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LIQUEFACT

Assessment and mitigation of liquefaction potential across Europe: a holistic approach to protect structures/infrastructure for improved resilience to earthquake-induced liquefaction disasters.

H2020-DRA-2015

GA no. 700748

Deliverable D5.3

Community resilience and cost/benefit modelling:

Socio-technical-economic impact on stakeholder and wider community

v. 1.0

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List of Partners

Participant	Name	Country
ARU (Coordinator)	Anglia Ruskin University Higher Education Corporation	United Kingdom
UNIPV	Universita degli Studi di Pavia	Italy
UPORTO	Universidade do Porto	Portugal
UNINA	Universita degli Studi di Napoli Federico II.	Italy
TREVI	Trevi Societa per Azioni	Italy
NORSAR	Stiftelsen Norsar	Norway
ULJ	Univerza v Ljubljani	Slovenia
UNICAS	Universita degli Studi di Cassino e del Lazio Meridionale	Italy
SLP	SLP Specializirano Podjetje za Temeljenje Objektov, D.O.O, Ljubljana	Slovenia
ISMGEO	Istituto Sperimentale Modelli Geotecnici Societa a Responsabilita Limitata	Italy
Istan-Uni	Istanbul Universitesi	Turkey

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Glossary

Acronym	Description
B/C	Benefit to Cost ration
CBA	Cost Benefit Analysis
CEA	Cost-Effectiveness Analysis
CI	Critical Infrastructure
CRED	Centre of Research on Epidemiology of Disasters
DRR	Disaster Risk Reduction
EEFIT	Earthquake Engineering Field Investigation Team
EERI	Earthquake Engineering Research Institute
EM DAT	Emergency Event Database
FEMA	Federal Emergency Management Agency
FM	Facilities Management
IAB	International Advisory Board
IVSC	International Valuation Standards Council
LRG	Liquefact Reference Guide
PAGER	Prompt Assessment of Global Earthquake for Response
RAIF	Resilience Assessment Improvement Framework
SELENA	Seismic Loss Estimation using a Logic Tree Approach
WTA	Willingness To Accept
WTP	Willingness To Pay

Contents

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Executive Summary

Recent events have demonstrated that Earthquake Induced Liquefaction Disasters (EILDs) are responsible for significant structural damage with, in some cases, EILDs accounting for up to half of the economic loss caused by earthquakes. With the causes of liquefaction being largely acknowledged, it is important to recognise the factors that contribute to its occurrence; to estimate the impacts of EILD hazards; and to identify and implement the most appropriate mitigation strategies that improve both building/critical infrastructure and community resilience to an EILD event. The LIQUEFACT project adopts a holistic approach to address the mitigation of risks to EILD events. The LIQUEFACT project sets out to:

- Achieve a more comprehensive understanding of the impacts that EILD events have on the resilience of communities and buildings/critical infrastructure on which they rely;
- Achieve a more comprehensive understanding of the range of mitigation techniques (technical, operational, managerial and organizational) that can be implemented to improve the resilience of communities and building/critical infrastructure to EILD events;
- Develop more appropriate mitigation techniques (technical, operational, organizational and managerial), for both European and worldwide situations; and
- Develop a Resilience Assessment and Improvement Framework (RAIF) to allow community and building/critical infrastructure stakeholders to make the business case for mitigation interventions.

This report outlines a range of cost-benefit analysis models that can be used as part of the Resilience Assessment and Improvement Framework (RAIF) to evaluate alternative EILD mitigation options for improving the resilience of community and critical infrastructure to EILD events. The alternative

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models will be tested in Work Package 7 and integrated into the final LIQUEFACT Reference Guide (LRG) tool (Work Package 6) and Built Asset Management Plan (Work Package 5 - Deliverable 5.4).

Introduction, Goal and Purpose of this document

The aim of this report is to provide an overview of cost-benefit analysis and outline a framework for its application to improving community and critical infrastructure resilience to future EILD events across Europe. To this end the report will:

- Present the background and context to the LIQUEFACT project;
- Describe the background to cost-benefit analysis and review its application in disaster mitigation;
- Outline a generic cost-benefit framework for evaluating mitigation options to improve community and critical infrastructure resilience to EILD events;
- Present alternative approaches which could be used as part of the LIQUEFACT RAIF;
- Outline the process for integrating cost-benefit analyses into the LRG;
- Summarise the key issues that will need to be considered by the LIQUEAFCT project partners as they use cost-benefit analysis to evaluate potential mitigation actions to improve community and critical infrastructure resilience to EILD events.

The framework presented in the report should be considered a work in progress which will be amended and modified throughout the duration of the LIQUEFACT project to reflect emerging issues identified by project partners, the International Advisory Board (IAB) and any location specific characteristics of the case study sites identified by the external stakeholders.

Goal: **This document aims to provide the project partners and researchers with an overview of cost-benefit analysis and presents alternative approaches that could be applied to evaluate the potential of mitigation actions to improve community resilience to EILD events.**

Scope of this document

This is a working document that will be amended and modified to reflect changing needs of the LIQUEFACT project and the views of the external stakeholder group and external advisory panel.

Target Audience

This is primarily an internal document intended for the LIQUEFACT partners and researchers.

