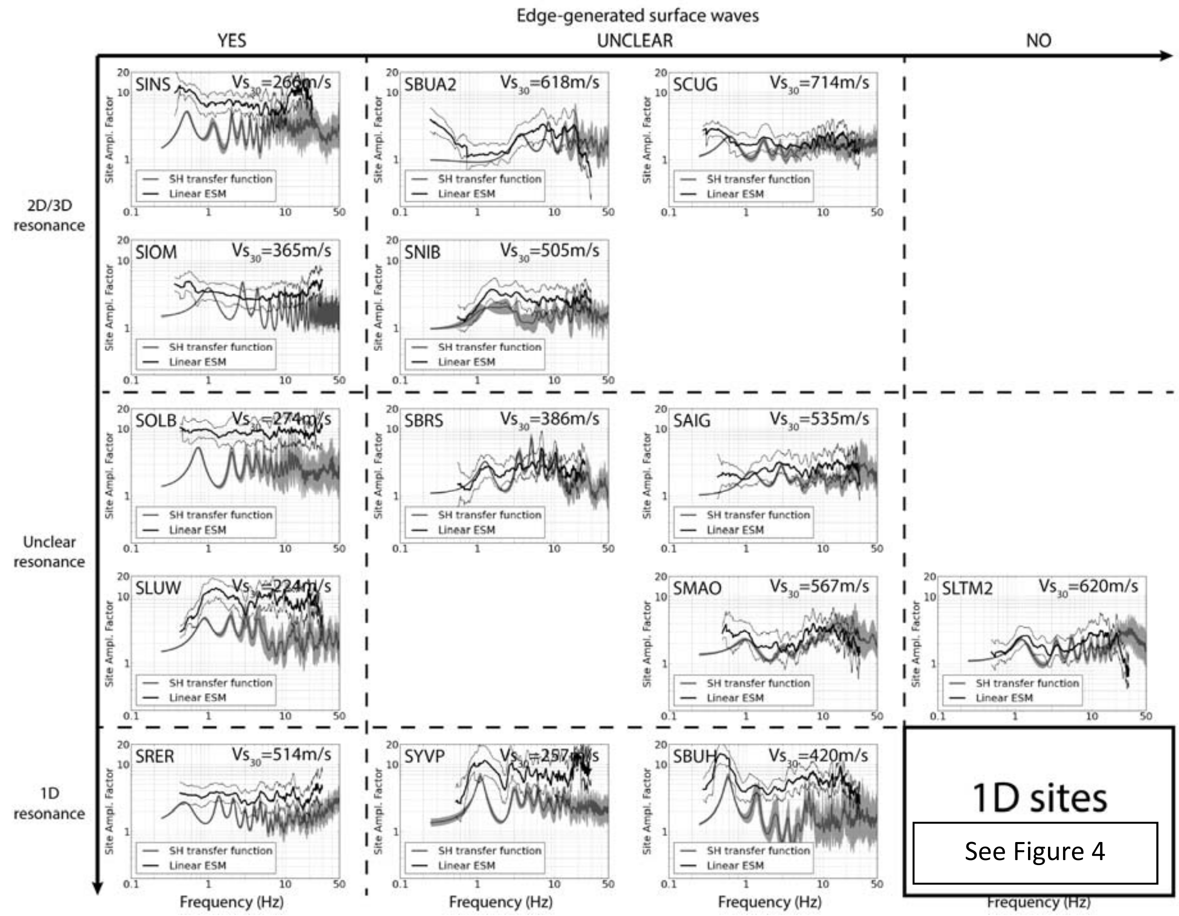


Figure 3. Comparison of observed amplifications at seismic stations using spectral modelling of ground motion with 1D site-response analyses at a number of seismic stations in Switzerland (modified from Michel et al., 2014). Resonance and edge-generated surface waves are contributing to ground-motion amplification. Examples for 1D sites are shown in Figure 4.

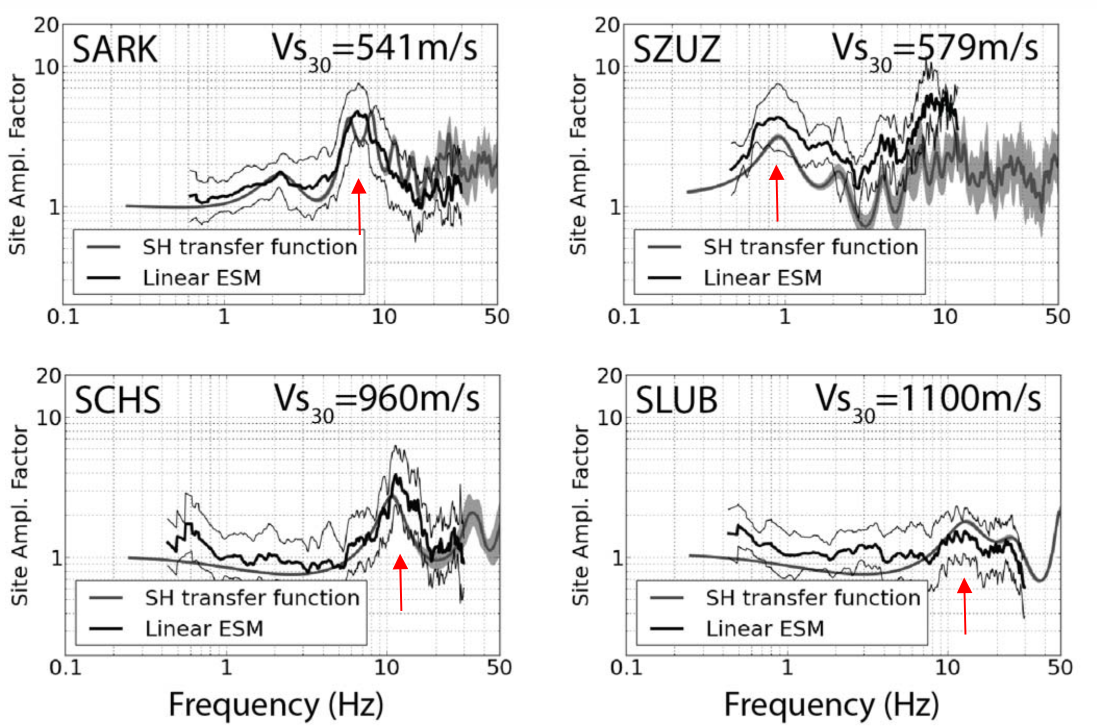


Figure 4. Comparison of observed amplifications at seismic stations using spectral modelling of ground motion with 1D site-response analyses at a number of seismic stations in Switzerland which are characterized by a 1D response (modified from Michel et al., 2014). Red arrows indicate the fundamental frequency of resonance f_0 caused by the strongest velocity contrast.

3.1. Seismic amplification

3.1.1. Response to velocity contrast

Theory of linear elasticity shows that a wave propagating across an interface between two media of different seismic impedance (the product of seismic velocity and density) modifies its amplitude and speed to satisfy the conservation of energy. As well, an earthquake signal passing through a heterogeneous soil with significant velocity changes will undergo amplification or de-amplification, according to the velocity distribution in the soil. *Seismic velocity contrast* is responsible for amplification/deamplification factors up to 2, and is one of the basic phenomena to consider in seismic response analysis. The free surface can be considered a particular velocity interface that leads to total reflection of an incident wave-field. Constructive superposition of incident and reflected phases produces amplification factors of around two at the surface. The free surface is an essential condition for the development of surface waves.

At most sites, seismic velocities are increasing with depth and therefore an amplification of the earthquake signal is generally experienced at the surface. Such phenomenon is particularly evident in the case of unconsolidated quaternary deposits with soft (low-velocity) sediments on top of rigid (high-velocity) bedrock, but also rock sites with moderate velocity gradients and/or surface weathering can significantly affect the ground motion amplitude, and their response should be properly characterized. Unfortunately, the effect of the seismic response at rock sites is nowadays largely underestimated, which often leads to systematic errors in GMPEs and regional seismic hazard analysis. Assessing the effect of the local velocity variability is also particularly important when comparing ground motion between different sites.

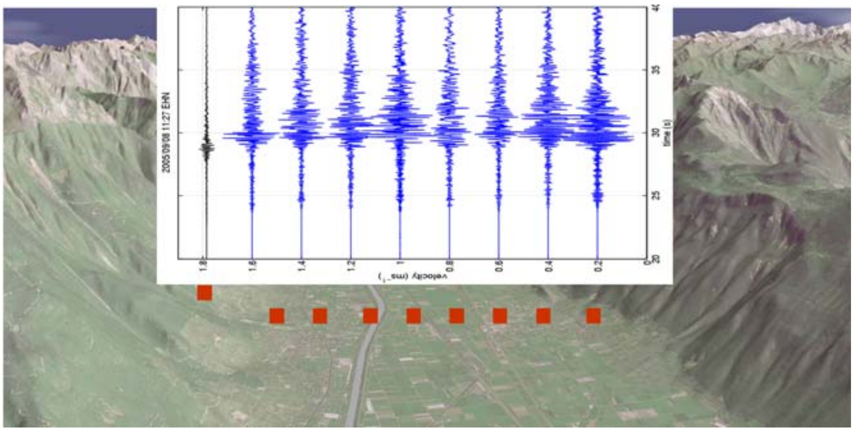


Figure 5. Recorded ground motion in the Rhone valley induced by the Vallorcine earthquake of September 8, 2005 (Mw=4.4). Only the EW-component is shown. In the basin, the ground motion is characterized by soil-amplification, local resonances and edge-generated surface waves.

3.1.2. Seismic resonance

Seismic impedance contrast is not the only origin of amplification of seismic waves. In sedimentary basins, it is also common to observe the phenomenon of “trapping” of the incoming waves, due to the multiple reflection and refraction of waves within the sediment layers and the free surface. In this way, the waves that reverberate in the structure may then lead to complex constructive or destructive interference patterns, depending on the geometrical characteristics of the basin and the analysed wavelength (and thus the frequency). Given the broad range of frequencies and wave types composing the earthquake spectrum, various wavelengths can interfere differently in the structure, providing amplification in some frequency bands and deamplification in some others. This phenomenon is termed resonance and can be described by a frequency-dependent amplification function, with maxima corresponding to the so-called resonance frequencies of the structure (f_N). If a unique large seismic impedance contrast is present, the largest amplification occurs at the *fundamental frequency of resonance* (f_0) (see red arrows in Figure 4), which is then one of the key-parameters to be mapped. Special techniques for the characterization of f_0 based on earthquake signals and ambient vibration recordings will be discussed in the next chapter.

3.2. Geometrical effects

3.2.1. Complex 2D/3D structures

Sedimentary basins with complex geometry and lateral heterogeneities can impose an interaction of waves with the 2D/3D structure, generating complex resonance patterns, focusing of wave energy, edge generated surface waves and large differential motion. In such a case, the effect on the ground motion can be strong, with amplification factors of ten and more, but well localized in delimited areas of the basin. Irregular interfaces between the sediments and bedrock can lead to focusing or defocussing of the propagating wave to the surface (the so-called lens effect), resulting in an additional contribution to amplification (or deamplification). Additionally, the interaction of the seismic waves with the border of sedimentary basins can lead to the development of edge-generated surface waves, which are responsible of large ground motion at the surface. In the deeply incised alpine valleys, where maximum bedrock depth is in the range of basin width, 2D and 3D resonances can easily develop (e.g. Roten et al., 2006; Ermert et al., 2014; Poggi et. al., 2015)

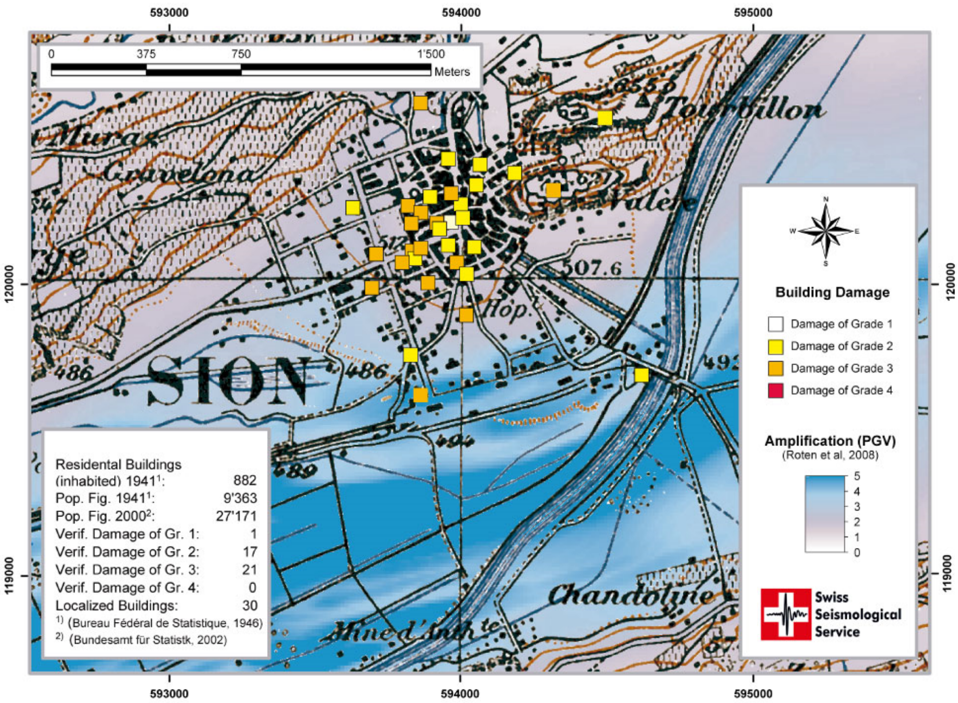


Figure 6. Comparison between observed damage in Sion caused by the January 25, 1946, Mw 5.8 earthquake and the amplification in peak ground velocity (PGV) from numerical modelling (from Fritsche & Fäh, 2009). Today the entire Rhone plain is densely populated.

Assessing the variability of f_0 over a larger area in the valley is a good indicator for the identification of 2D/3D resonances in sedimentary basins. In case of smooth variations of the geophysical properties and bedrock depth, so that the basin can be assumed to behave one-dimensional, f_0 tends to vary progressively and smoothly decreasing while approaching the valley edges. On the opposite, sedimentary basins with pronounced 2D/3D behaviour show a nearly constant value of the fundamental frequency across the whole area. Such evidence can be supported by results from more specialized techniques. Wave-field polarization and directional analysis can highlight the presence of preferential direction of the resonance in relation to specific geometrical features. Array analysis that can show that all points of the array move synchronously at the frequencies of resonance resulting - at least theoretically - in infinite phase velocity.

3.2.2. Surface topography

Effects of surface topography on ground motion are site-specific and existing instrumental or observational data are not complete enough to justify statistical analyses. For such reason the few empirical relationships or predictive models are today strongly questioned. Theoretical and numerical models of isolated geometrical effects predict a systematic amplification of seismic motion at ridge crests, and, more generally, over "convex" topographies (such as cliff borders), and a correlative de-amplification over "concave" parts of surface topography, such as foothills. The maximum amplification from such geometrical effects reaches maximum values around 2 in particular frequency bands (e.g. Assimaki et al., 2005). However, this is far too low from observed values, clearly indicating that amplification is mostly related to the surface velocity structure (e.g. Paolucci et al. 1999; Gallipoli et al. 2013): strong site-effects are mainly due to weathered or fractured rock, or colluvium covering the topographic feature (Burjanek et al., 2014a; 2014b, 2016). Irregular topography however contributes to the scattering of wave energy and the related modification of waveforms (Imperatori & Mai, 2015; Takemura et al., 2015).

3.3. Non-elastic behaviour

3.3.1. Anelastic attenuation

Anelastic (or intrinsic) attenuation is a property of the visco-elastic materials, which dissipate the energy of the wave travelling through them by the effect of friction of the constituting elements (minerals, sedimentary grains, dislocations, etc.). The energy lost in the process is then converted to heat and dissipated. Attenuation can visibly affect the ground motion observed at the surface, by its filtering effect on the high-frequency components of its spectrum, and is therefore an essential parameter to properly model amplification. Empirical ground-motion prediction equations model this high-frequency attenuation effect with a site-specific parameter called kappa (κ; Anderson et al., 1984) that however is difficult to be transformed into soil and rock attenuation properties. Kappa can directly be obtained by analysis of earthquake spectra and summarize the contribution of the whole soil structure to the ground motion attenuation. Despite its simplicity, kappa suffers the limitation of not being directly related to specific material properties, and thus it can hardly be predicted from geological or geotechnical information (e.g. Edwards et al., 2015; Ktenidou et al., 2015).

Different constitutive models exist to describe anelastic attenuation, but they fall at mostly into two major categories: frequency independent and frequency dependent models. The former is represented in seismology by the parameter Q (quality factor; Knopoff, 1964) and in engineering with the damping. It is of more simple formulation and calibration, but works preferentially at low attenuation levels, short propagation paths and narrow frequency bands. Conversely, at regional distances and in case of significant anelastic attenuation along the path, a frequency dependent model has to be used. Such frequency dependence is not necessarily due to the material properties only, but includes effects of the wave-field composition in the different frequency bands (body wave with different propagation paths, surface waves), and is therefore affected by geometrical effects and the influence of different layers in the earth crust. Further dependencies on rheological (viscosity, temperature, pressure) and geotechnical (e.g. porosity, saturation) properties have been studied and are available, but far from a practical application in site-response analysis due to the difficulty for calibration.