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Community resilience and cost-benefit modelling

Socio-technical-economic impact on stakeholder and wider community

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1. Introduction

1.1 This report provides an overview of cost-benefit analysis and outlines a framework for its application to improving community resilience to future earthquake induced liquefaction disaster (EILD) events across Europe. The report presents:

- An overview of the aims and objectives of the LIQUEFACT project;
- The background to cost-benefit analysis and reviews its application to the built environment development process and as an options appraisal tool for disaster mitigation;
- Outlines of a generic cost-benefit framework for evaluating mitigation options to improve community and Critical Infrastructure (CI) resilience to earthquake induced soil liquefaction disaster events;
- Alternative approaches which could be used as part of the LIQUEFACT Resilience Assessment and Improvement Framework (RAIF);
- Outlines of the process for integrating cost-benefit analyses into the LIQUEFACT Reference Guide (LRG); and
- A summary of the key issues that will be addressed by the LIQUEAFCT project partners as they use cost-benefit analysis to evaluate potential mitigation actions to improve community and CI resilience to EILD events.

1.2 The framework presented in this report should be considered a work in progress that will be amended and modified throughout the duration of the LIQUEFACT project to reflect emerging issues identified by project partners, the International Advisory Board and any location specific characteristics of the case study sites identified by the external stakeholders.

2. Background and Context to the LIQUEFACT Project

2.1 The LIQUEFACT project aims to develop a more comprehensive and holistic understanding (primarily from a European point of view) of the earthquake-induced soil liquefaction phenomenon and the effectiveness of alternative mitigation techniques (available now and currently under development) that can be used to protect structural and non-structural systems and components from its effects.

2.2 Earthquakes are one of the most destructive natural phenomena. During the 20th century they caused the deaths of 1.5 million people worldwide; and incurred an estimated economic loss of €75 billion in the last quarter alone. Over the past decade, earthquakes proved to be the deadliest of all European disasters, with almost 19,000 fatalities and direct economic losses of approximately €29 billion[1]. A large part of Europe is at risk from earthquake disaster events, with the most seismically active areas being located in Italy, Greece, Turkey, Cyprus and Portugal, Switzerland, Slovenia, Macedonia and Island. Other European countries, such as Croatia, Bulgaria, Slovenia, Romania, France, Austria, Spain, the Czech Republic, Germany and Malta are also at risk (Woessner et al., 2013). While rehabilitation and strengthening of buildings and infrastructures after earthquakes is a widely studied subject, foundation improvement to mitigate the effects of earthquakes on them and the superstructures is still a subject of investigation in order to make those techniques less invasive and costly. This is particularly true when the earthquakes result in soil liquefaction.

2.3 Excessive deformations of the ground surface caused by earthquakes are of great concern for civil engineering works, human lives and the environment. Such ground deformations are sometimes associated with soil liquefaction. Earthquake-induced soil liquefaction is a phenomenon where the soil decreases in strength and stiffness as a result of increased pore water pressure in saturated cohesionless materials during seismic ground shaking (as a result of the applied stress); hence the soil behaves like a liquid and not a solid (National Academy of Sciences, 2016).

2.4 This phenomenon is often classified as a ground failure caused by earthquakes (National Institute of Building Sciences, 2013). It does not generally result in direct fatalities, but in significant damage to structures and infrastructures (public buildings, including schools and hospitals; together with elevated highway and port installations, water treatment facilities, crude oil storage tanks etc.) during and after a seismic event; sometimes resulting in collapse of structures and infrastructures. Recent experiences with EILDs; e.g. 2012 Emilia, northern Italy (Cimellaro et al., 2013); 2011 Tohoku Oki (National Academies of Sciences, 2016), Japan; Kumamoto, 2016 (Kiyota et al., 2017) and particularly 2010/11 Canterbury-Christchurch, New Zealand (National Academies of Sciences, 2016); has highlighted the liquefaction phenomenon and raised its public profile. In the Emilia case; despite the relatively moderate magnitude of the earthquake around 5.9 MW (Rossetto et al., 2012), the post-event survey showed costly damage to infrastructure roads, pipelines due to soil liquefaction; in old masonry and recent constructions 12,000 buildings were severely damaged (Fioravante et al., 2013). In the case of Canterbury-Christchurch, soil liquefaction affected nearly 60,000 residential buildings and the horizontal infrastructure over one third of the city area (Cubrinovski et al 2014, van Ballegooy et al, 2014). During the Great East Japan earthquake,

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[1] ePACT. https://media.epactnetwork.com/geographical-breakdown-natural-disasters-europe/

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approximately 27,000 houses, more than 2,000 levees and several ports suffered damage from the resulting ground liquefaction (Yasuda et al, 2013).

2.5 These recent experiences have clearly articulated the risks that EILD events pose to urban communities, and highlighted the need to explore the potential of mitigation actions to improve community resilience through effective disaster risk reduction. Whilst the causes of soil liquefaction and the soil conditions that make an area susceptible to this phenomenon are well known, the effectiveness of liquefaction mitigation techniques applied to increase the resistance of a particular structure or infrastructure to the impact of the EILD event and improve community are less well understood.

2.6 Whilst resistance involves designing a structure (superstructure and foundations) and improving its strength to withstand an EILD event and reduce the immediate impact of event on the built asset; resilience is about increasing the ability of the overall urban community system to cope with the impact of an EILD and quickly recover after the event. Thus, whilst resistance is primarily considered from a technical perspective; resilience requires a holistic assessment of the impact that the EILD event will have on individual (people, businesses, organisations etc.) stakeholders and the wider urban community collectively. The aim of the LIQUEFACT project is to develop a more comprehensive understanding of the factors that affect community resilience to EILD events and develop a range of tools to evaluate the potential of mitigation interventions to improve community resilience to such events.

2.7 The LIQUEFACT project has previously reviewed the effects of EILD events on the vulnerability, resilience and adaptive capacity of communities (see Deliverables D1.1) and developed a Resilience Assessment and Improvement Framework (RAIF) (Deliverable D1.3) to help stakeholders develop mitigation plans to improve community resilience to EILD event (Figure 2.1). This report introduces the concept of cost-benefit analysis to support the evaluation of alternative mitigation options to improve community resilience to earthquake induced liquefaction disaster events.

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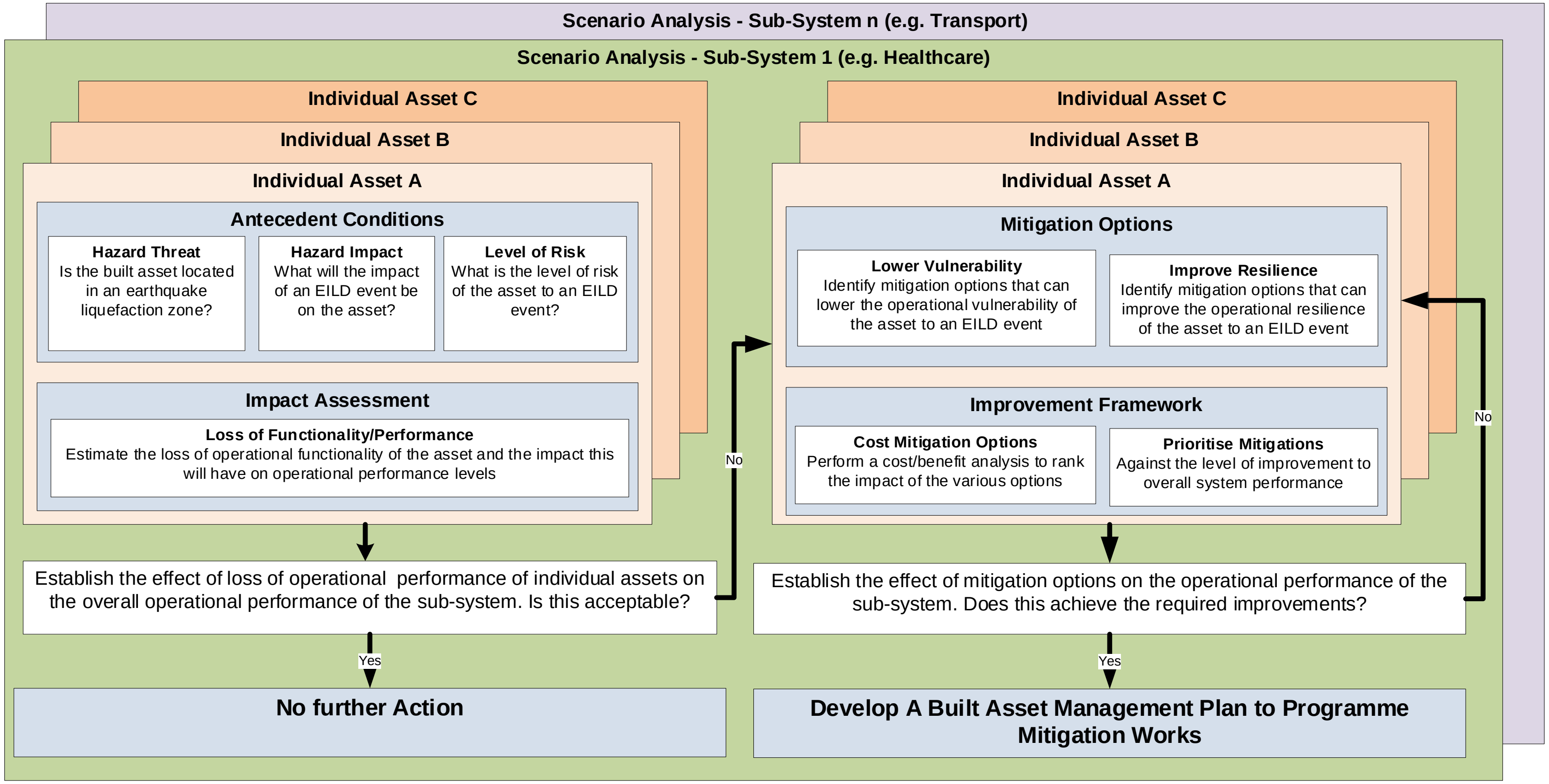

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Figure 2.1: The Resilience Assessment and Improvement Framework

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3. Introduction to Cost Benefit Analysis

3.1 Cost-benefit analysis (CBA) is a major and well-recognised option appraisal technique to compare the costs and resultant benefits of alternative development/mitigation projects. The technique is particularly useful when government or public institutions are seeking to justify significant investments to improve local infrastructures, increase security and improve the community resilience to the disasters. The basic idea of CBA is to identify the costs of undertaking development/mitigation projects and compare these to the benefits over time that could accrue from the development/mitigation projects. The benefit to cost ratio (B/C) provides a dimensionless indicator that can be used to help inform the business decision on whether development/mitigation projects should be funded now or not. Cost-benefit analysis can be applied at different scales, from assessing development options for individual stakeholders to evaluating the potential net benefit of development options across multiple stakeholder groups. In the LIQUEFACT project, CBA will be used to evaluate the economic viability of different liquefaction mitigation options on both individual built assets (individual stakeholder group) and the wider community (multiple stakeholder groups).