Attenuation can be directly estimated on soil samples though specialized laboratory analysis, but extrapolation to large-scale attenuation models is not straightforward and affected by large uncertainties. As major issue, results from laboratory tests and field data are often in disagreement, mostly because of the dissimilarity between controlled laboratory conditions and natural conditions (e.g. different pore fluid pressure, reshuffling of the material).

3.3.2. Scattering attenuation

Geological and geotechnical units are often approximated as homogenous material, but in reality, a certain level of heterogeneity is always present from the scale of the mineral/grains to the size of big rock inclusions. Wave-field propagation is affected by the presence of these irregularities, resulting in scattering of energy and loss of coherency. The phenomenon is frequency dependent, as longer wavelengths are less affected by the presence of small, random irregularities, while very short wavelengths (high frequencies) can totally be scattered. The macroscopic effect in the observed ground motion is twofold: a decrease of the signal amplitude along the path is associated to an increase of its duration due to the multiple interactions of the seismic waves with local heterogeneities. In contrast to anelastic attenuation, no energy is lost during scattering, which results in an apparent attenuation of the traveling waves.

A formal separation between scattering and intrinsic attenuation on observed ground motions is practically not easy, although some techniques based on theoretical models are available (e.g. Hudson, 1991, Hoshiba 1993, Pilz and Fäh, 2016). For simplicity, a unique attenuation model often describes the two phenomena, even if conceptually different. This simplification however leads to further mismatches between estimates from laboratory analysis and field geophysical techniques. A common parameter used to describe the characteristic scale of the heterogeneities in the medium is the so-called correlation length (CL), computed upon statistical models. Wavelengths comparable to the correlation length are normally the most affected by scattering.

3.3.3. Non-linear soil response

It has been often observed during strong earthquakes that, as the excitation amplitude increases, the dynamic behaviour of loose soils cannot be described accurately by classic linear elasticity. A typical non-linear soil response is characterized by a progressive increase in damping (attenuation) at high frequency, with an associated reduction of the shear modulus (and thus of the seismic velocity) due to an increase of pore pressure. A reduced shear modulus alone may imply an increased amplification. However, increased damping generally tends to dominate, resulting in reduced amplification factors, and even possible de-amplification at high strain levels. Accounting for the non-linearity in soil response is therefore relevant in hazard analysis, as it alters the level of expected shaking for large ground motions and can be accompanied by changes in soil resonance frequencies. On the other side, the parameters controlling the non-linear soil response are difficult to measure. Although numerous laboratory methods as well field tests aimed to retrieve the non-linear parameters of the material are available, there is a lack of a sufficient number of ground motion observations to produce a reliable statistic for the variety of different soil condition.

3.4. Soil parameters

To quantify the impact of local site conditions on the ground motion, a model of the local structure is necessary. If only linear soil response is considered, the most relevant parameters to characterize the soil behaviour are the seismic velocity of body waves (S- and P-wave seismic velocities), the density (ρ) and the attenuation quality factors (Qs and Qp). The way these parameters are geometrically distributed controls the amplification of ground-motion during an earthquake. Shear-wave velocity Vs, in particular, is the most important material property driving the amplification phenomena, because shear-waves and surface waves are generally the strongest phases in the earthquake ground motion. Shear-wave velocity profiles are thus needed for microzonation studies and for the interpretation of recorded earthquake ground motion.

Velocity information can reliably be obtained nowadays through a variety of geophysical investigation techniques, each one characterized by a certain resolution (details of the velocity profiles) and penetration depth (maximum resolved depth). On the contrary, mapping density and in particular quality factors over wide areas and large depths in most cases remain an open issue. Direct sampling and laboratory analysis are a useful mean, but the assessment is generally limited to the near surface or to boreholes that are expensive. There is a particular need for new cost-effective methods for reliable estimation of site-specific attenuation.

If non-linear soil response is the target, the variation in shear moduli and damping as a function of strain has to be additionally measured. These functions are preferentially calibrated on soil samples using direct laboratory analysis, but empirical relations are also available in literature, with the drawback of introducing large uncertainties in the estimation.

4. Earthquake-induced effects

Earthquakes can have a direct impact on the vulnerability of the natural environment though triggering so-called earthquake-induced or secondary effects. Among these effects are soil liquefaction and ground settlements, landslides and rock fall, tsunamis and seiches. All these effects were observed in past strong earthquakes in Switzerland (Schwarz-Zanetti et al., 2003, 2004; Fritsche et al, 2012). Definition of their hazard potential requires the use of specialized techniques and complex analytical and numerical models to simulate the behaviour of the geological structures under dynamic loading.

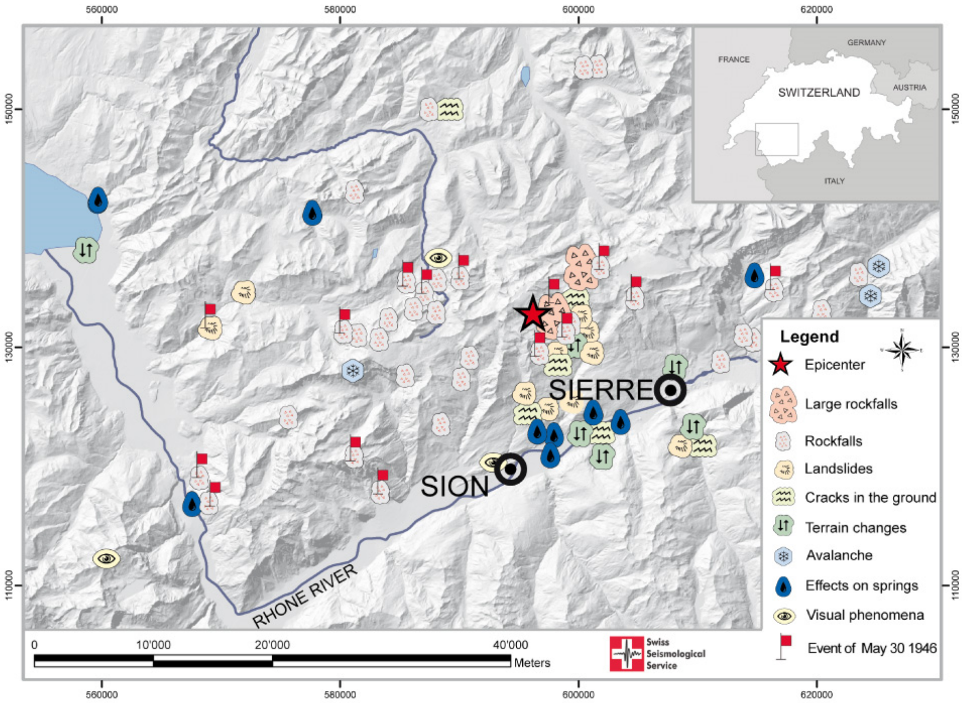


Figure 7. Earthquake induced effects caused by the January 25, 1946, Mw 5.8 earthquake (from Fritsche & Fäh al., 2009).

4.1. Liquefaction and cyclic mobility

Liquefaction is a process in which water-saturated sediments temporarily lose their strength and stiffness, and act as a fluid. Liquefaction takes place when loosely packed, waterlogged sediments at or near the ground surface loses their strength in response to strong ground shaking. This reduction of strength is due to the fact that during dynamic excitation, pore water pressure in the sediment tends to increase. The increase of pore water pressures causes the effective stress to decrease, which in turn affects the soil strength and stiffness. If the effective stress drops to zero, soil loses its shear strength completely, and it liquefies. Buildings and structures built on loose water-saturated sediments experiencing liquefaction can suffer irregular settlement, tilting and differential loss of support at the foundations, which in turn can induce heavy structural damages or even collapses.

Cyclic mobility is a special case of liquefaction where soil progressively loses its effective stress under application of cyclic loading (periodic contractive and dilative states) over a time span. This phenomenon causes progressive accumulation of deformation that can develop into ground spreading and failure, and affects the ground motion by development of large-amplitude high-frequency pulses in acceleration.

The principal conditions for liquefaction to be considered in microzonation are:

- The soil consists of a loose granular material (mostly sand and silt) of at least 1 meter thickness.
- High water table near the surface (<15m), resulting in saturated soil conditions.

The knowledge of the detailed soil conditions at a specific site is therefore essential in order to be able to predict the liquefaction susceptibility. Principal investigations for the assessment of the liquefaction potential with empirical methods are SPT (standard penetration tests), CPT (cone penetration tests), and laboratory tests on extracted soil samples. For predicting liquefaction with numerical models, the most relevant parameters to be estimated are the density, cohesion, grain size and plasticity index. Modelling the phenomenon requires also the definition of an appropriate constitutive model of the soil and its parameters (including water table level) and a number of adequate shacking scenarios (according to the de-aggregation of the hazard).

Geological and geomorphological criteria have also been used in literature to map liquefaction potential over large areas using lithological consideration (e.g. Wakamatsu, 1992). Although very appealing, this approach lead to considerable uncertainty and is therefore not suggested in microzonation.

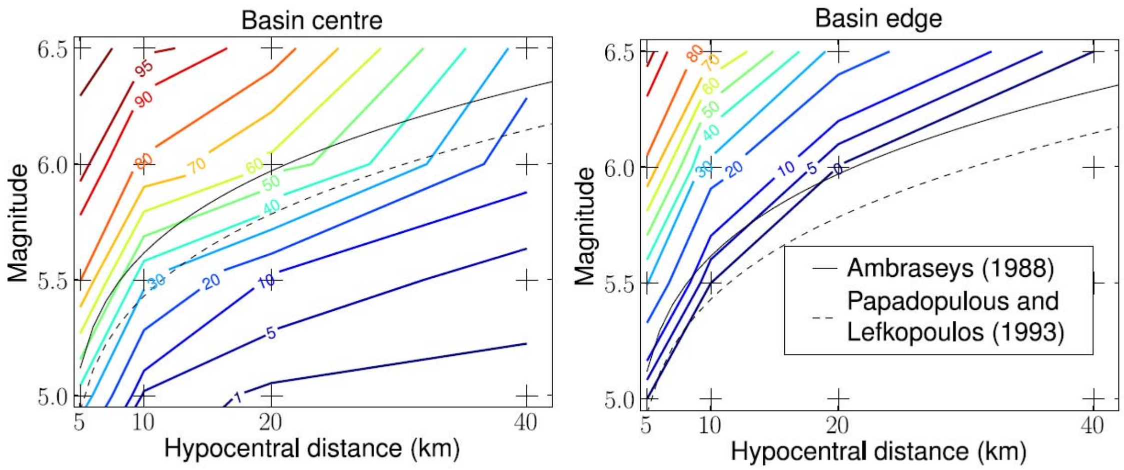


Figure 8. Liquefaction probabilities (in %) as a function of hypocentral distance and magnitude for the Rhone valley centre (left) and the basin edge (right) (from Roten et al., 2009). For comparison two empirical relations are shown.

4.2. Landslides and rock falls

Slope failures and rock fall during earthquakes have caused a large number of casualties and have been a major cause of damage to structures and facilities constructed on or near the slopes (Maranò et al. 2010). Such earthquake-triggered effects are very important in mountain areas such as the Swiss Alpes; widespread effects were observed during past earthquakes (Fritsche et al., 2012).

The loss of strength may have different causes, such as pore water pressure built-up during the dynamic excitation, or initiation of sliding in shear planes. Motions due to loss of strength caused by excess pore water pressures may continue after the excitation, due to the fact that it takes time for the excess pore water pressures to decrease again. The stability of a slope depends on its geometry and its soil or rock conditions, as well as on the hydrostatic conditions. Ground motion amplification plays an additional role.

The amount of rainfall before an earthquake may be of utmost importance. The knowledge of the soil conditions as well as the hydrologic situation is a prerequisite in order to be able to predict the behaviour of the slope. Models to describe landslide triggering are still very simplistic and often based the Newmark’s method (Newmark, 1965). Newmark's method cannot take account pore water pressure built-up during cyclic loading. Numerical methods are used today that include elasto-plastic and simulate pre-yield elasticity. Such modelling requires detailed field investigations to define the soil and rock parameters, and dynamic response of the instability (e.g. Burjánek et al., 2010, 2012; Moore et al., 2011)

4.3. Tsunamis and seiches

Tidal waves or tsunamis are phenomena originated by instantaneous differential displacement and subsequent gravity restore of large masses of water in water basins of any size, ranging from oceans and lakes to small water reservoirs. High-speed and long-wavelength gravity waves are generated in this way, whose amplitude can dramatically increase while approaching the coastline. This phenomenon can be directly or indirectly induced by the action of an earthquake, and represents an important factor contributing to the increase in local seismic hazard in coastal areas.

Tsunami waves are generally generated by the outcropping faults with vertical displacements at the seafloor. Secondary earthquake-induced phenomena, such as marine, lake or coastline landslides and rock falls, can also generate sufficient water displacement to trigger the development of tsunami waves. Such slides were the main reasons for the generation of known tsunamis in Swiss lakes (e.g. Strasser et al., 2013; Kremer et al., 2015; 2016). Still, scientific work is needed to include tsunami hazard assessment in microzonation studies.

Moreover, water basins with appropriate geometrical features (length and depth) can lead to phenomena of wave-trapping and interference similar to the development of seismic resonances in sedimentary basins. This phenomenon produces long-periods (and wavelength) standing-waves called seiches, with eigenperiods that depend on the water volume. Oscillations are lasting for many hours and even several days before energy is being dissipated.

Chapter III

Site characterization techniques

1. Introduction

In order to compute the seismic response of a site with some confidence, an adequate level of knowledge of the structural characteristics of the area is necessary. As a matter of fact, the final ground motion is highly controlled by the geophysical properties of the first tens to hundreds of meters of the soil and rock. The available level of knowledge, however, is often limited because of the considerable investments required for measurements. Cost efficient methods to estimate soil parameters - mainly shear-wave velocity as a function of depth - are therefore the major issue in local seismic response evaluation and site-specific seismic hazard assessment. In the following, the most common techniques for site characterization are presented, subdivided in categories by survey and observation type.

2. Invasive methods: borehole and geotechnical sampling

Invasive investigation techniques are based on the direct sampling of the soil material, in order to assess in-situ or with laboratory analysis the main geotechnical (e.g. permeability, porosity, compressibility, shear resistance) and geophysical properties (S- and P-wave seismic velocities, density and quality factors) of the ground. In-situ sampling has the clear advantage of being a direct (theoretically unbiased) observation of the material properties. The major drawback, however, is that the result is punctual and therefore not always representative of the average site conditions around the measuring location. In case of laboratory measurements, moreover, an important issue is related to the sampling and transport process, which can alter the material properties before the analysis, introducing considerable bias in the results.

Direct investigations are generally limited to the first few upper meters of the soil profile (e.g. trenching, excavation) but in few cases exceptionally high depths can be reached by drilling boreholes. The main limitation in this case stays in the relatively high cost of implementation, as the drilling price tends to increase non-linearly with the depth. For such reason the use of deep boreholes in local microzonation studies is usually limited to few selected sites of significance if at all, mostly in support and as constraint of non-invasive geophysical analyses.

In case of boreholes, seismic velocities can be estimated using travel-time analysis of active-source recordings. Different variants of such approach are available, depending on configuration and number of sources and receivers. Down-hole and up-hole methods are (nearly-) vertical seismic profiling techniques that make use of a single borehole. Down-hole approach is by far the most common, consisting in one or more sensors placed along the well, while the source remains at the surface. With this configuration special shear-wave sources (of difficult in-hole implementation) can be used, such as shear-beam or seismic vibrators; this is difficult with the reversed configuration used in the up-hole method. Quality of the results decreases with depth, due to propagation of errors in the conversion between travel time (interval) and layer velocities, with a maximum resolution of several tens of meters (~50-100m), depending on source type and soil characteristics. Cross-hole analysis requires two wells (one for the source(s) and one for the receiver(s)), or more. Such techniques is of more difficult implementation (e.g. borehole deviation survey might be needed, use of in-hole sources) but it has the advantage of overcoming most of the limitations of the down-hole approach. Maximum penetration depth is just limited by the length of the boreholes, and 2D tomographic imaging of the velocity structure between wells is possible.

For shallow geotechnical investigations SPT (*standard penetration test*) and the more sophisticated CPT (*cone penetration test*) are used. In both cases, a probe is inserted vertically into the ground. The penetration resistance (function of the soil friction) is then measured and interpreted in terms of geotechnical properties. Specifically for CPT measurements, good correlation exists with geophysical parameters, which makes this method suitable to estimate seismic velocities (e.g. Imai & Tonouchi, 1982) or the likelihood of co-seismic effects, like the *soil liquefaction*. SPT and CPT methods are nowadays not particularly expensive, they can be applied over wide areas. However, they have conversely a limited penetration depth, generally not more than 10-20m depending mostly on the presence of gravels or larger boulders at depth.