3.2 Cost-benefit analysis developed out of welfare economics, where the underlying concepts of consumer surplus and externality (the consequence of an economic activity on an unrelated third party) which originated from Europe in the 1840s (Mishan and Quah, 2007). When the externality is negative then the cost to society is greater than the cost of the individual stakeholder. When the externality is positive the cost to society is less than the cost to the individual stakeholder. In the latter, there is a net benefit to society (see Johansson and Kristom, 2016 for a fuller explanation). Thus, whilst CBA is in many ways similar to other investments appraisal techniques (Net Present Value, Internal Rate of Return, Return on Investment etc.), it differs in so much as CBA takes a wider perspective and aims at estimating the profit (or loss) of investment options for society (Mechler, 2005).

3.3 The CBA approach is now routinely used as an option appraisal technique across a wide range of disciplines (e.g. environmental policy, transport planning, healthcare, military projects and disaster mitigation projects) to evaluate the costs and benefits associated with alternative solutions or policies (Johansson and Kristrom, 2016). By deriving a dimensionless benefit/cost ratio for each alternative solution or policy, it is possible to rank the solutions/policies in order. Where the benefit/cost ratio exceeds one there is a net benefit of investing in the solution/policy. Further, the higher the cost benefit ratio, the greater the potential benefit the solution/policy will realise (note: the ultimate decision to invest or not in the solution/policy

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is not solely dependent upon the benefit/cost ratio but also needs to consider a range of other, non-quantifiable, criteria.).

3.4 In Europe, CBA was used as a technique to evaluate environmental interventions as early as the 19th century as part of the infrastructure appraisal process (OECD, 2006). In 1936, the US Flood Control Act made CBA a mandatory technique for flood control projects. In other countries, CBA gained popularity after World War II where the pressure for efficiency in government spending, and in particular the need to justify major public investment decisions resulted in a robust methodology to calculate net societal benefits (OECD, 2006). In the UK, CBA is now the recognised methodology provided within the Treasury's Green Book for appraisal and evaluation of public expenditure proposals and is regularly used to evaluate large-scale public investment projects.

3.5 The general structure of a CBA are summarised in Table 3.1. The first step in the CBA process is to identify the stakeholders' objectives/outcomes that are to be achieved by the development/mitigation project. Once these are established, the range of possible development/mitigation options (physical, operational, social etc.) are identified, and if possible, ranked in preference order (those options that are unacceptable to the stakeholders, are eliminated from further analysis). For the remaining options, project costs are calculated and compared against the estimated benefits. These options are then ranked according their benefit/cost ratio. Combining the results from the ranking list with an assessment of risk and the un-monetarised factors not considered in the CBA produces a final ranking order of preferred development/mitigation solutions. Whilst, this process appears straightforward, there are many uncertainties associated with assessing both the costs and benefits, and how stakeholders (and in particular key decision makers) respond to these uncertainties is critical to the confidence that they have CBA approach. Thus, any CBA must acknowledge the existence of these uncertainties and provide a clear rationale to all the assumptions made when evaluating them.

Step 1. Rationale for intervention	Identify the objectives or outcomes the policy maker wishes to meet through intervention
Step 2. A long-list of options	Consider how best to meet the stakeholder's objectives by considering a long-list of options, including a wide range of possible approaches. These should be assessed for viability and filtered down to a short-list.

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Step 3. Short-list appraisal	Expected costs and benefits are estimated and the trade-off is considered. This is done using Social Cost Benefit Analysis (CBA) or Social Cost-Effectiveness Analysis (CEA).
Step 4. Identification of the preferred option	The detailed analysis at the short-list appraisal stage to determine which option provides the best balance of costs, benefits, risks and unmonetizable factors.
Step 5. Monitoring and Evaluation	Evaluation is the systematic assessment of an intervention's design, implementation and outcomes. Both monitoring and evaluation should be considered before, during and after implementation.

Table 3.1. General cost-benefit analysis structure

3.6 The cost component of the CBA methodology is calculated by considering both the capital and operating costs associated with an intervention. In UK, these cost components are defined in the Royal Institution of Chartered Surveyors New Rule for Measurement (Royal Institution of Chartered Surveyors, 2013). "Capital costs include:

- **Facilitating works costs**: These are related to specialist works which, normally, need to be completed before any building or infrastructure works can commence (e.g. major demolition works, soil stabilisation works and or temporary diversion of mains drainage);
- **Building works costs:** This includes the cost of constructing the permanent built environmental intervention. This is often the major component of the capital cost and can be estimated using cost models such as functional unit rates (e.g.: cost per car parking space) or any other cost model such as cost per m of constructing a motorway.
- **Construction cost estimate including preliminaries and contractor's overhead and profit:** cost of preliminaries are associated with costs of ancillary works which are not included within the building works estimates. These may include but not limited to the constructors (contracted by the client) costs associated with management and staff, site establishment, temporary services, security, safety and environmental protection, control and protection, common user mechanical plant, common user temporary works, the maintenance of site records, completion and post-completion requirements, cleaning, fees and charges, sites services and insurances, bonds, guarantees and warranties. Contractors overhead costs are the costs associated with head office administration apportioned based on the scope of the development. Profits are the contractor's return on capital investment.