3. Non-invasive methods: geophysical techniques

Non-invasive investigation techniques (or geophysical methods) use the properties of the physical fields (electric, magnetic, gravity, seismic) to infer information on the soil structure. Usually the observation point is on the free surface. Geophysical methods can be grouped in two big categories: static-field and wave-field methods. In the former category fall those techniques that rely on the time-independent observation of the target field properties, like:

- Electrical methods (resistivity, self-potential)
- Magnetic methods (magnetic susceptibility)
- Gravimetric methods

In the second category are the techniques based on the recording of signals that propagate from a source (known or unknown) to the observation points. This is the case of:

- Electromagnetic methods (radar)
- Seismic methods (active and passive)

The most suitable approach for seismic characterization is clearly the seismic method, because it provides a direct estimation of those geophysical properties (mostly seismic velocities) that are necessary for the evaluation of the site response. Nevertheless, other methods are sometimes useful in combination with seismic acquisition, e.g. the gravity approach to constrain the geometrical characteristics of buried sedimentary structures (e.g. mapping the bedrock shape), or the electrical methods to constrain the depth of the groundwater table, which is an important parameter to evaluate liquefaction potential of a site. In the following we focus specifically on the description of seismic techniques (including active and passive) being so far the most widely used techniques in microzonation studies.

4. Overview on seismic methods

Depending on the type of source that generates the seismic wave-field, the seismic methods can be further classified in active and passive seismic techniques (Table 1). In the first case, an artificial source is used to excite the propagation of a signal, while in the second case the natural ambient vibration wave-field (also called “seismic noise”) is directly recorded and analysed. One of the major differences between the two categories of techniques stays in the energy content of the wave-field. Active sources have a high-frequency content (usually > 10Hz), which determine a relatively scarce depth resolution; high frequencies are rapidly attenuated along the propagation path. Artificial generation of low-frequency signals is however impractical (requires special devices or the use of explosive not applicable in urban environment) and particularly expensive, which limits its application only to special studies (e.g. oil exploration). Conversely, the ambient vibration wave-field has a considerable low-frequency content (<10Hz), which makes it suitable for the characterization of deep structures. Unfortunately, processing of ambient vibrations is more complex than in active methods and requires a level of expertise not always available among practitioners.

4.1. Active seismic techniques

Active seismic techniques are divided in two major categories: the travel-time and surface wave methods. The first approach is essentially based on the measurement of the time needed for the seismic waves to propagate from an artificial source to one or more receivers. The receivers can be located at the surface (as in the case of reflection and refraction seismic, including tomography) or as already discussed in boreholes (down-hole and cross-hole seismic). The measured travel-times are later converted to a velocity model. With the exception of down-hole borehole seismic, where just a vertical velocity log is provided, the majority of active seismic methods can provide 2D (and sometimes 3D) representations of the soil structure. This is very useful in case of complex structures with lateral velocity variations. From a 2D seismic profile (or section) it is always possible to derive equivalent “average” one-dimensional velocity models.

Table 1. Summary table of common seismic techniques in site characterization.

<table>
<tr><th colspan="4">Technique</th><th>Soil Model</th><th>Depth Resolution</th><th>Cost</th></tr>
<tr><td rowspan="6">Active source</td><td rowspan="4">Travel time</td><td rowspan="2">Surface</td><td>Refraction</td><td>2D/3D</td><td>Low</td><td>Medium</td></tr>
<tr><td>Reflection</td><td>2D/3D</td><td>Low/Medium</td><td>Medium/High</td></tr>
<tr><td rowspan="2">Borehole</td><td>Down-/up-hole</td><td>1D</td><td>Low/Medium</td><td>Medium/High</td></tr>
<tr><td>Cross-hole</td><td>2D</td><td>Low/Medium</td><td>Medium/High</td></tr>
<tr><td rowspan="2">Surface waves</td><td colspan="2">SASW</td><td>1D</td><td>Low</td><td>Medium</td></tr>
<tr><td colspan="2">MASW</td><td>1D</td><td>Low</td><td>Medium</td></tr>
<tr><td rowspan="11">Ambient Vibration</td><td rowspan="5">Single Station</td><td rowspan="4">H/V</td><td>Standard</td><td>--</td><td>--</td><td>Low</td></tr>
<tr><td>Directional</td><td>--</td><td>--</td><td>Low</td></tr>
<tr><td>Wavelet</td><td>--</td><td>--</td><td>Low</td></tr>
<tr><td>RayDec</td><td>--</td><td>--</td><td>Low</td></tr>
<tr><td colspan="2">Polarization Analysis</td><td>--</td><td>--</td><td>Low</td></tr>
<tr><td rowspan="6">Array</td><td rowspan="3">F-K</td><td>Beamforming</td><td>1D</td><td>High</td><td>Medium</td></tr>
<tr><td>High-resolution</td><td>1D</td><td>High</td><td>Medium</td></tr>
<tr><td>MUSIQUE</td><td>1D</td><td>High</td><td>Medium</td></tr>
<tr><td colspan="2">SPAC/MSPAC</td><td>1D</td><td>High</td><td>Medium</td></tr>
<tr><td colspan="2">WaveDec</td><td>1D</td><td>High</td><td>Medium</td></tr>
<tr><td colspan="2">Cross-correlation</td><td>2D/3D</td><td>High</td><td>Medium/High</td></tr>
</table>

In the surface wave methods (SASW, MASW and ReMi; Park et al. 1999; Louie, 2001), conversely, receivers are only located at the surface, as the surface waves propagate parallel to this interface with amplitude decaying with increasing depth. Surface waves have a dispersive behaviour; this means they propagate with different velocity at the different frequencies, when the soil is vertically heterogeneous. By analysing the relative phase delays between pairs of receivers it is then possible to map their velocity dispersion function versus frequency, which can later be inverted into one-dimensional velocity models that best explain observations. The inversion process is nevertheless highly non-linear and non-unique, and convergence to a meaningful solution might require the use of additional *a priori* information from local geological considerations and/or other surveys. It is not uncommon to perform a combined inversion with other geophysical datasets, such as results from ambient vibration methods (see next section).

In all surface wave methods (also including passive methods), maximum resolved depth depends, together with energy content of the source, on the geometry of the sensor array and the velocity distribution of the structure. As a general rule, large sensor deployments and low seismic velocities lead to resolution on longer wavelengths and therefore to higher investigated depths. Different empirical relations have been proposed in literature to define resolution limits, but they all work in first approximation. More properly, a theoretical array response analysis should be performed, followed by a sensitivity study on the inverted velocity structure; this in order to define the depth range where seismic velocities are poorly constrained by available surface wave information.

Travel-time methods (reflection and refraction seismic) have generally the higher resolution on velocity interfaces than surface wave methods, which conversely can provide a better representation of the velocity structure with lower implementation costs. As common drawback of all the active seismic techniques, however, they all have a rather scarce penetration depth when conventional artificial sources are employed (e.g. hammer, mini-gun). As already introduced, more energetic sources can be nevertheless used, like explosives and vibrator tracks (vibroseis), but these require a considerable investment and special permissions. In urban environment, moreover, such source type cannot be used at all. Consequently, maximum resolved depth with active methods in urban environment is generally limited to the first 20-40m of the soil structure. As additional issue, soil parameters obtained from the interpretation of just high frequencies data might not be representative of the soil response in the frequency band of interest in seismic hazard assessment (roughly 0.5-10Hz). This issue is presently investigated by the scientific research community.

4.2. Passive seismic techniques

Nowadays, an increasing interest is in the development of geophysical methods based on passive acquisition of ambient vibrations and micro-tremors (for a literature review see Bonnefoy-Claudet et al. 2006). The broadband nature of ambient vibrations leads to a high-resolving power for passive seismic techniques: from the upper meters of soil down to several hundred meters. As additional advantage, these methods can be applied in urban areas where in general it is not possible to carry out active measurements due to the lack of space for the experimental linear setup or the impossibility to use explosion sources. The use of ambient noise recordings is indeed very appealing: it is non-invasive and well suited for dense urban environments, the required equipment (sensitive seismometers and data acquisition systems) is available at affordable cost, and the processing techniques have been the topic of many developments in recent years.

Ambient vibration methods make use of the properties of surface waves, and allow the determination of important properties of the soil, like the fundamental frequency of resonance and the shear-wave velocity profiles from the inversions of surface-waves dispersion curves. Their depth resolution is significantly larger than with active methods due to the low-frequency content of the ambient vibration wave-field (Horike, 1985). Their use is therefore challenging in modern microzonation, particularly in combination with other methodologies such as active seismic methods to better constrain the velocity structure at the surface.

The ambient vibration techniques can be divided in two major categories: the single station and the array methods. In the category of the single station methods, the more widely used is the horizontal-to-vertical (H/V) Fourier spectral ratio (Nakamura, 1989), which allows a fast and sufficiently precise estimation of the SH-wave fundamental frequency of resonance (f_0) of the site

(Lachet & Bard, 1994; Bonnefoy-Claudet et al., 2006b). This information is valuable because it provides a major constraint for the definition of depth and seismic impedance contrast of the geophysical bedrock, as well as qualitative information about the amplification as a function of frequency (Figure 9). Assuming a sediment layer over bedrock, the expected amplification is large at the fundamental frequency of resonance and significant for the frequency range above, but absent in the frequency range below $f_0/2$.

Directional H/V ratios can be used to identify 2D resonances in Alpine valleys. For a 1D structure, H/V ratios are useful to provide information about the Rayleigh wave ellipticity function (e.g. Lermo & Chavez-Garcia, 1994; Fäh et al., 2001), which is a principal factor controlling the shape of the H/V's curve and can be used to constrain the major velocity contrasts of the soil profile in inversion procedures of dispersion curves (combined inversion, e.g. Poggi et al., 2012).

The relative low-cost of implementation and the short time needed for the analysis give the possibility to use this approach over wide areas, with the aim of mapping the variability of the subsoil structure. This is of major concern, as it can confirm the fulfilment of assumptions like the one-dimensionality of the site, necessary for a proper selection of the modelling technique to be used in the seismic-response analysis. The H/V technique has been investigated during the European research project SESAME and the user is recommended to study the specific recommendations provided on the project WEB-page (http://sesame-fp5.obs.ujf-grenoble.fr).

Several authors have used the amplitude of H/V spectral ratios to predict amplifications. This procedure however is not recommended, because the amplitude of the H/V spectral ratio is not directly related to the amplification of waves during earthquakes, and may depend on the nature of the ambient vibrations wave-field. However, the amplitude of the H/V spectral ratio at the fundamental frequency f_0 is an indicator for the S-wave velocity contrast between bedrock and sediments (Figure 9), and provides some information on the severity of possible resonance effects during earthquakes. The higher the amplitudes the larger is the velocity contrast. However, the amplitude of the H/V ratio depends not only on the velocity contrast but also on the source-depth distribution and source-distance distribution. Amplitudes of the H/V ratio therefore provide only a qualitative indicator of possible resonance effects.

Finally, H/V spectral ratios can be considered a fingerprint of the local structure. By comparing measurements within an area, similar H/V curves are observed for similar local structures and therefore show similar amplification effects during earthquakes. Such comparison can therefore be used in combination with geological and geotechnical information to define zones in microzonation studies, as in the case of Basel (Figure 2).

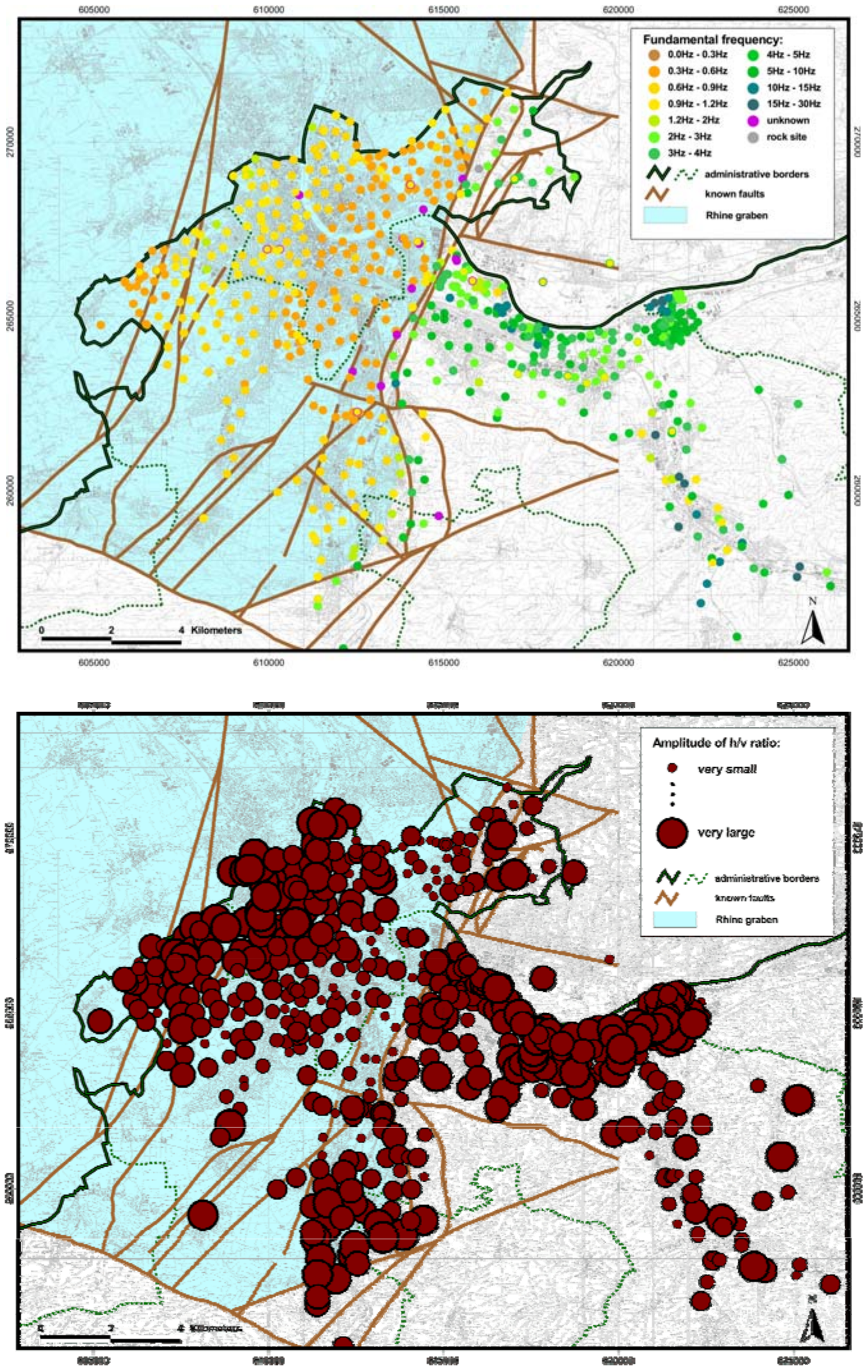


Figure 9. Top: Measurements of the fundamental frequency f_0 of resonance in the Basel, area (Fäh & Huggenberger, 2006). The main faults are drawn with brown lines. The Rhine Graben structure (coloured in light blue) is visible through the low values of f_0. The Tabular Jura structure is covered with thin layers of soft sediments and the observed values of f0 are mainly above 1.5Hz. Bottom: Amplitude of the H/V spectral ratio which indicates the velocity contrast between sediments and bedrock.

Ambient vibration array methods were established by Horike (1985) after the pioneering work done by Aki (1957). Array analysis is a multi-station spectral technique that allows retrieving the directional and the dispersion characteristics of the surface waves. The surface-wave (Rayleigh and Love) dispersion curves are then inverted to obtain an estimation of the seismic velocity profile of the site (mainly S-wave velocity as a function of depth, and to a lesser extend P-wave velocity; e.g. Tokimatsu 1997). Array techniques can be summarized in two groups: the frequency-wavenumber methods (F-K; Lacoss et al., 1969; Capon, 1969; Schmidt, 1986) and the spatial auto-correlation techniques (SPAC; Aki, 1957; Asten, 2006). In the first category fall several approaches like beam-forming (standard and high-resolution), MUSIC and different derived algorithms. F-K can be regarded as a modification of the 3D-Fourier transform of the seismic wave-field (two spatial and one temporal dimension). These processing algorithms are similar to those actually used in active surface wave analysis. The SPAC techniques, conversely, are based on the analysis of the correlation functions between noise recordings at different locations. Recent developments in array analysis allow using all the three components of motion to extract simultaneously the Rayleigh (vertical and radial) and Love (transversal) dispersion curves as well as other properties such as the Rayleigh wave ellipticity (Poggi & Fäh, 2010; Maranò et al., 2012).

a) STIEG (Active) **b)** HAMIK (Passive)

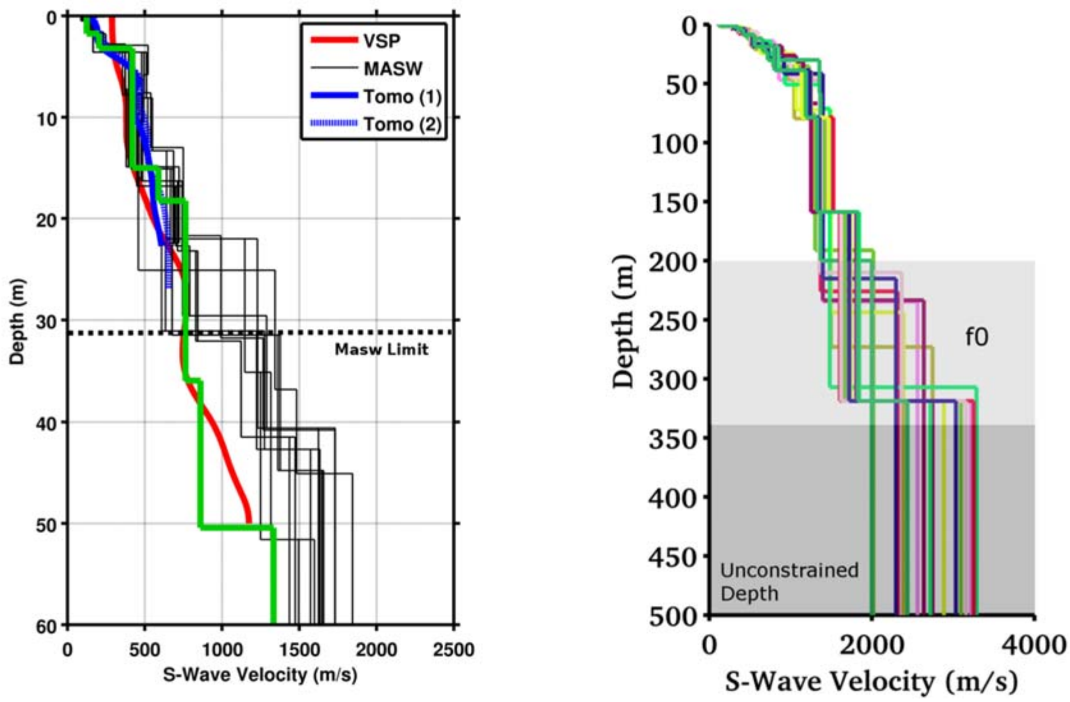


Figure 10. Example of shear-wave velocity profiles (Vs) from the SED site characterization database (Poggi et al., 2016). On the left, the results from active seismic processing (station STIEG) and on the right results from array analysis of ambient vibrations (station HAMIK). For each site, a set a plausible velocity models is provided, to represent the uncertainty of the estimation.

The stochastic nature of the ambient vibration wave field imposes a statistic approach to the analysis, which is made through the windowing process. In all noise-based techniques, the signal is portioned in short time windows, whose duration is proportioned to the lower most frequency of interest of the processing (usually calculated as few cycles). For each window a separate processing is performed (e.g. H/V, F-K) and the results from all windows analysed statistically by computing the expected value. To produce reliable and stable results, tens to hundreds of windows are usually needed, depending on many factors, such as the coherency of the surface wave signal and the average noise level. For these reasons, the length of a single record may vary between not less than 30 min (as in the case of H/V spectral ratios) to few hours (for very large arrays). The presence of disturbing factors such as short high-energy transients induced by human activity might impose to further extend the duration of the recording.

5. Criteria for site characterization

Decision on the type and number of techniques used for the characterization of an area subject to microzonation study is not straightforward. Adopted techniques may vary in relation to the extension of the area, the diversity of the geological conditions and the available budget. It is nevertheless possible to provide a set of recommendations about some important steps that are necessary for the definition of the geophysical model.

5.1. Preliminary analyses: geology and H/V survey

Planning of a site characterization campaign should be done only after having obtained all the geological, geotechnical and geophysical information available for the area in the form of maps, reports from previous studies or publications. This is an important step to get an overview of the local geological conditions, including (but not limited to) the extension of the quaternary deposits, locations of outcropping bedrock, type of expected lithology and sediments. This preliminary model, although not accurate, is particularly helpful to limit the extension of the survey area and to isolate locations of particular interest (qualitative microzonation), such as the basin edges or zones with possible high amplification or liquefaction potential. An example of such qualitative microzonation is given in Appendix B.

Before assessing the material properties of the study area, it is useful to perform a preliminary evaluation of the variability of the geophysical conditions. This can be done through the use of H/V spectral ratios of ambient vibration recordings. Being such technique of relatively cheap and quick implementation, wide areas can be mapped by rapidly performing a large number of single-station measurements. It is not possible to provide a direct recommendation on the number of required H/V measurement points, but it should be kept in mind that the distance between measurements should be sufficient to capture sharp variations in geology. As a general rule, a minimum distance of 100m to several hundred meters between points is sufficient, depending on the geological structure, with possibility to have a denser sampling to capture sharp lateral variations or special underground structures (e.g. buried faults, areas with infill).

As previously introduced, using H/V ratios gives the possibility to map the variation of the fundamental frequency of resonance (f_0) across the area, which is related to the depth of the main velocity interface (usually the bedrock, but in some cases also a sediment-sediment interface with different degree of consolidation or either rock-rock interfaces). A uniform value of f_0 over the whole study area where lateral variations of the geological structure have to be expected (e.g. Alpine valleys) is a good indicator of possible 2D/3D resonance behaviour. Such hypothesis must be nevertheless verified by subsequent analysis of the basin morphology or by other specialized techniques, such as site-to-reference spectral ratios and/or wave-field polarization analysis. Nonetheless, not only the value of the fundamental frequency from H/V is a useful indicator to highlight variations in the local geology, but also the amplitude of the peak can be considered. Although this value cannot be used to estimate directly the level of amplification at the site, it is useful to identify variations in the seismic impedance at depth. Similarly, the analysis of the relative variation in amplitude for other frequency bands can be advantageous as it might reveal (relative) changes in the soil structure across the area. The high frequency part of H/V ratios is strongly affected by the heterogeneities of the first meters of the soil; presence of reshuffled surface material or strong weathering/alteration can be identified in such a way.

In combination with H/V ratios, it is also possible to use gravity data to get a first-order representation of bedrock geometry variations in sedimentary basins. In practice, soft sediments have a lower density with respect to the underlying bedrock, which locally produce negative anomalies with respect to the theoretical gravity model computed on reference ellipsoid (e.g. WGS84) and corrected for elevation and local topography. Stronger anomalies therefore can indicate the presence of deep structures.

5.2. Geophysical model definition

The preliminary model obtained from geological considerations and H/V measurements has to be used to plan the subsequent geophysical investigations in the area, necessary to build up the seismic velocity model.

If possible, array measurements of ambient vibration should be considered first. Array methods work preferentially on soft sediment sites, where the low velocity of the materials and the large impedance contrast with the underlying bedrock are favourable for trapping of wave-field energy and for generation of surface waves. Recent studies nonetheless demonstrated the possibility to successfully apply these techniques also to stiff soil and hard rock sites, provided that in this last case the velocity increases progressively with depth (gradient-like function) due to presence of weathering and/or fracturing close to surface.

Since there is a direct correspondence between penetration depth and array size, the extension of the resolved velocity profile is generally limited by these factors:

- *Available space for measurement*; this is particularly critical in urban environments, where buildings might represent an obstacle to the array deployment, which often results in irregular geometries of limited extension.
- *One-dimensionality*; all f-k processing techniques assume one-dimensionality of the soil structure over the measuring area and therefore their use should be applied with caution in locations with evident 2D/3D characteristics (as for example close to basin edges) or in case of considerable lateral variability of the soil structure. Fortunately, such conditions can be mostly recognized by preliminary H/V analysis.
- *Irregular topography*; array methods work fine on gentle slopes, but very irregular topography should be avoided, as they also break the required 1D assumption. Topography is generally not an issue in urban environments, which are preferentially settled in flat alluvial valleys, but can be critical in Alpine regions with dipping slopes and variably outcropping rock.

Given a specific array size, maximum resolution depth is also controlled by the average velocity of the investigated site. Low-velocity sediments result in shorter wavelengths at given frequency, and may require arrays of considerable extension to resolve deep sediments. This problem is not present for stiff-soil and rock sites that have good resolution at high frequencies.

As in the case of H/V ratios, it is not possible to give clear recommendations on the number of arrays to be performed. This strongly depends on the characteristics of the study area. As a general rules, one or more large arrays (> 100-200m in diameter) should be performed at places where the basin is assumed to have maximum bedrock depth (usually along the valley axis), plus several “satellite” arrays of smaller diameter deployed in the surrounding regions with the goal to map the progressive variability of the velocity structure across the area. A preferential location for investigation is also at the places of temporary and permanent seismic stations, whose underlying soil should be properly characterized before use of the recordings. For some permanent seismic stations such measurements already exist (see http://stations.seismo.ethz.ch).

In all cases where array processing would fail or at sites of special characteristics, additional geophysical measurements are to be planned. For instance, two-dimensional seismic refraction profiles are generally suggested at places with shallow or outcropping bedrock close to basin edges. An accurate characterization of these sites is essential for the proper implementation of the geophysical model in light of subsequent 2D/3D numerical simulation. Computed ground motion has demonstrated to be sensitive to model boundaries and an improper characterization of the sediment-rock contact can lead to unrealistic ground motion estimates. Active surface-wave analysis is also recommended in conjunction with passive array analysis to provide high frequency constraints for the inversion, possibly including higher modes of surface waves. This is useful to improve resolution on the shallower portion of the velocity profile, which might not be fully resolved with ambient vibration techniques.

The use of other more specialized and demanding seismic techniques, such as ambient vibration tomography or large-scale reflection seismic studies, should be evaluated case by case. Implementation of such techniques may require considerable resources and level of expertise, often only available in the research and academic environments and in large geophysical exploration companies.

5.3. Invasive investigations and special sites

Invasive (direct) investigations and geotechnical techniques should be planned and performed after a sufficient knowledge of the area has been acquired. Due to the high cost of implementation, their use should be limited in number to sites to define particular soil characteristics. This is the case for soils expected to induce high amplification or behave non-linearly, with high liquefaction potential or in unstable conditions.

Explorative boreholes can be performed in areas where it is difficult or impossible to apply non-invasive techniques. It is also suggested to design them at locations where prior sufficient confidence on the model structure has already been obtained from previous analyses. In this way, log data can be used for the validation and eventually the recalibration of the geophysical model.

Cone Penetration Test (CPT) represents a reliable way to characterize the behaviour of saturated, sandy and silty soils susceptible to liquefaction. CPT measurements are minimally invasive, relatively cheap and characterized by high resolution (e.g. one measurement every 1 cm). They can be combined with the measurements of other parameters of interest such as S-wave and P-wave velocities. Due to the nature of the measurement, they can be applied only in loose or partially consolidated sediments down to 30-35 meters. The presence of gravels and boulders limits the possibility of application.

Chapter IV

Ground motion modelling

1. Introduction

Once a subsurface structure is well established with different field measurements, we can apply numerical methods to:

a) simulate ground motions for different earthquake scenarios,
b) estimate the expected amplification effect from the local soil conditions.

Depending on the level of knowledge of the model, numerical methods of different complexity can be applied. We distinguish different levels of complexity in the methods related to geometry (1D, 2D, 3D and topography), soil behaviour (linear elastic, linear anelastic, equivalent linear, plastic, non-linear), ground motion excitation (vertically incident plane wave, point source, finite source including path effects). More sophisticated numerical methods require generally many input parameters, which are often difficult to calibrate using standard investigation techniques. The use of complex modelling methods is therefore advisable only when all input parameters can be defined within a reasonable uncertainty.

The main frequency band of interest in engineering seismology corresponds to about 0.5-10Hz, the range of the resonance frequencies of buildings and constructions. However, modelling of large complex 3D structures with very-low velocity sediments can have high computational costs especially for higher frequencies (> 1Hz). The situation gets even more complicated if source and propagation path are included in the model or if special soil behaviour have to be accounted for. In these cases, it is preferable to use simplified approaches. Careful consideration of epistemic uncertainties can provide an approximate solution to the problem.

Highly recommended is the use of hybrid techniques (e.g. Liu et al., 2006; Seyhan et al., 2013) that can be used when the low and high frequency bands are simulated separately using deterministic numerical modelling and stochastic modelling methods respectively. Deterministic models can simulate special effects of the source and propagation path that the structural model is able to resolve; stochastic simulations at high frequency can take into account the source, path and site effects using a ground-motion prediction equation for reference rock condition and an amplification function to simulate the local soil response.

For the final interpretation, numerical results in terms of amplification obtained from different models and scenarios are collected, validated and analysed to map the variability of the ground motion at particular sites and over the study area. Aleatory and epistemic uncertainties have to be defined and documented. Simplified approaches require a careful assessment of the epistemic uncertainties that also include all effects that cannot be modelled with simplified approach. This *per se* is a difficult task and requires expert knowledge.

2. Model parameterization

The complexity of the input model for the simulation depends on many factors, including the available data, the numerical technique used for the soil-response analysis, and the level of confidence required for the final output. If the structure is horizontally layered (limited lateral variations of the site characteristics are expected over the area), one-dimensional models (1D profiles) can describe the site-response sufficiently. In such 1D models, material properties change only along the vertical direction. 1D profiles can be continuous as in the case of smooth gradients and described by a functional equation (e.g. exponential, power low) or can be discrete and described by a sequence of homogenous layers. This last representation is more convenient in numerical studies because of its intrinsic discrete nature, but as drawback, it might suffer approximation errors when used to describe smooth property variations using only few layers. In few cases, travel time (or harmonic) averaging is used to simplify complex structures. It should be nevertheless noted that most of the invasive and non-invasive field-investigation techniques provide natively discrete 1D profiles of the different geotechnical and/or geophysical parameters.

Complex 2D/3D models can be obtained directly from measurements, as in the case of seismic tomographic methods, or indirectly by interpolation of a sufficient number of punctual investigations together with the geological/geotechnical information. In the second case, particular care should be paid to the reliability of the interpreted results, as the input information might have been obtained applying techniques requiring the local 1D assumption even if performed in a more complex 2D/3D structure. This mismatch of the assumptions may lead to a certain inaccuracy that should be properly accounted for when defining the epistemic model uncertainties. 2D/3D models should also include a proper representation of the local topography at a scale proportional to the resolution of the model parameters. Digital elevation models (DEM) are nowadays available at very high resolution (25m and lower), which make them suitable for the use in microzonation studies.

3. Modelling seismic amplification

Amplification of the ground motions can be modelled analytically or numerically. Analytical solutions are accurate, but limited to simple structures (simple geometries). Whereas, numerical solutions can be applied almost to any model of arbitrary complexity (at the expenses of increased computation time), but as a drawback, they require verification of the stability and accuracy of the result as well as expert knowledge for their application. Numerical methods are usually first verified with analytical solutions for simplified models.

3.1. Analytical solutions

In case of the horizontally layered 1D structures, resonance patterns are predictable using simplified approaches. In particular, SH-wave transfer function is calculated as a spectral ratio between ground motion at the surface with respect to the bedrock at depth considering vertical incident plane wave (however, any arbitrary angle of incidence is possible as well). The SH-transfer function is the amplification function of the site assuming a reference. To make the estimation comparable between different locations, the local transfer function has to be normalized by the transfer function of a common regional rock reference profile (Edwards et al. 2013) at the free surface. This is essential for site response analysis in microzonation.

The velocity structure is approximated by a linear system under impulse excitation (Kennet & Kerry, 1979). The transfer function method is very simple and provides satisfactory results in many cases, but careful attention should be paid to the proper discretization of the model into finite layers as mentioned above; using too few layers and sharp interfaces to model smooth velocity gradients might lead to unrealistic amplification and artefacts in the form of high-amplitude resonance peaks. A convenient way to overcome this problem is to perform the computation of the amplification function for a certain number of equivalent velocity models (whereas all profiles are explaining the measurement data) and subsequent averaging.

Alternatively, approximate analytical solutions such as the quarter-wavelength (QWL) averaging method can be used for structures with smooth velocity gradients. The method relies on the fact that, if a layer thickness is much smaller than the specific wavelength of interests (at a given frequency), the seismic impedance contrast to neighbouring layers will not be resolved by the wave-field. This is the case for thin layers close to the surface (e.g. a weathering region), which does not produce significant amplification at low frequencies (long wavelengths) but visibly affects the high frequency part of the spectrum. A wavelength-dependent description of amplification can then be produced by travel-time averaging of the soil properties; averaging length varies as function of wavelength and ultimately of the analysed frequency:

$$Vs^{QWL}(f) = z\left[\int_0^{\frac{\lambda(Vs,f)}{4}} \frac{1}{Vs(z)}\,dz\right]^{-1}$$

Such averaging produces an equivalent presentation of the input velocity profile (the quarter-wavelength representation), for which corresponding amplifications can then be computed in relation to the maximum seismic impedance at depth. Again these amplifications need normalization to the reference rock profile. The quarter-wavelength approximation produces amplification levels lower to those obtained from SH transfer function modelling, as it cannot model resonances. Therefore, the QWL method can just be considered as a lower bound of the potential amplifications.

3.2. Numerical solutions

When the structure has complex 2D or 3D geometry (such as thickness variations in sediment-filled valleys) various effects can influence the wave-field, like trapping of the waves in the basins, basin-edge generated surface waves and fault guided waves. In such case, observed seismic amplification cannot be usually explained with a simple analytical solution, and complex numerical techniques are required instead. 2D/3D numerical models are also essential when the combined effect of the seismic source and the propagation path should be accounted for. This is for modelling a near-field scenario at close distance to the fault when directivity effects and source properties shape the wave-field.