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- **Design and other consultation fees:** These are the costs paid to the consultants such as designers, planners, engineers for their pre-construction, construction and post-construction related services.
- **Other development costs:** costs that are not necessarily directly associated with the cost of constructing the building, but form part of the total cost of the building project to the employer (e.g. land acquisition costs, fees for letting agents, marketing).
- **Risk estimate:** the amount added to the base cost estimate for items that cannot be precisely predicted to arrive at the time of estimating.
- **Inflation estimate:** predicted an upward or downward movement in the average level of prices and or costs during the design and construction phases.
- **VAT/TAX as applicable:** Tax payable (if any)." (ibid)

3.7 Where available, capital cost analyses of previously completed projects and other published construction cost models can be used to determine the capital cost of built environment interventions. These are normally found in published price books, cost databases (such as BCIS cost data in the UK), or by access to official reports. However, these published cost data are generic and need to be adjusted to suit individual project circumstances such as inflation, location, access and storage restrictions, use of special construction methods and materials, etc. More accurate capital cost can be obtained with the help of constructors' quotations requested specific to the project circumstances.

3.8 Operating costs include:

- **Repair and refurbishment costs:** for refurbishment and major repair projects these costs are similar in nature to the capital costs outlined above. For responsive maintenance, costs should also include an estimate of the disruption that a failed item (e.g. air cooling plant) has on the performance of the organisation (e.g. loss in productivity, reputational loss etc.). For cyclical maintenance (e.g. periodic service to lifts/boilers; compliance testing etc.) and planned maintenance (actions that result from condition surveys) costs should include the management of the process as well as the costs of undertaking the actual work.
- **Utilities costs**: these are the costs associated with operating the asset (e.g. energy, water, sewerage, etc.) over the project life-cycle.
- **Disposal costs:** these are the costs associated with the disposal of the asset at the end of its useful life. Such costs can include demolition, site restoration, and waste disposal.
- **Other** costs include the provision of soft facilities management services (e.g. cleaning, security, etc.) required for the effective operation of the asset.

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3.9 It is generally accepted that the operating cost of a built asset is substantially higher compared to its capital costs. Capital cost of an asset is similar to the exposed tip of some much larger iceberg. However, the proportion of capital cost to operating cost is hard to determine and depends on the type of the asset. An early paper from the Royal Academy of Engineering (Evan et al., 1985) stated that the ratio between these as 1:5:200 (1 = construction cost; 5 = maintenance and building operating costs; 200 = business operating costs) for commercial office buildings. However, whilst this ratio is often quoted and used to inform major investment decisions, it is not supported by any fundamental research (Hughes et al., 2004). As such, bespoke capital, maintenance and operating costs need to be developed for individual development/mitigation project options.

3.10 Finally, all costs need to be discounted to current value to account for future cash flow projections. Future cash flow is discounted using a discount rate to derive present value estimates that are used to allow direct comparison between the cost of investments and the expected return on that investment over time.

3.11 The benefits associated with development/mitigation interventions are calculated by considering both tangible and intangible impacts the benefits associated with tangible market products can be directly identifiable. For example, the value of a house built under a refugee settlement programme, could be calculated using the market value of a similar house built elsewhere. However, those benefits associated with intangible impacts are more difficult to value directly. In these circumstances proxy measures may need to be used (these will be discussed in more detail later in this report). For example, the value of re-creational facilities built as part of the same refugee settlement programme could be associated with the enjoyment and social welfare gain experienced by the community and be attractive to external visitors' thus increasing economic prosperity through increased tourism. CBA uses a variety of different techniques to measure and quantify tangible and intangible benefits.

3.12 In their latest version, the International Valuation Standards Council (IVSC) (2016) identifies three main approaches to estimate the value of tangible products: the market approach; the income approach; or the cost approach.

- **The market approach:** This approach provides an indication of value by comparing products with identical or comparable (that is similar) products for which price information is available. According to the IVSC (2016) "the market approach should

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be used as the primary basis for a valuation under the following circumstances: (a) the asset has recently been sold in a transaction appropriate for consideration under the basis of value, (b) the asset or substantially similar assets are actively publicly traded, and (c) there are frequent or recent observable transactions in substantially similar assets."

- **The income approach:** This approach estimates the value of a product by reference to the value of income, cash flow or cost savings generated by the product. According to the IVSC (2016) "the income approach should be used as the primary basis for a valuation under the following circumstances: (a) the income-producing ability of the asset is the critical element affecting value from a market participant perspective, and (b) reliable projections of the amount and timing of future income are available for the subject asset, but there are few, if any, relevant market comparable."
- **Cost Approach:** The approach provides an indication of value by calculating the current replacement or reproduction cost of an asset and making deductions for physical deterioration and all other relevant forms of obsolescence. According to the IVSC (2016) "the cost approach should be used as the primary basis for a valuation under the following circumstances: (a) market participants would be able to recreate an asset with substantially the same utility as the subject asset, without regulatory or legal restrictions, and the asset could be recreated quickly enough that a market participant would not be willing to pay a significant premium for the ability to use the subject asset immediately, (b) the asset is not income-generating (directly or indirectly) and the unique nature of the asset makes using an income approach or market approach unfeasible, and (c) the basis of value being used is fundamentally based on replacement cost, such as reinstatement value."