The choice of the numerical technique depends on the problem type (e.g. flat models vs. models with topography), available computational resources and available input parameters (Table 2; see Semblat, 2011 for a review of the various numerical methods for wave-propagation simulation). Elastic isotropic solvers are generally used when the seismic velocities and densities are known and the effects of attenuation can be neglected. Anelastic effects are usually introduced through visco-elastic models that are easy to implement in frequency-domain methods. The quality factors can be obtained by direct measurements or, as these are often not available, from empirical correlations with other known parameters (e.g. Vs-Qs). Complex non-linear behaviour that is generally implemented in time-domain methods, and anisotropic models can be used in exceptional cases and when strong evidences for the occurrence of such phenomena are present (i.e., very close to the faults, where large strains are expected). Such solvers, however, require usually a large number of additional parameters, which are difficult and expensive to calibrate. A careful evaluation of the cost-benefits should be performed before the implementation. Linear anelastic modelling constitutes a conservative choice of the amplification in most of the cases. Non-linear soil response can significantly reduce the ground motion due to additional damping and changes in the frequency content of the ground motion. However, in some cases, it can even amplify the motion (e.g. hardening of the material, trampoline effect, etc.) Without field measurements or laboratory soil testing, non-linear soil-response calculations are not recommended. Finally a large number of scenarios (e.g. magnitude, distance, incident wave-field) need to be modelled to capture the variability caused the seismic sources, because some site-response features are source dependent.

Table 2. Common numerical methods used for wave-propagation simulation.

Numerical scheme	Domain	Model	Dimensions	Advantages	Disadvantages
Finite-differences	Time	Linear Non-linear	1D/2D/3D	Simple implementation; Simple model setup (mostly using regular grid discretization)	Difficult to model topography; instable if model not properly dimensioned
	Freq.	Linear	1D/2D (3D possible)	Attenuation properly modeled using complex velocities; Solution always stable.	Computational demanding in term of memory, limiting the method to the 2D case
Finite-elements	Time	Linear Non-linear	1D/2D/3D	Accurate representation of complex geometries	Can suffer numerical dispersion; Model meshing can be difficult for complex geometries
Spectral-elements	Time	Linear	1D/2D/3D	Highly accurate	Model meshing can be difficult for complex geometries
Boundary techniques	Time	Linear	2D	Good description of the radiation condition	Computationally expensive at high frequencies

4. Modelling non-linear soil behaviour

The description of non-linear effects has resulted in several review and state-of-the-art papers (e.g. Beresnev and Wen, 1996; Field et al., 1998; Archuleta et al., 2000; Idriss and Boulanger, 2008). Non-linear effects are applied in engineering practice and are accounted for in current building codes. The general practice is to use an *equivalent-linear* 1D or 2D physical model (e.g. program SHAKE developed by Schnabel et al., 1972) to account for non-linear effects. Such approximation is limited to a strain level of about 0.1%–0.5%, where the soil behaviour becomes very complex.

Truly non-linear calculation with an appropriate rheological model has to be applied in order to estimate the soil response at larger strain levels (e.g. program NOAH, Bonilla, 2001; program HERCULES, Taborda et al., 2012). However, such calculation requires additional parameters that are difficult to measure in situ or even impossible at large depths. Broadly speaking, such model describes the hysteretic behaviour of the stress–strain relationship and the generation of pore pressure under cycling loading and undrained conditions. Moreover, the dilatant nature and cyclic mobility of sands (e.g. Iai et al., 1990) can be taken into account. The non-linear material behaviour is commonly defined by the strain-dependence of the shear-modulus (G/Gmax) and hysteresis damping, which is generally measured with standardized geotechnical laboratory tests. The dynamic behaviour of a dilatant material is typically described by additional dilatancy parameters, which can be derived from laboratory measurements such as cyclic triaxial tests, field measurements (e.g. cone penetration tests, CPT) or strong motion records on vertical arrays (Roten et al., 2014). For more information related to specific methods, see Appendix C. A recent benchmark (Régnier et al., 2016) showed a considerable code-to-code variability. Such epistemic uncertainties are related to wave propagation modelling using different nonlinear constitutive models. In any case, without field measurements or laboratory soil testing, true non-linear soil-response calculations are not recommended.

Chapter V

Empirical observations

1. Introduction

A microzonation with strong-motion data can only be applied in high-seismicity regions and where the distribution of instruments is sufficiently dense. However, in many areas of the world, occurrence of strong earthquakes is (fortunately) not so frequent. This is particularly the case for low- and moderate-seismicity areas such as Northern Europe. However, modern strong-motion stations have become sufficiently sensitive to record low-magnitude and distant earthquakes, which are by far more frequent than big events. Furthermore, the development of new portable instrumentation at affordable costs facilitates the setup of temporary dense networks as alternative and to complement permanent installations. The value of such networks is that they can be installed in a limited area for short time span, and then moved on new target locations. Short-term installation can be deployed for aftershock measurements after strong earthquakes as well or for the monitoring of small events during a limited period of a seismic swarm.

If a sufficient number of low-magnitude events is collected, recordings can be used in microzonation studies for the calibration and verification of numerical site-response simulation, or to develop empirical amplification models. The limitation of using small-magnitude events is that they can approximate the effect of large earthquakes only in the linear domain. The low strain level is not sufficient to fully represent the non-linear behaviour of soils under strong dynamic loading. Moreover, frequency content and duration are generally different, requiring some adjustment to strong ground motion.

Nonetheless, empirical observations are an essential component of site-response analysis and microzonation studies, which cannot rely just on simulations to predict the effect of local geology on the ground motion.

2. Site response from empirical observations

There are different ways to isolate the effect of local site-response from ground motion recordings. In all cases, the influence of source and path propagation has to be removed from the recordings through a procedure called *deconvolution*, which in frequency domain is just a division between two earthquake spectra, affected and non-affected by site amplification effects. Such procedure implies knowledge of the source and path characteristics; these can be obtained from simultaneous measurement at sites supposedly free of local effects (so called *reference site techniques*) or can be modelled analytically or numerically using model assumptions (*non-reference site techniques*).

Among the reference site techniques we distinguish between the standard spectral ratio (SSR; Borcherdt, 1970) and the surface-to-borehole spectral ratio methods. The first approach is more common and consists in deploying two sensors, one at the target site, for which site amplification is wanted, and one on the rock reference, usually the closest outcropping rock site. The major problem is the identification of a suitable reference location, with a negligible site response and at the same time sufficiently close to the target site to assume similar path characteristics. These requirements can be problematic, as outcropping rock sites are often affected by surface weathering and fracturing, which lead to modification of the ground motion at high frequencies (Steidl et al., 1996). Vs-profiles at rock sites can as well vary significantly requiring Vs measurements at the reference site.

The problem is partially solved in surface-to-borehole methods (SBSR), where the reference sensor is located at large depth below the target site (e.g. Liu et al., 1992), but at the cost of drilling a borehole and using specialized equipment, and requiring Vs measurements in the borehole. Such method is nevertheless not completely free from complications; in practice, boreholes so deep to reach the sediment-bedrock interface are quite rare. Moreover, the reference sensor is not at free-surface condition and affected by the separation of the up-going and down-going waves reflected from the surface, which introduces frequency-dependent constructive and destructive interferences in the reference records.

In the second case of non-reference site techniques, a reference spectrum is modelled using the event-specific inverted source characteristics and ground-motion attenuation models for each station. The general form of source and path effects can be described through a regional ground motion prediction equation (GMPE) providing the spectral shape on a reference velocity structure as a function of a number of parameters (corner frequency or stress drop, seismic moment and distance, regional Q-model and geometrical spreading model, reference near site attenuation term, etc.; see for example Boore, 2003; Edwards at al., 2011). A generalized inversion scheme is needed to derive the different model parameters from the recordings, which might be affected by trade-offs. An efficient method to minimize these effects is to perform the inversion simultaneously on a large number of stations of the network and a large number of events. The advantage is that for a specific event, the source characteristics are common over the different stations, while for a specific station, site amplification can be considered constant between events of similar characteristic. Such redundancy reduces the non-uniqueness of the inverse problem, giving the possibility to better constrain the model parameters and minimizing the trade-off with the estimation of the local site-amplification factors. This procedure is nowadays routinely applied for all real-time stations of the Swiss network (Edwards & Fäh, 2013; Edwards et al., 2013) and is a by-product in the development of stochastic ground-motion prediction equation for Switzerland (see Figures 3 and 4).

Alternatively, the Empirical Green Function method can be used (Hartzell, 1978; Irikura, 1984). The method does not require the calibration of source and path models; the ground motion expected at a site from a large event is derived from stacking a number of low-magnitude events sharing the same target source area. In this way, realistic time histories can be produced, given that a sufficient number of recordings is available.

3. Rock reference conditions

All amplification functions (measured or simulated) need to refer to a common reference rock Vs-profile that should as well correspond to the rock reference for the regional probabilistic seismic hazard assessment. The use of an incorrect reference condition may lead to over- or underestimation of the final computed seismic hazard (Poggi et al., 2016). On a regional scale, the reference rock condition is often assumed a priory (e.g. Vs30 of 800m/s) by geological/geotechnical considerations. Such estimate is usually oversimplified. A well-defined rock reference velocity profile is needed to compare ground motion and hazard products calibrated on different reference profiles. Between-region adjustments can be provided to account for this diversity, such as the Vs-kappa adjustments.

One of the central components to derive a reference velocity profile is the development of a ground-motion prediction equation (GMPE) tailored to the regional crustal structure and seismicity, and the derivation of site-specific empirical amplification functions. Measured quarter-wavelength S-velocity at a site can then related to the corresponding frequency-dependent empirical amplification. Combining this comparison for many stations, a set of frequency-dependent calibration relationships can be established. Assuming that the reference profile is defined by a lack of any relative amplification, the quarter-wavelength velocity profile that corresponds to unitary spectral amplification can be extracted. The reference velocity profile can then be obtained through an inversion procedure and defines the regional rock reference for the ground-motion prediction equation (GMPE). More details related to Swiss reference rock profile can be found in Poggi et al. (2011).

4. Set-up of a temporary network

Depending on the number of available instruments, it is suggested to deploy a temporary network initially on a coarse grid taking into account the available geological information. This allows to overview the possible variability of the site response over the study area. At best, a station should be deployed on each different geological unit of the area. In a subsequent phase, a denser installation can be restricted to smaller regions of unclear or particular response. Areas at risk of special phenomena, such as edge generated surface waves or 2D/3D resonances, should also be taken into account when planning the temporary deployment. In this case, the decision on installation will mostly be driven by the results from numerical simulations and the interpretation of the H/V spectral-ratio survey.

The main problem in the deployment of temporary network is the search for suitable locations for the installation. Strongest limitations are related to the level of urbanization of the area, which also limits the available space for measurements. Major issues in this sense are related to:

- ✓ *Level of noise*. Since the target is to record mostly low-magnitude events, a low-level of anthropogenic noise is desirable, to provide better spectral resolution. However, in many cases high noise levels cannot be avoided in urban environment.
- ✓ *Free-field conditions*. To study the site-response and not the building response, stations should be placed in the free-field or in very small constructions. Careful documentation of the local conditions is mandatory for the interpretation of the recordings.
- ✓ *Effect of nearby structures (urban free-field)*. Large structures such as tall buildings and bridges can have a considerable soil-structure interaction, which can affect the estimation of the site-response. However, estimation of the combined effect of basin and structures can also be target of the analysis.
- ✓ *Permissions*. Since the stations will be deployed for a relatively long period (few months), some agreement should be made with the owners and local municipalities. An easy choice is the use of public structures, such as areas around schools, hospitals and cemeteries.

It is not possible to define univocally the amount of time required for the recordings of a temporary installation. This depends on the seismicity of the target area and the level of noise. In regions with moderate-to-high seismicity and low level of urbanization, a period of 2-3 months might be sufficient to collect an adequate number of events. In low seismicity regions and within big cities, a longer period might be necessary (e.g. 6 months to 1 year).

5. Macroseismic observations and damage survey

Macroseismic intensity is an empirical classification of the severity of ground shaking, based on observed effects on human beings, objects, buildings, animals and natural environment. Intensity level is defined by means of a discrete set of standardized descriptions of the earthquake effects. Several scales exist; currently in use in Switzerland is the European macroseismic scale (EMS 98) established in 1998 (Grünthal ed., 1998). An intensity scale provides a series of idealized descriptions of earthquake’s effects, starting with the very weakest (intensity I in EMS 98: the shaking is imperceptible) up to the very strongest (intensity XII in EMS 98: everything is totally destroyed). During an event, an intensity value can be assigned at each village or city quarter. The resulting intensity map gives a comprehensive picture of the pattern of the effects of an earthquake, including the effect of local site conditions.

Macroseismic observations (damage distribution or intensity variations) can then be used for the verification of site response models in microzonation studies (see Figure 11). The clear advantage with respect to instrumental monitoring is that reports on past moderate and large historical earthquakes can be used (if available). In low-seismicity regions, this might be the only mean to compare results from numerical modelling with observations of large events.

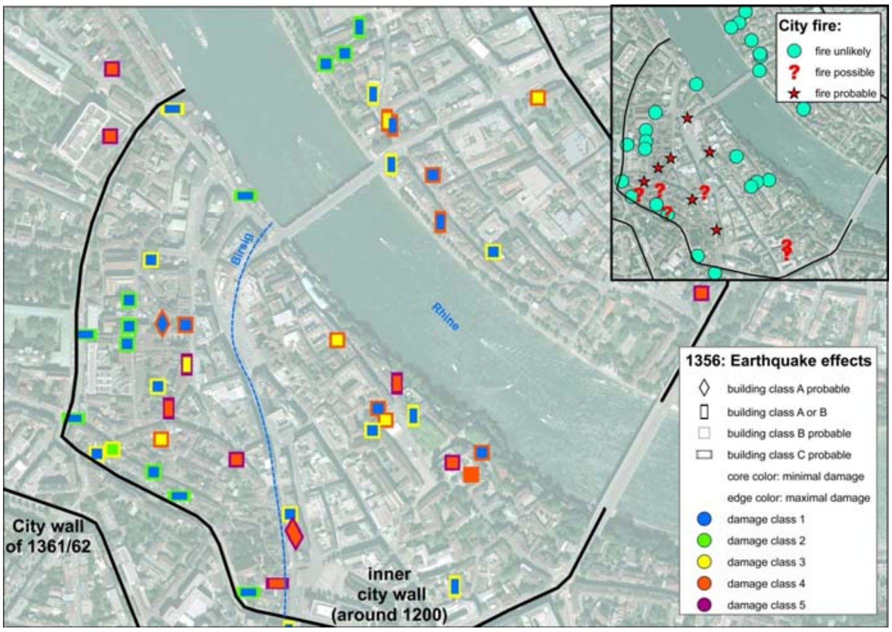


Figure 11. Overview of the assessed building damage in Basel after the 1356 earthquake, including the most probable vulnerability class and the range of possible direct earthquake damage. Damage is concentrated in areas with low building quality. Top right: archaeological traces of the fire after the earthquake (modified from Fäh et al., 2009).

Verification with macroseismic data is to be considered rather qualitative, since many sources of uncertainties are affecting the method. As first, uncertainty of macroseismic data depends on the accuracy of the respective observation on the effects produced by an event, including damage to buildings and objects and their vulnerability, but also subjective perceptions, which are very variable. Secondly, comparison with simulations requires the conversion from ground motion to intensity (e.g. Wald et al., 1999; Faenza & Michelini, 2010), which relies on the use of empirical relations calibrated on a large number of intensity observations from different places (and sometimes using different scales) and in different geological conditions.

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Appendix A

Engineering issues and available datasets

1. The reference earthquakes

For engineering applications, the definition of reference earthquakes and related ground motions, which are representative of the expected local shaking scenario, might be necessary. The reference events can be obtained from the de-aggregation of the site-specific regional seismic hazard and the selected return period. The earthquakes are defined by magnitude-distance pairs. These reference events and the related ground motion on the reference rock condition have to be corrected for the local seismic response, by direct convolution with the site-amplification function (measured or modelled). The resulting time-history can then be used for the computation of the 5% damped response spectra of the single degree-of-freedom system, or directly as input for numerical simulations of the building response.

The definition of the reference ground motions is usually done in two ways; by selection of recorded ground motion or by numerical modelling. The former is the most suitable, as real signals are used, but has the disadvantage to find adequate recordings for the selected magnitude-distance pairs on the predefined rock conditions. To face this problem, sometime small events are scaled in amplitude to match the desired ground-motion level or uniform hazard spectrum, while in other case events from regions extern to the study area are imported.

The waveform can also be modelled by numerical simulation. This has the advantage to deterministically provide a signal with desired characteristics. However, the modelling of the source and the whole propagation path can require detailed information (e.g. rupture time-function, stress-drop, anelastic and scattering attenuation) which is often unknown and epistemic uncertainties need to be considered carefully. A regional ground-motion prediction equation (GMPE) can be used to simulate stochastic ground motions that reflect many of the regional properties (e.g. Edwards & Fäh, 2013). In this case, only the Fourier spectrum of the target event is prescribed by the GMPE, and the waveform is then stochastically constructed from this, assuming random phase distribution, shape envelope and duration (e.g. Beresnev & Atkinson, 1997). A computation using complex deterministic numerical models can also be used, however is demanding and requires model details and computational resources not always available. The use of hybrid models might be an alternative combining deterministic at low frequency with stochastic simulations at high frequency.

2. Available geological and geotechnical datasets

In the following, we provide a description of geological and geotechnical datasets owned by the Federal Office of Topography (Swisstopo), cantonal offices, and universities or available at the Swiss Seismological Service (SED). These datasets are of potential interest for site-response assessment and microzonation at different scales (national, regional, local).

2.1. Geological, geotechnical maps

Geological, lithological and geotechnical maps are available in digital format (vector or georeferenced raster) from Swisstopo. These maps provide a first order representation of the main exposed geological units at different resolutions (1:500’000 to 1:25’000) and can be used as discrete proxy for seismic soil classification at level I (national) in combination with other information, such as surface topography, resonance frequency and sediment thickness. Direct application to level II (regional) and III (local) is however discouraged, due to the unavoidable oversimplifications used in describing the different units when compared to the true local variability; for analyses at regional and local scale, direct measurements and reinterpretation of available data (borehole, geophysical, geotechnical) should always be preferred.

Available geological information in Switzerland has been homogenized within the project GeoCover. The project had the goal of providing high quality vector description of the surface geology, including data from the previous Geological Atlas of Switzerland (1:25’000) and a compilation of special and unpublished geological maps at various scales. Conversely, the geotechnical map of Switzerland 1967 (1:200’000) of the Schweizerischen Geotechnischen Kommission (SGTK) is outdated. An update has been planned for years, however to our knowledge, the final product is not yet available.

Geotechnical and geological ground types can be used as a proxy for the estimation of mean site response over large areas, for instance by computing for each soil type the average seismic amplification from several empirical observations in the different frequency bands of engineering interests. With such simplified approach, epistemic uncertainty is expected to be significant, and needs to be carefully assessed. The advantage of such data is that hazard potential can rapidly be mapped over large areas by using GIS tools.

2.2. Soil classes

In order to facilitate the selection of the appropriate design spectrum of the Swiss Code SIA 261, the Federal Office for the Environment (FOEN) has supported cantonal soil-class maps showing 6 soil classes from A to F (Mayoraz et al., 2016). The soil classes A to E for which a spectrum is given in the building code are shown in Table A.1. Soil class maps are generally based on indirect evaluation of the surface geology and available borehole data. Epistemic uncertainty of the average amplification in the different soil classes is significant and overlapping between classes, as outlined in the PhD thesis by Steimen (2004) (see Figure A.1).

A study performed by SED for BAFU (Gassner-Stamm & Fäh, 2014) has shown the limitations of the geotechnical-based SIA 261 classification scheme when compared with direct Vs30 measurements. The comparison between Vs30 values received from geophysical measurements and the soil classes from geology shows discrepancy in about 50% of the cases, mainly if the site has a Vs30 value in the range of soil class B. In the 6 cases of soil class D, the class can be defined correctly from geological assessment. Soil class C assessed by geology in many cases are in fact soil class B or even A. More than 50% of these cases are not correct. Soil class B in the geological assessment is completely absent in this dataset which indicates a methodological problem in the procedure to develop soil class maps from geological information. This was also noted in the PhD by Steimen (see Figure A.1). Finally in 5 of 12 cases assessed as soil class A from geology; the assessment is not correct, most probably due to a thick layer of weathered material that could not be identified from geological considerations. Looking at measured f_0-values results using H/V spectral ratios indicate a rather good separation of soil class A and soil class E.

These results show that indirect extrapolation of geological/geotechnical classes alone is not necessarily reliable. On top of that, the relation between local amplification and site classes is generally non-univocal, and might lead to large errors in the final estimation of the local response, particularly when assessing amplification as a function of frequency. A combination with additional information, such as topography data and measured fundamental frequency of resonance, is therefore advised to define a new set of classes with reduced uncertainty in the site response.

Soil Class	**Description**	**Vs30 [m/s]**
A	Stiff or soft rock, covered by at most 5m soil layer.	> 800
B	Cemented gravel and sand and/or preloaded soils (including moraines) with a thickness above 30m.	400-800
C	Normal consolidated and not cemented gravels and sands and/or moraines with a thickness above 30m.	300-500
D	Normal consolidated fine sands and silts with a thickness above 30m.	150-300
E	Surface layer of soil classes C or D with a thickness between 5 and 30m, over soil deposits of class A or B.	-

Table A.1. Definition of soil classes according to the Swiss Code SIA 261 (2003). The existing soil class maps published before 2015 refer to this definition. The description in the 2014 version of the code was changed towards a geotechnical description (see Mayoraz et al., 2016).

2.3. Digital elevation models and topography data

Digital elevation models (DEM) are available from Swisstopo at a resolution of 25m (Figure A.2) and as small as 2m (SwissALTI3D). This information has a large potential for the use as continuous proxy in site response analysis both at large scale and for local studies, e.g. as input for direct numerical modeling. The main advantage is the possibility to perform automatically rapid calculations over large areas and at high resolution using standard software.

Nowadays, the large potential of using topographic information in site response analysis has not been fully exploited. Examples on effectively using topographic information are available in literature; in recent studies (Wald and Allen, 2007) slope gradient was used to assess geotechnical soil classes based on Vs30. Such approach, in spite of being widely used at local scales, is affected by considerable uncertainties in the estimation (Lemoine et al., 2012), which makes it unsuitable for the use at national scale without additional information.

In a similar manner, other topography-derived parameters can be useful to better assess geological and morphological features directly related to particular seismic response characteristics. This is the case of relative elevation (difference in altitude to a reference local value) and of topographic convexity (derivative of the slope), which are useful to evaluate deposition energy potential, related to sediment granulometry and ultimately affecting seismic propagation velocity (Stewart et al. 2013). Also texture parameters (such as roughness and pattern correlation length) might be of potential interest, being related to weathering and material type. As well, basin shape parameters (width, elongation), which are important for a first order evaluation of the occurrence of 2D/3D effects, can be derived automatically over large areas. As noted by Burjánek et al. (2014a, 2014b) all these parameters however relate to a specific length scale or wavelength that needs transformation into information in the frequency domain.

Direct influence of topography on ground motion has been extensively (and still is) analyzed in numerical studies. As described in the main report, geometrical effects in amplification can reach values up to a factor of 2, which is much lower than the factors observed during earthquakes. Local amplification is controlled in first place by the sub-surface velocity structure (Burjánek et al., 2014a).

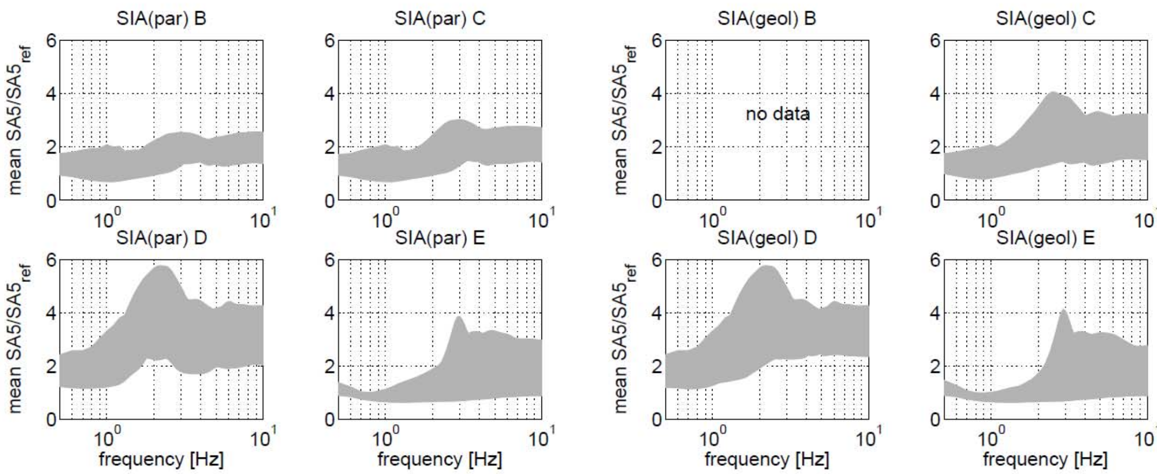

Figure A.1. Range of mean spectral amplification (5% damping) from 1D numerical modeling, for site classes defined using parametric (left) and geologic/geotechnical (right) definitions of site classes (Steimen, 2004).

2.4. 3D structural models

Within the framework of the transnational project GeoMol 2007-2015 (http://www.geomol.eu) partners from Austria, France, Germany, Italy, Slovenia and Switzerland are collecting data on the geological structures of the Molasse and Po basins. Swisstopo is the coordinating institution in Switzerland, and GeoMol will provide 3-dimensional subsurface information of the Molasse basin in the Swiss Foreland. Such information will help to improve regional seismic hazard assessment in the Swiss Foreland. Moreover, Swisstopo initiated project GeoQuat related a 3D model of quaternary in Switzerland. An initial model derived from different studies since the 1980ties was implemented at Swisstopo, but was recognized to be highly heterogeneous in quality. For this reason, a pilot study was lunched to improve this model in four areas (Birrfeld, Vierwaldstättersee, Aaretal, Visp) based on the analysis of additional data, in particular gravimetric and borehole data. Such pilot study allows to estimate the effort for a Swiss-wide 3D model of the quaternary. The usefulness of the existing 3D information would need further investigation in order to define regional reliability of this information as input for site-response analysis. In the long run, 3D models from project GeoQuat together with the assembled gravimetric and borehole data would be highly valuable datasets for level II studies, and in the end for level I studies as well.

For some areas (e.g. the wider Basel area) 3D geological models exist (Fäh & Huggenberger, 2006) and would need integration in GeoQuat and GeoMol.

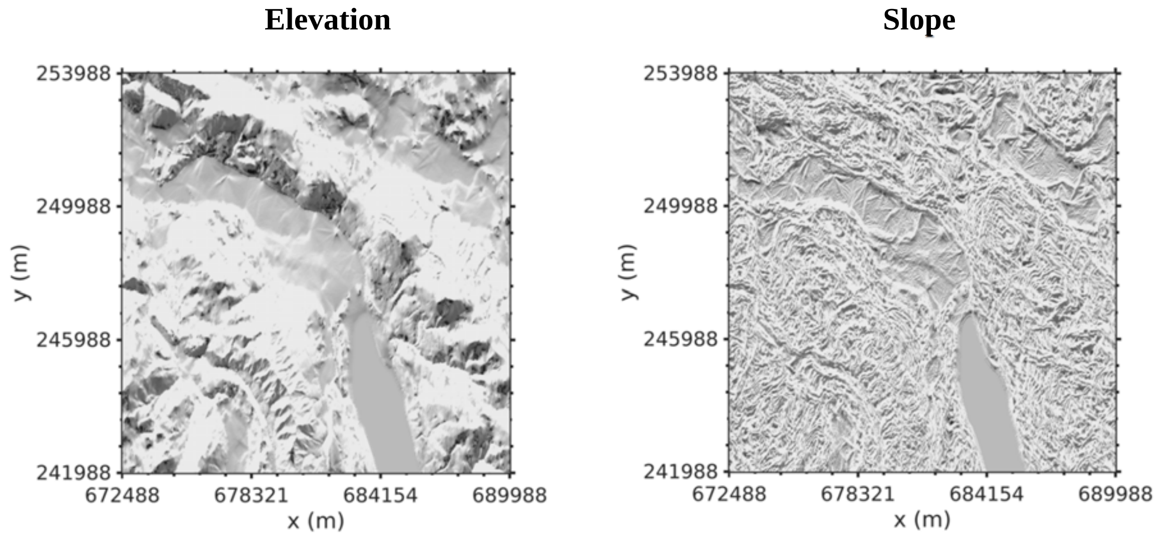

Figure A.2. Digital Elevation Model (DEM) at 25m resolution of the Zurich area. On the right the derived slope (gradient) model to be potentially used as additional proxy for ground type discrimination.
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3. Proxy definition

Empirical amplification factors are calibrated on a large number of ground motion observations and subsequently linked to a specific soil type by means of a *proxy*. A proxy is a measureable property of the site and can be continuous or discrete. In the former case, the given proxy (e.g. average velocity or topographic slope) or a combination of proxies is compared to the available empirical amplification factors at many sites, assuming a certain functional correlation between them. A prediction equation is then calibrated, which gives the possibility to extrapolate the ground motion amplification behaviour at any arbitrary value of the proxies (generally within the calibration range, or outside it, if a physically-constrained prediction model is implemented). The advantage of using continuous proxies is that a limited dataset can be used for the calibration, as the whole set is fitted simultaneously; however, the selection of a proper functional relation might be difficult and, if improper, leading to increased uncertainty in the prediction outside the range of calibration dataset. Complementary, discrete proxies are defined on properties which can be described by classes (e.g. geological and geotechnical domains, f_0 and Vs30 ranges). In this case, a local modifier is calibrated for each class as the mean and uncertainty of the corresponding ground motion amplification observations. There is no need to assume any particular functional relation, but a large dataset might be necessary, especially when a large number of classes is used. A continuous proxy can be transformed to discrete ranges. However, the definition of ranges can be very subjective, often controlled more by the availability of data than by any scientific justification. Conceptually, predictions based on continuous proxies should provide lower uncertainties, even though the quality of the prediction is mostly controlled by quality and accuracy of the input data, and the degree of correlation between proxy and observation.

4. Treatment of uncertainties

As for the case of probabilistic seismic hazard analysis, uncertainty in microzonation can be expressed by its aleatory and epistemic contributions. The aleatory part is related the intrinsic variability of the measured input parameters, such as the observed ground motion amplification and to some extent the material properties (seismic velocities, quality factors, shear resistance). This contribution can hardly be reduced, but it is essential to be properly quantified by defining appropriate confidence levels and assumed statistic. This information has then to be propagated through the whole chain of analyses and incorporated into the final model, with the only recommendation of carefully avoiding double-counting the uncertainties especially when microzonation is combined with the regional seismic hazard.

Complementary, epistemic uncertainty is related to the level of approximation when describing a certain physical phenomenon with a model, or related to the resolution of the structural models used in microzonation. It depends on the available knowledge and the complexity of the working assumptions. This contribution can conceptually be reduced by progressively introducing new measurement and knowledge on the model (at the expenses of increasing complexity) and improved methods of analysis based on better assumptions. A logic-tree approach is useful to take into account epistemic uncertainties. An analysis of the different branches that were introduced to simulate epistemic uncertainties is highly recommended. Practically, this provides an indication for setting priorities to those parameters that would need further assessments when an efficient reduction of epistemic uncertainties is required.

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Appendix B

Qualitative microzonation based on geology

Surface geology can be used to derive a qualitative microzonation in a region of interest. A qualitative microzonation map contains information on the 'positive' and 'negative' behavior of the soils in case of an earthquake. The procedure to derive a qualitative microzonation involves several steps:

a- Characterization of amplification relevant phenomena and parameters
b- Association of parameters to the surface geology in the region
c- Detailed mapping of the parameters (layers)
d- Definition of the importance of the individual layers (point-rating scheme)
e- Summation of all effects leading to the qualitative microzonation map
f- Calibration of qualitative map to ground motion variation

This concept will be explained at an example for the Basel area (Fäh et al., 1997; Noack et al. 1997). Seven parameters were considered in the Basel study (Figure B.1), four of them account for the influence of the Quaternary gravels in the region (see also Table B.1 for more details):

1 The consolidation of the Quaternary gravels expressed by their age. The lower a sediment is consolidated, the lower its shear-wave velocity and the higher is the expected amplification during earthquakes.

2 The type (grain size and lithification) of the Quaternary sediments. As a qualitative rule, it can be assumed that the smaller the grain size of sediments, the lower the shear wave velocity. Very soft sediments (clay, sand) have shear-wave velocities in the range 80m/s-200m/s, whereas gravels show velocities in the range 300-500m/s.

3 The thickness of the Quaternary sediments weighted by their type. The thickness of the sediments defines the frequency band at which amplification effects occur. The thicker the soils, the lower the fundamental frequency of resonance. Assuming the simple model of one layer over bedrock the expected amplification is maximum at the fundamental frequency of resonance and significant for the frequency range above. No amplification effects have to be expected below about half the resonance frequency.

4 Lateral variations in the thickness of the Quaternary sediments to account for the excitation of local surface waves and resonance. It is often observed during earthquakes, that lateral heterogeneities can excite local surface waves. Due to their strong attenuation, they propagate only over a limited distance, but within this range they can significantly increase duration and amplitude of the ground motion. Subsurface bedrock variations may also induce focusing of

wave energy at particular points at the surface. Sediment filled basin type structures and topographic features can be subject to resonance phenomena.

5 Parameter that considers the potential of liquefaction. The geologic and hydrologic factors that affect liquefaction susceptibility are the age and the type of sedimentary deposits, the looseness of cohesions-less sediments, and the depth to the ground water table. The liquefaction is mostly limited to water-saturated, cohesions-less, granular sediments at depths less than 10m. Due to the geological conditions in the Basel area, liquefaction is considered as of minor importance, as we find water saturated sands only at very few places. The liquefaction potential in our rating scheme is expressed by the depth of the water table.

6 Parameter that characterizes the lithological differences in the lithified Prequaternary sediments underneath the Quaternary gravels. Although geologically spoken they are considered as bedrock, seismically, these sediments belong to the sediments characterized by S-waves velocities of the order of 500-900 m/s. In our example, these sediments vary outside the Rhinegraben from Mesozoic limestones, mostly Muschelkalk (hard bedrock), to marls and shales of the Keuper and Opalinusclay (considered as soft sediments). Within the Rhinegraben the different lithologies of the Tertiary sediments, shales, sands and limestones, give a different contribution to the amplification of seismic waves at the surface.

7 Parameter that rates the influence of the lateral variation at the eastern bordering fault of the Rhinegraben. At this complex fault zone of Oligocene age a throw of about 1500m can be observed, bringing Tertiary sediments next to cristalline and calcareous bedrock. Resonance effects may be expected at very low frequency of the order of 0.4-0.5Hz, due to the basin structure of the area within the Rhinegraben nearby the fault.