3.13 There are a range of widely used approaches to estimate the economic value of nonmarket or intangible impacts. These approaches fall under three main categories.

- **Revealed preference approaches:** Revealed preference methods are benefit quantification methods developed by economists to indirectly quantify the value of non-market products using market information and behaviour to infer the economic value of an associated non-market impact (OECD, 2006). The value is derived based on actual behaviour and utilizes complementary and substitutive relationships between public and various marketed goods to infer the value attributed to public goods from market transactions in private goods (Frey et al., 2004). Revealed preference methods are used to assess non-tangible products, such as (a) services sold on the market; (b) services or products used for the production of a product or

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service sold on the market (this includes travels); (c) interventions that change values of products or services sold on the market; (d) interventions to prevent negative impact to products or services sold on the market.

- **Stated preference approaches:** Stated preference techniques of valuation use specially constructed questionnaires to elicit estimates of people's Willingness to Pay (WTP) for or Willingness to Accept (WTA) a particular outcome (Fujiwara and Campbell, 2011 – HM Green book), or to offer people choices between "bundles" of attributes from which analysts can infer society's WTP or WTA (OECD, 2006). Stated preference methods are used to estimate (a) compensation values to be paid to increase risk; (b) comparison in terms of benefits between services or intervention.

- **Subjective Well-Being approach/The Life Satisfaction approach:** These recently developed approaches attempt to measure people's experiences rather than their preferences and they use direct measures of well-being, such as life satisfaction, rather than the degree to which one's preferences have been satisfied, to better estimate the usefulness of an intervention to the individual (Fujiwara and Campbell, 2011 – HM Green book). Subjective Well-being approach is used to give a value to some condition of public well-being.

3.14 Whilst in many cases the costs associated with development/mitigation options appear easier to estimate than the benefits of such interventions care must be taken to avoid, or at least minimise, optimism bias and risk. Optimism bias (the proven tendency for appraisers to be too optimistic about key project parameters) and risk perception (uncertainties that arise in the design, planning and implementation of an intervention) are known to have a significant impact on cost estimates (HM Treasury Green book, 2018). If unaccounted for this may lead to projects or interventions being halted due to lack of funding, or not realising initially expected benefits or other returns. As such, it is imperative that CBA includes a sensitivity analysis, which shows the degree to which the benefit/cost ratio is susceptible to changes in the values associated to the input variables. It is also suggested that when interventions have significant direct effects on markets (that are subjected to the interventions), compliance costs should be estimated using general equilibrium analysis which captures linkages between markets across the entire economy (U.S. Environmental Protection Agency, 2010). Ideally, compliance costs would be estimated using general equilibrium analysis.

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4. Cost Benefit Analysis in Disaster Mitigation

4.1 The generic approach to CBA described in section 3 has been adapted to reflect specific scenario circumstances (e.g. different disaster, levels of uncertainty, return period, quality of data sources etc.) to assess the efficiency and benefits of mitigation interventions that seek to reduce disaster impacts. Figure 4.1 presents the five basic steps of CBA for disaster mitigation according to Smyth et al. (2004). However, as with the generic approach to CBA there are a number of practical issues over the quantification of costs and benefits that have to be addressed if the technique is to be successfully applied to disaster scenarios. Again, as with generic CBA these primarily concern the quantification of tangible and intangible impacts. In disaster mitigation CBA the costs represent the expenditure needed to retrofit or refurbish an asset, whilst, the benefits are related to avoided damages (to assets and people) due to improved performance of retrofitted assets. The cost of retrofitting assets should be compared with the future benefits, quantified in terms of equivalent annualized values and discounted to present-day values that could be realised in the future, if a disaster occurs. However, uncertainty in the timing, location, and severity of future disasters complicate the loss assessment methodology and the calculation of future benefits have to be taken into account when considering applying CBA to disaster mitigation scenarios. Also, reconstruction costs have to include a multiplier to account for increased costs associated with a surge in demand for construction services immediately following a disaster event (Ruddock et al, 2010).

4.2 There is a substantial amount of recorded disaster loss data worldwide. More than 60 disaster loss and damage databases are known to exist at national and regional levels (UNDP, 2013). In addition, EM-DAT, NatCatSER-VICE, Desinventar and Sigma contains disaster loss data at a global level (Fakhruddin et al., 2017). However, these databases are not without limitations. Most of the databases have developed their bespoke protocols for collecting and recording data and this, combined with much missing data, and inconsistent economic valuations of physical damages and losses (Fakhruddin et al., 2017) limits their interoperability and usefulness as a comparative tool. In an effort to develop operational tools to translate the Sendai Framework for Disaster Risk Reduction (UNISDR, 2015) into practice, the Joint Research Centre of the European commission developed 'the guidance for Recording and Sharing Disaster Damage and Loss Data' (EU expert working group on disaster damage and loss data, 2015). It is expected that standardising the data collection process should make it possible to evaluate the effectiveness of policies and to determine their impact on loss trends (De Groeve et al., 2014). The guidance identified that losses should be recorded against four key types of 'affected elements': Social, Economic, Environmental and Heritage (Figure 4.2).