Figure B.2 shows the maps for the seven parameters in the Basel area. The contributions of the effects of each parameter are classified to values between 0 and 4 units on a qualitative scale. Value 0 indicates no amplification with respect to a bedrock site and 4 a very strong amplification. Table B.1 gives an overview on the rating scheme. It has to be noted, that the proposed parameter set and the weighting scheme depends on the area under study, where the characteristics of the surface layers may have different importance. A similar study for an Alpine valley or a site characterized by lake deposits would require a different focus, parameter set and weighting scheme.

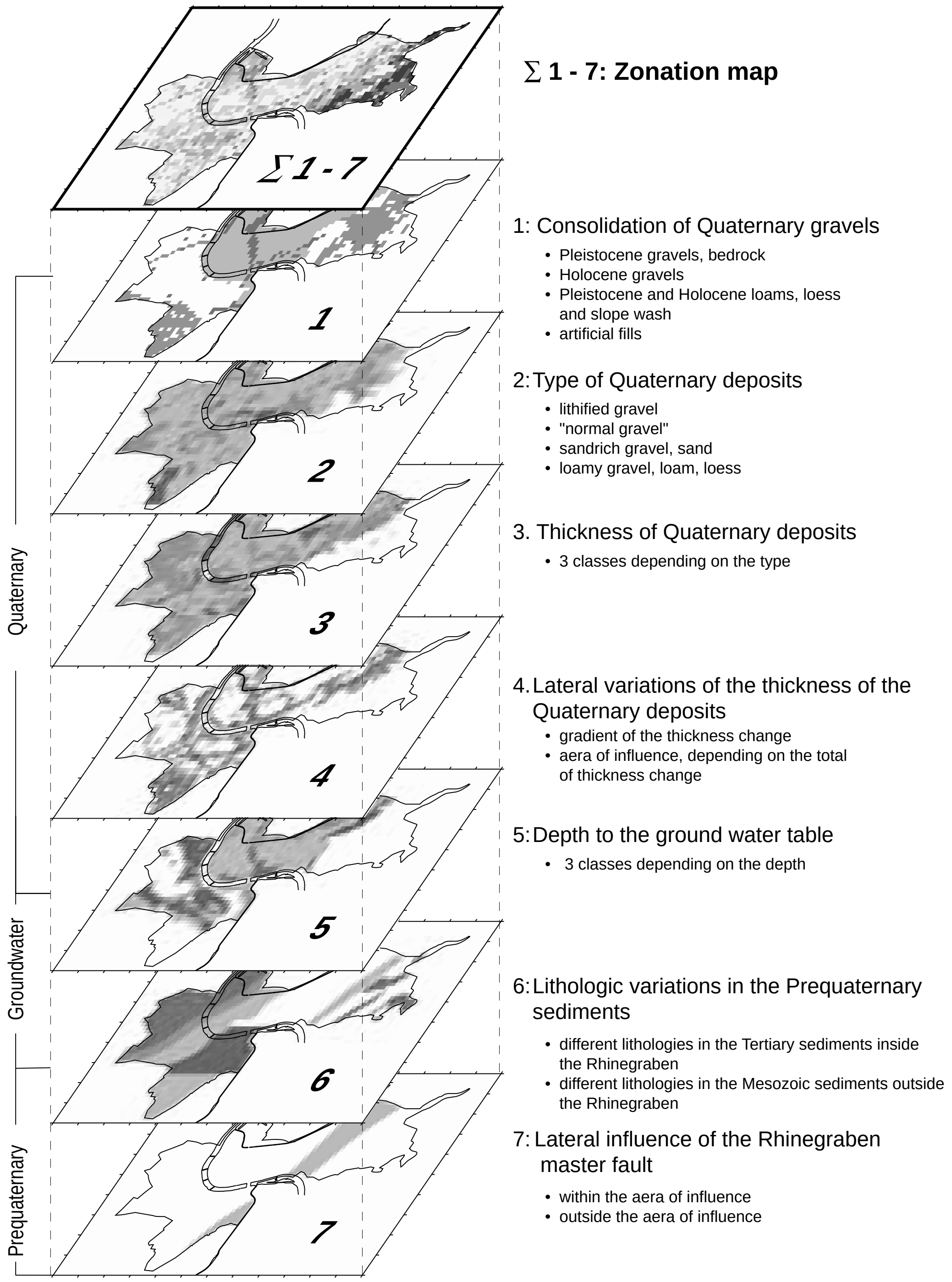

Figure B.1. Schematic representation of the application of a qualitative rating scheme. The local contributions of each characteristic parameter are mapped on a grid. The zonation map is the sum of all the different contributions at each grid cell.

PARAMETER	WEIGHT	REMARKS
1. Consolidation of the Quaternary sediments (as a function of age)		
• Pleistocene alluvium (highly consolidated)	0	• No contribution from Prequaternary lithified sedimentary rocks
• Holocene alluvium (medium consolidated)	2	
• Pleistocene and Holocene slopewash and Pleistocene loess (low consolidation)	3	• Map is compiled from geologic maps, well data, map of hazardous waste and outcrops at building sites
• artificial fill (very low consolidation)	4	
2. Type of Quaternary sediments (grain size and cementation)		
• lithified gravel: Gc	1	
• gravel with little sand, "normal" gravel: Gn	2	• map is compiled from lithologic descriptions of well logs
• sand rich gravel, sand: Gs	3	
• loamy gravel, loam, loess: Gl	3	
3. Thickness of Quaternary sediments (dependant on the type of the sediment)		
• Gn: 0-10 m; Gs, Gl: 0- 7 m; Gc: 0-12 m	1	• Weights are dependant on the type of the Quaternary deposit
• Gn: 10-20 m; Gs, Gl: 7-15 m; Gc: 12-25 m	2	• Map is calculated from well data, a digital terrain model and the map of paramter 2
• Gn: 20-45 m; Gs,Gl:15-45m; Gc: 25-45 m	3	
4. Lateral variations of the thickness of the Quaternary sediments		
• Gradient: 0.05 - 0.10	1	• For areas with a gradient ≥ 0.15 an extended zone is defined where local surface waves are expected
0.10 - 0.15	2	
≥ 0.15	4	• Map is calculated from the map of the thickness of the Quaternary sediments
• Extended zone: 12*Δh>d	2	
24*Δh>d>12*Δh	1	
(d=distance from lateral heterogeneity, Δh=thickness-change at the lateral heterogeneity)		
5. Depth to ground water table		
• 10 - 20 m	2	• Map is calculated from the map of the mean groundwater table and the digital elevation model
• 3 - 10 m	3	
• 1 - 3 m	4	
6. Lithologic variations in the lithified Prequaternary sediments		
Tertiary sediments		
• "Meletta Schichten" (lithified clay and marl)	4	
• transition zone from clay rich to sandy marl	3	
• uppermost "Meletta Schichten" (lithified sand, marl)	2	
• "Tüllinger Schichten", "Molasse Alsacienne"		• Weights for the "Tüllinger Schichten" and the "Molasse Alsacienne" correspond to a facies change from calcareous in the north to more sandy in the south
northern part	0	
eastern part	1	• Map is compiled from geologic maps, well data, outcrops at building sites and ambient noise measurements
southern part	2	
Mesozoic sediments outside the Rhinegaben		
• Middle Triassic limestones	0	
• Middle to Late Triassic Marls and Shales	4	
• strongly weathered sediments	4	
• Liassic limestones tectonically fractured near the master fault	2	
7. Lateral influence of the Rhinegraben master fault		
• within the area of influence (1000 m inside the Rhinegraben)	2	• Map is compiled from geologic maps and well data
• outside the area of influence	0	

Table B.1. Details of the qualitative rating scheme for Basel.

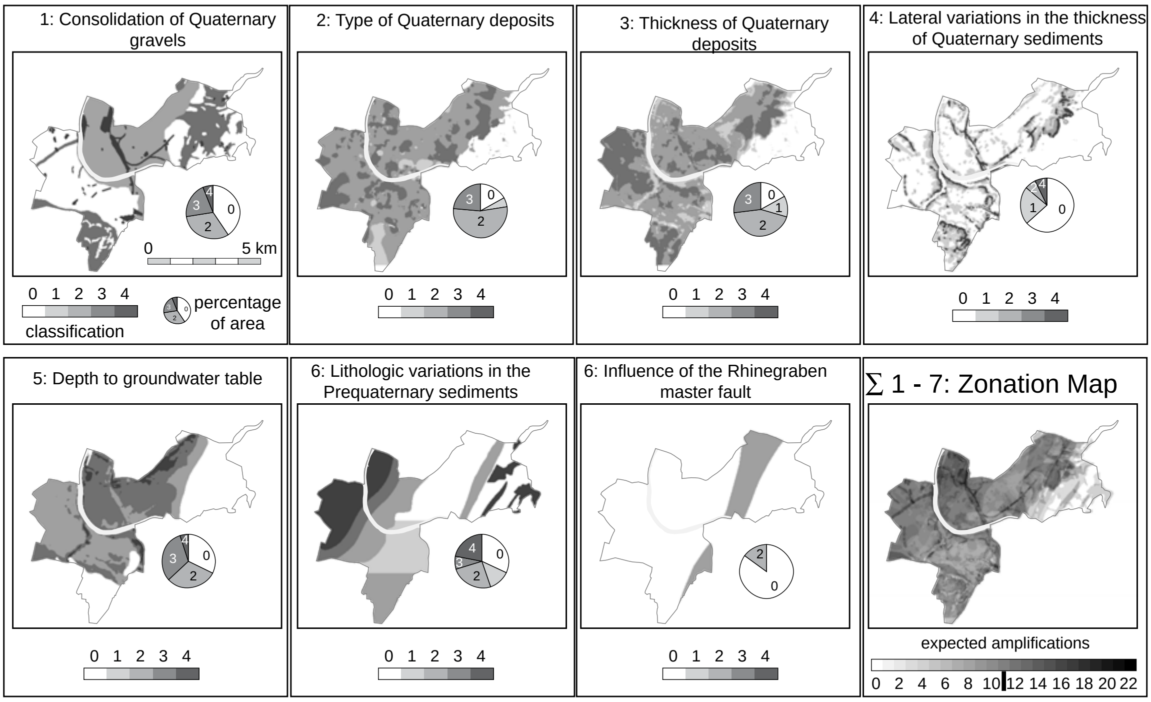

Figure B.2. Maps of the influences of the different parameters on local amplification according to the rating scheme (compare Figure B.1 and Table B.1).

The sum of all layers corresponds to a qualitative microzonation that rates the quality of the soft sediments (Figure B.2). The user of such maps has to be aware that such qualitative maps are only of an indicative nature, highlighting areas where the soil conditions may behave very unfavorable during earthquakes.

Rating schemes have a high level of uncertainty due to their subjectivity. This uncertainty can be accounted for by including the opinion of different experts. In our example, several experts agreed basically upon the importance of the seven parameters to contribute to local amplification effects. However, different opinions ‘expert opinions’ were given on the relative importance of each of these parameters. The application of different weights for the individual layers results in different microzonation maps (Figure B.3). And finally, the average of all experts provides the final qualitative microzonation. The uncertainty of such a map is expressed by the variability of the estimates given by all experts.

By comparison with observed macroseismic data the given range of values can be calibrated. For the Basel case, the range of values from 0 to 22 is estimated to correspond to +/- 1 intensity degree from the regional intensity value (Fäh et al., 2001).

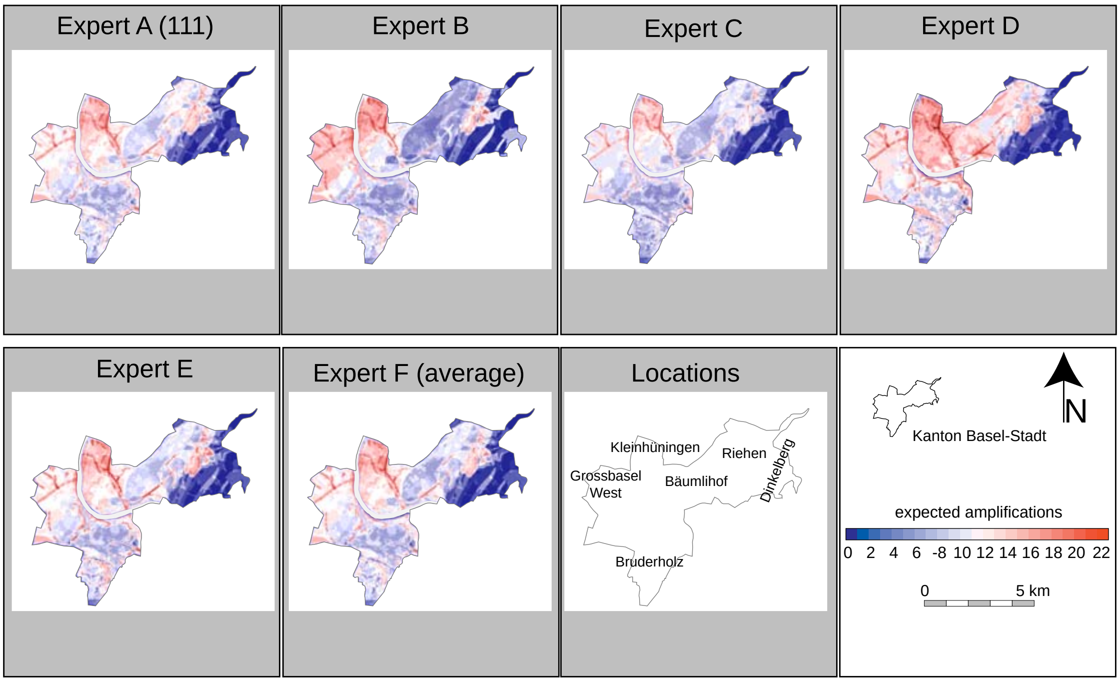

Figure B.3. Zonation maps calculated with the different expert opinions. The variation in the amplification from 0 to 22 units corresponds to about plus and minus one intensity degree (EMS scale) from an average regional intensity value.

As intensity is a qualitative measurement of ground motion, several attempts have also been made to derive some more quantitative relationships between surface geology and local amplification. As some of these relations were derived on sites where detailed intensity data and ground motion recordings were available (San Francisco and California), it has been possible to propose relations between the average horizontal spectral amplification and the intensity increment, and peak ground motion values (PGV) and macroseismic intensity (e.g. Borcherdt and Gibbs, 1976;Wald et al., 1999). For our Basel case, the database with recorded ground motion is too small to derive reliable relations to such ground motion measures.

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Appendix C

Estimation of nonlinear soil parameters from geotechnical tests

A contribution by Daniel Roten with input from Alain Pecker

Estimation of nonlinear soil parameters from geotechnical tests

This text gives a short overview of the soil parameters required by three fully nonlinear programs (NOAH, SUMDES and DYNAFLOW) and the geotechnical experiments (in-situ or laboratory) that are required to define them.

Linear parameters

It is assumed that the soil characteristics required for linear wave propagation are already known as a function of depth (Tab. 1). v_p may be measured directly during sonic logging if geotechnical

Description	Symbol	Characterisation
Compressional wave velocity	v_p	Seismic surveys, sonic logs
Shear-wave velocity	v_s	Array measurements, shear-wave seismics, RC test
Density	ρ	borehole logs, laboratory measurement
Quality factor for P-waves	Q_p	Empirical relations, attenuation studies
Quality factor for S-waves	Q_s	"

Table 1: Strain-independent soil parameters

drilling is carried out, or from reflection / refraction seismic surveys. v_s may be measured in situ using S-wave seismic surveys, multichannel spectral analysis of surface waves (MASW) or array measurements of ambient noise. If a resonant column (RC) test is performed, v_s may be calculated from the shear modulus at low strain $G_{\max}$ using $G_{\max} = v_s^2 \rho$. The density is determined in situ or in the laboratory. The quality factors Q_p and Q_s are usually not well constrained, but of minor importance since anelastic damping is typically less important than hysteretic damping during strong ground motion in nonlinear soils. Q_p and Q_s may be estimated from v_s using empirical relations (e.g. Brocher, 2006).

NOAH

The nonlinear anelastic hysteretic finite difference code NOAH (Bonilla, 2001) uses different soil models depending on the chosen parameters. It is able to treat undrained conditions of effective stress using the multispring mechanism model introduced by Towhata and Ishihara (1985), and an extension of this model which treats cyclic mobility and soil dilatancy (Iai *et al.*, 1990b,a).

Total stress analysis. For total stress analysis the multispring model gives the same result as a single element hyperbolic model following the Generalized Masing rules (Bonilla, 2000). It is very similar to the nonlinear model "NOAHH" described in Hartzell *et al.* (2004).

In the hyperbolic model, the reduction of the shear modulus G with increasing strain γ is described by

$$\frac{G}{G_0} = \frac{1}{1 + \frac{\gamma}{\gamma_r}} \tag{1}$$

where G_0 is the shear modulus at low strain. The reference strain γ_r is defined as

$$\gamma_r = \frac{\tau_0}{G_0}, \tag{2}$$

and τ_0 is the maximum shear stress that the material can support in the initial state. The nonlinear relation between stress τ and strain γ is described by a backbone curve during initial loading:

$$F_{\mathrm{bb}}(\gamma_{xy}) = \tau_0 \frac{\frac{\gamma_{xy}}{\gamma_r}}{1 + \left|\frac{\gamma_{xy}}{\gamma_r}\right|} \tag{3}$$

Subsequent loading and unloading cycles are expressed as

$$\frac{\tau_{xy} - \tau_r}{\kappa_H} = F_{\mathrm{bb}}\left(\frac{\gamma_{xy} - \gamma_r}{\kappa_H}\right) \tag{4}$$

The hysteresis scale factor κ_H controls the shape of the loop in the strain-stress space (Bonilla *et al.*, 1998), and κ_H equals 2 in the original Masing (1926) formulation. In the extended Masing rules (e.g. Pyke, 1979; Vucetic, 1990; Li and Liao, 1993) this constrain on κ_H is released to prevent the computed stress from exceeding the maximum strength τ_0 of the material. Bonilla (2000) generalized the Masing rules further by defining a variable hysteresis scale factor κ_H that assures the stress-strain path during each loading/reloading is bounded by the maximum shear strength τ_0. This hysteresis formulation was named the Generalized Masing rules because it includes the Cundall-Pyke hypothesis (Pyke, 1979) and Masing's original formulation as special cases.

In the original NOAHH code, the maximum shear stress τ_0 for the backbone curve (eq. 3) is calculated from the angle of internal friction φ and cohesion c (Hartzell *et al.*, 2004) using the Mohr-Coulomb failure criterion (e.g. Jaeger *et al.*, 2007):

$$\begin{aligned} \tau_0 &= \sigma_m \cdot \sin(\varphi) + c \cdot \cos(\varphi) \\ \sigma_m &= \sigma_v \frac{1 + 2K_0}{3} \end{aligned} \tag{5}$$

where σ_m is the effective mean stress, σ_v the vertical effective stress and K_0 the coefficient of Earth at rest. The parameters φ, c and K_0 are required for each layer in the model. The angle of internal friction φ (and the cohesion c) are determined in the laboratory from triaxial or torsional shear tests. To compute the effective mean stress the water table must also be known. The coefficient of Earth at rest K_0 is determined from the overconsolidation ratio (OCR) and φ using

$$K_0 = (1 - \sin\varphi)\mathrm{OCR}^{\sin\varphi} \tag{6}$$

If no value for the cohesion is available, the code can be slightly modified to compute the maximum shear stress τ_0 directly from the reference strain γ_r using equation 2. Since the reference strain generally increases with depth, it must be specified for each layer in the soil. This requires resonant column or shear tests at different confining pressures.

In the hyperbolic model the damping ratio at large strains approaches $2/\pi \sim 63\%$ (Ishihara, 1996), which is much larger than the typical damping ratio of 20–25% for clays or 30–40% for sands. Ishihara *et al.* (1985) suggested a method to control the damping ratio by computing a new backbone curve, which follows a hysteresis path controlled by the required damping ratio. The required strain-dependent damping ratio ξ_H is calculated with the following expression (Hardin and Drnevich, 1972):

$$\xi_H = \frac{\frac{\gamma}{\gamma_r}}{1 + \left|\frac{\gamma}{\gamma_r}\right|} \xi_{\max} \tag{7}$$

where γ is the level of deformation and $\xi_{\max}$ is the maximum damping ratio at large strains. By equating ξ_H in eq. 7 with the damping ratio from the hyperbolic model, NOAH finds a solution for the reference strain γ_r' that is compatible with the desired damping value ξ_H. This new reference strain γ_r' is then used to recompute the backbone curve, and the procedure is repeated for each time step. The maximum damping ratio $\xi_{\max}$ is derived directly from damping curves, which are developed from resonant column, torsional shear or triaxial shear tests.

In addition to hysteretic damping, NOAH models intrinsic attenuation with constant Q (Tab. 1) by the rheology of the generalized Maxwell body (Day, 1998). Table 2 lists the nonlinear parameters required by NOAH for total stress analysis. For a more detailed description of the model refer to Bonilla (2001).

Description	Symbol	Characterisation
Angle of internal friction	φ	shear tests
Cohesion	c	
Reference strain (depth-dependent)	γ_r	shear test or RC test
Water table	z_w	geotechnical drilling
Overconsolidation ratio	OCR	laboratory tests on soil samples
Coefficient of Earth at rest	K_0	computed from OCR and φ using eq. 6
Maximum damping ratio	$\xi_{\max}$	damping curves (shear tests)

Table 2: Nonlinear parameters required by NOAH for total stress analysis. γ_r can be used instead of φ and c.

Effective stress analysis. For effective stress analysis the parameters $\xi_{\max}$, c and γ_r are not required. Instead, the phase transformation angle φ_p, porosity n and five dilatancy parameters must be specified for layers where pore water pressure is simulated. The porosity n is determined in situ or on samples in the laboratory. It is related to the void ratio e as

$$n = \frac{e}{1+e} \tag{8}$$

The phase transformation angle φ_p can either be determined by examining stress path in cyclic triaxial tests or computed from the angle of internal friction φ (Ishihara and Towhata, 1982):

$$\tan \varphi_p = \frac{5}{8} \tan \varphi \tag{9}$$

By examination of laboratory data, (Towhata and Ishihara, 1985) found that the excess pore water pressure correlates with the cumulative shear work produced during cyclic loading. The model developed by (Iai *et al.*, 1990a,b) describes this correlation with five dilatancy parameters: the parameter w_1 controls overall dilatancy; p_1 the initial and p_2 the final phase of dilatancy; S_1 represents the ultimate limit of dilatancy and c_1 the threshold limit. Details of this constitutive model, as well as a description of the procedure for determining the dilatancy parameters from laboratory data, can be found in (Iai *et al.*, 1990a,b). The dilatancy parameters must be determined by fitting laboratory data from stress-controlled cyclic mobility tests. The following test data must be provided:

i. a liquefaction resistance curve (i.e. the cyclic shear stress ratio vs. the number of the cyclic loading N_1 required to cause shear strain of 5% in double amplitude)

ii. an envelope of the excess pore water pressure generation curve

iii. an envelope of the shear strain amplitude

NOAH provides a tool that simulates simple shear laboratory experiments for a given set of dilatancy parameters. By using a trial-and-error procedure, these parameters are adjusted manually until a satisfactory fit to the observed values are found.

SUMDES

The program SUMDES (Li *et al.*, 1992) performs dynamic response analyses of Sites Under Multi-Directional Earthquake Shaking. It implements the reduced order bounding surface model, which is capable of effective stress analysis. Table 3 lists the key parameters required by the code. SUMDES also requires the angle of internal friction ϕ, which can be determined from monotonic

Description	Symbol	Characterisation
Angle of internal friction	ϕ	shear tests (laboratory)
Soil compressibility (virgin or unloading/reloading path)	λ, κ	laboratory, or eq. 11
Modulus coefficient	G_0	from eq. 10
Shear modulus / strain relationship	h_r	Modulus reduction curve
Liquefaction resistance	d, k_r	laboratory
Shear path during virgin loading	b	$\sim$2
Shear strain increment due to mean effective normal stress	h_p	$\sim$35
Phase transformation / failure line slope	R_p/R_f	stress path

Table 3: Parameters required by SUMDES

triaxial compression tests. The coefficient G_0 defines the elastic shear modulus and can be derived from the low-strain shear modulus $G_{\max}$ and the initial void ratio e_0 using

$$G_{\max} = G_0 \frac{(2.973 - e_0)^2}{1 + e_0} \sqrt{p \cdot p_{\text{atm}}} \tag{10}$$

λ defines the soils compressibility along the isotropic virgin loading path. The value of λ is usually insignificant to the analysis results, since earthquake loading does not usually follow the virgin loading path. A typical value is $\lambda = 5\ \kappa$. κ represents the soil compressibility along the unloading / reloading paths. The value of κ can be found experimentally or estimated using

$$\kappa = \frac{3(1 + e_0)^2 \cdot (1 - 2v)}{2G_0(2.973 - e_0)^2\ (1 + v)} \tag{11}$$

where v is the Poisson ratio. The coefficient h_r describes the relation between shear modulus and shear strain magnitude. It can be defined by fitting of modulus reduction curves with a utility program provided by SUMDES.

For effective stress analysis SUMDES requires the parameters d, h_r, R_p/R_f b and h_p. d and h_r characterize the rate of the effective mean normal stress change caused by shear unloading and loading, respectively. These parameters are tuned by running a utility program (`TESTMODL`), which basically simulates triaxial shear tests, until a satisfactory fit between experimental and synthetic results is obtained. The ratio of the slopes of phase transformation to failure line R_p/R_f can be obtained from the friction angle ϕ and phase transformation angle ϕ_p

$$\frac{R_p}{R_f} = \frac{\tan\phi_p}{\tan\phi} \tag{12}$$

For clays, which exhibit little dilatancy, this ratio equals 1; for sands it is normally between 0.7 and 0.8. The parameter b defines the stress path during virgin loading. It has little effect on the results and is typically set to 2. h_p describes the amount of the shear strain increment due to the change of the maximum effective mean normal stress in the loading history. Modeling results are usually insensitive to the choice of b, and Li *et al.* (1992) recommend a typical value of 35.

DYNAFLOW

Dynaflow[1] is a general purpose finite element analysis program for linear and nonlinear, two- and three-dimensional, elliptic, parabolic and hyperbolic initial boundary value problems in structural, solid and fluid mechanics (Prévost, 2010). It comes with a library for different materials, including a multi-yield elasto-plastic model (Prévost, 1978, 1985) that is applicable to both cohesive and noncohesive soils. The model employs a collection of nested yield surfaces in the stress space. In the case of cohesive materials (clays, silts) von Mises type surfaces are employed, while Mohr-Coulomb type surface are used for non-cohesive materials (sands, gravel).

The parameters required by Dynaflow are subdivided into state parameters, low strain elastic parameters, yield and failure parameters, and dilation parameters. State parameters include solid mass density ρ_s, fluid mass density ρ_f, porosity φ and permeability k; these are determined in the laboratory. The shear modulus G, bulk modulus B and fluid bulk modulus K_f represent low-strain elastic parameters. Below the water table G is derived from v_s, ρ_s and porosity φ using:

$$G = (1 - \varphi)\rho_s v_s^2 \tag{13}$$

If Poisson's ratio v is known the bulk modulus can be derived from the shear modulus:

$$B = \frac{2G(1 + v)}{3(1 - 2v)} \tag{14}$$

However, it's better to directly measure B from the volumetric changes in monotonic drained triaxial tests. The fluid bulk modulus K_f can be derived from the following relation:

$$B = \rho v_p^2 - \frac{4}{3}G - \frac{K_f}{\varphi} \tag{15}$$

In perfectly saturated soils K_f assumes a value of $\sim 2.2 \cdot 10^9$ Pa, which is the bulk modulus of water. The obtained values of the shear and bulk soil moduli must be converted to a value at an arbitrary reference mean effective stress p'_1 using a power law:

$$x = x_1 \left(\frac{p'}{p'_1}\right)^n \tag{16}$$

where x stands for the shear modulus or bulk modulus, x_1 refers to the corresponding value at the reference mean effective stress, p'_1, and p' denotes the actual mean effective stress. The power exponent n is a material constant.

Yield and failure parameters required by DYNAFLOW are the angle of internal friction $\phi$and the three parameters α, x_l and x_u describing the shape of the stress-strain curve:

$$y = e^{-\alpha x} f(x, x_l) + (1 - e^{-\alpha x}) f(x, x_u), \quad f(x, x_i) = \frac{(2x/x_i + 1)^{x_i} - 1}{(2x/x_i + 1)^{x_i} + 1} \tag{17}$$

[1] http://www.princeton.edu/~dynaflow/

where

$$y = \frac{\tau}{\tau_{\max}}, \quad x = \frac{\gamma}{\gamma_r}, \quad \gamma_r = \frac{\tau_{\max}}{G_l} \tag{18}$$

These parameters are determined by fitting of modulus reduction curves using a trial and error method. Additionally, the coefficient of Earth at rest K_0 must be known to define the initial stress state.

Dilatancy parameters include the phase transformation angle $\overline{\phi}$ and the material parameter X_{pp}. $\overline{\phi}$ is determined from volumetric strain measurements, or calculated from the angle of internal friction ϕ assuming a constant ratio between $\tan\overline{\phi}$ and $\tan\phi$ (eq. 12). For the determination X_{pp}, cyclic triaxial shear tests need to be carried out. Numerical simulations of these tests are performed using different values of X_{pp} until the simulated behaviour is consistent with the observations. Alternatively, the dilatation parameter X_{pp} may be evaluated based on results of liquefaction strength analysis.

Summary

Since the three codes are based on different soil models, the soil parameters required as input are different as well. However, they rely on the same geotechnical tests to define the required soil parameters. Depending on the specified parameters, all the three codes allow to model only total stress or effective stress including dilatancy. The three models require a varying number of state or low-strain parameters, though some of these (e.g. fluid density, permeability) are only required for saturated layers. For total stress analysis the three codes only require the angle of internal friction, the water table and one to three additional parameters to model the modulus degradation. NOAH and DYNAFLOW also require the cohesion. NOAH also requires the damping ratio at large strains $\xi_{\max}$ for damping control.

Geotechnical tests

This section gives a list of geotechnical tests than may be used to determine the parameters listed in Table 4.

Drilling profile. From evaluation of the drilling profile the stratigraphic layering and the depth of the groundwater table z_w (required for all three codes) can be determined. The P-and S-wave velocities and bedrock porosity may be provided by borehole logging, if available. It is recommended to achieve undisturbed cores for further testing of the material in the laboratory.

Standard geotechnical tests in-situ or on soil samples. Laboratory tests on undisturbed soil samples are performed to define the in-situ dry density, the relative density, porosity and specific weight of the solid phase ρ_s. Additionally the overconsolidation ratio is determined from undisturbed soil samples, from which the coefficient of Earth at rest K_0 can be determined using eq. 6.

Static triaxial shear tests. To determine the angle of internal friction and the cohesion, drained and undrained triaxial shear tests are performed on at least three specimens at different confining pressures in a realistic range of occurence in the soil strata.

	NOAH	SUMDES	DYNAFLOW
Low-strain parameters			
P-wave velocity	v_p		
S-wave velocity	v_s		
non-dimensional modulus coefficient		G_0	
low-strain elastic moduli			G_l, B_l
density	ρ	ρ	
solid mass density		ρ_s	ρ_s
fluid mass density			ρ_f
water table	z_w	z_w	z_w
permeability		k	k
Quality factor	Q_s		
damping ratio for shear / compression		η_n, η_c	
bulk modulus of pore fluid		κ_f	κ_f
soil compressibility (virgin path)		λ	
porosity	n		φ
reference mean effective stress			p'_l
power exponent			n
Total stress analysis			
angle of internal friction	φ	ϕ	ϕ
cohesion	c		c
shear modulus degradation		h_r	α, x_l, x_u
damping control	$\xi_{\max}$		
coefficient of earth at rest	K_0	K_0	K_0
Additional parameters for effective stress analysis			
phase transformation angle	φ_p		$\overline{\phi}$
slopes ratio		$\frac{R_p}{R_f}$	
soil compressibility (un-/reloading)		κ	
dilatancy parameters	w_1, p_1, p_2 S_1, c_1	d, b h_p	X_{pp}

Table 4: Summary of soil parameters required by the three different codes

Resonant column tests. Resonant column (RC) tests provide another method to define the modulus degradation curves and damping curves as a function of shear strain. RC tests also allow a direct measurement of the maximum shear-wave velocity $v_{s,\max}$ in the laboratory. Usually the results from RC tests show lower values than measured in the field.

Cyclic triaxial shear tests. Cyclic triaxial tests are a key requirement to characterize the dynamic properties of soil, which are required for effective stress analysis. The cyclic triaxial tests need to be performed in undrained, stress-controlled conditions for a series of cyclic shear stress ratios until the strain reaches 5% double amplitude. (The cyclic shear stress ratio is defined as the applied shear strain τ_{xy} normalized by the initial effective confining pressure σ'_{m0}). Results are presented as pore water pressure, stress and strain as a function of time (or the amount of equivalent load cycles, respectively). The evaluation of the hysteresis loops of stress vs. strain allows determination of damping. Additionally, the liquefaction resistance curve is plotted, which is defined as the cyclic shear stress ratio as the numbers of cycles to failure (5% double amplitude). To derive the liquefaction resistance curve at minimum three tests are required for a given confining stress. The confining stress needs to be chosen as realistic as possible. As dilatancy is stress dependent, testing has to be done at different confining pressures.

NOAH, SUMDES and DYNAFLOW come with auxiliary programs which are able to simulate stress-controlled cyclic triaxial shear tests. To calibrate the dilatancy parameters the model is adjusted until the simulated shear tests reproduce the behavior observed in the laboratory.

Standard penetration and cone penetration tests. Results of standard penetration tests (SPT) or cone penetration tests (CPT) may be used for calibrating the dilatancy parameters in an indirect way. First a liquefaction resistance curve is derived from SPT or CPT data using empirical relationships (e.g. Idriss and Boulanger, 2006). Simulations of cyclic triaxial shear tests are performed with varying parameters, and the corresponding synthetic liquefaction resistance curve is defined from the simulated soil response. The dilatancy parameters are then adjusted until the calculated liquefaction resistance does not contradict the curve derived from the SPT tests. The advantage of this method consists in the use of in-situ soil measurements, rather than laboratory tests with undisturbed soil samples, which are often difficult to obtain. However, the correlation between liquefaction resistance and SPT blow counts is largely empirical, which represents a disadvantage of this approach.

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