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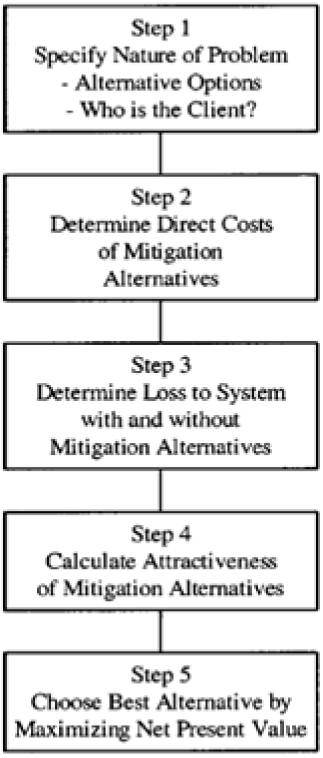

Figure 4.1 Steps of CBA for disaster mitigation. Source: Smith et al. (2004).

Mechler (2005) developed a CBA approach for natural disaster risk management in developing countries based on measure of tangible and intangible economic impacts of disaster mitigation strategies. Mechler (ibid) used a B/C ratio to estimate the positive benefits of mitigation interventions for different kinds of disasters, calculating ratios ranging from 2.2 to circa 100 across a range of disaster scenarios. Whilst the range of B/C ratios was very wide, what was clear from Mechler's analysis is that the CBA methodology could be successfully used as a major decision support tool for estimating the efficiency of disaster mitigation projects across a range of disaster scenarios. The CBA approach was validated by the scholar using exemplary case studies (i.e. flood protection measure during El Nino events in Piura, Peru; an integrated water management and flood protection scheme in Semarang, Indonesia).

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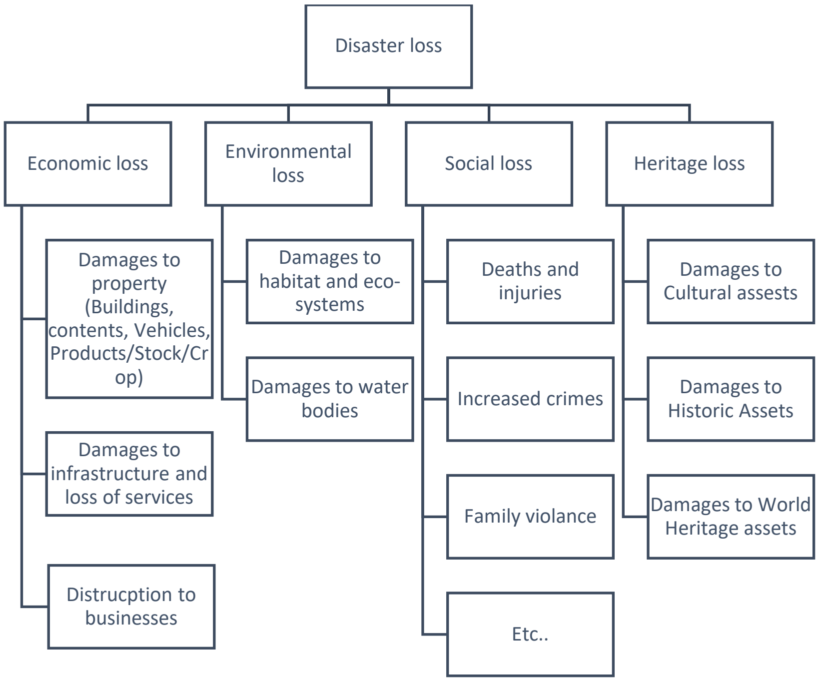

Figure 4.2 Summary of loss categorisation provided in 'The guidance for Recording and Sharing Disaster Damage and Loss Data' (EU expert working group on disaster damage and loss data, 2015).

4.3 There are many different approaches to developing CBA models for disaster mitigation. Kull et al. (2013) developed a quantitative, stochastic CBA framework for flood and draught risk reduction in India and Pakistan. The notable feature of Krull's framework is that it also takes account future climate change impacts. The potential impact of climatic change were incorporated using a statistical down-scaling model for the flood analysis developed by Opitz-Stapleton and Gangopadhya (2011). Figure 4.3 presents the cost-benefit framework developed using the drought risk management in Rohini Basin, India, as the basis of Krull's study is Mechler CBA approach for risk disaster management.

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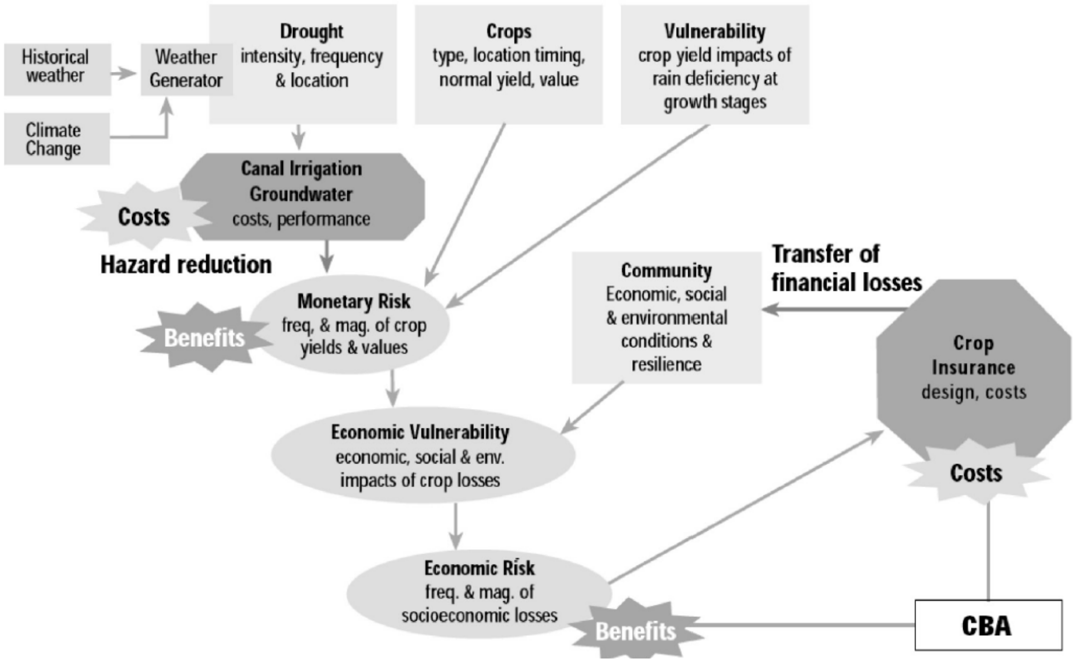

Figure 4.3 Framework for forwards-looking risk and cost–benefit analysis for drought risk management in the Rohini Basin, India. Source: Kull et al, 2013 based on Mechler et al. (2008).

4.4 Jonkman et al. (2004) investigated the applicability of CBA for the assessment of flood protection strategies in the Netherlands and concluded that CBA can be a useful tool in decision-making. They built their approach on an economic optimisation, i.e. the minimization of the total costs (C_{tot}) obtained as the sum of expenditure for a safer system (I) and the expected value of economic damage ($E(D)$) –:

$$\min(C_{tot}) = \min(I + E(D)) \tag{1}$$

4.5 The expected value of the economic damage $E(D)$ is calculated from considering the probability of flooding (P_f), and the damage caused by the flood (D) discounted using the so-called reduced interest rate (r') as the discounting factor:

$$E(D) = P_f \times D/r' \tag{2}$$

Where $r' = r - g$ takes into account both the interest rate (r) and the economic growth rate (g) and both I and D depends on the probability of occurrence of floods.

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Whilst this approach has the benefit of simplicity, it does have its limitations (Eijgenraam, 2003). For example, the method is not adaptive: it does not allow for adjustments to the planning strategies of mitigation interventions and economic growth (Jonkman et al., 2004).

4.6 As a consequence, the criterion measuring the cost effectiveness of a mitigation intervention depend on the ratio between investment and risk reduction. From a decision-making perspective, this means that the strategy that can offer the highest level of protection from a natural disaster (smaller optimal failure probability at a lowest cost) is worth investing in. However, while in theory this framework seems to be very straightforward, rational and easy to use, it can be much more challenging to implement it in practice, taking into account that a plethora of indirect impacts and factors are not considered by this approach, such as ecological, social, psychological and political aspects.

4.7 Shreve and Kelman (2014) reviewed individual CBA disaster mitigation case studies across various geographies, hazard types and vulnerabilities. Whilst the authors provided supporting evidence on the economic effectiveness of CBA in Disaster Risk Reduction (DRR), they also highlighted numerous limitations of the existing approaches to disaster management. Amongst these limitations were the need for sensitivity analyses as a fundamental part of the CBA process, the absence of meta-analysis of the academic literature in this field providing an overview of the existing theoretical and empirical findings, and a narrative approach to defining vulnerability and the disadvantages of DRR measures (Shreve and Kelman, 2014).

4.8 White and Rorick (2010) performed a CBA of a DRR project undertaken in Nepal to help local communities address the impacts of annual severe flooding. For the purpose, the authors developed a quantitative methodology for assessing the cost effectiveness of community-based DRR projects. However, whilst their methodology can be applied to other DRR projects in various cultural and economic settings and environmental contexts it does have one significant weakness: it considers only direct impacts of disasters. The reason behind this choice is the difficulty in quantifying the indirect impacts. Among the direct costs the direct losses of personal assets (including the annual crop, grain storage, livestock, belongings, etc.), and community assets, such as infrastructure were included. Thus, the result expressed in terms of Benefit/Cost ratio under-estimate the true benefits, because the indirect benefits identified by key stakeholders were ignored.

4.9 Another notable study was proposed by the National Institute of Standards and Technology (NIST) in 2013. It investigated the cost premiums associated with earthquake-resistant building construction in the middle Mississippi River Valley region, and the benefits expected from instituting modern building code provisions for seismic safety (National Institute of Standards and Technology, 2013). Cost premiums and relative benefits were developed by comparing design requirements in national model codes and current local codes both with

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and without seismic requirements. Similarly to White and Rorick (2010), this analysis was not able to provide a framework for quantification of the indirect losses and benefits, which seems to be a common limitations of the CBA for disaster management.

4.10 Another attempt to analyse the potential cost-effectiveness of risk management across a range of disasters and community intervention programs was proposed by Wethli (2014).

4.11 The study showed a significant variation in B/C ratios across a range of countries (the range of values in each area is very high) which can be explained by differences in hazardous events occurrence and the different costs of implementation of risk management interventions in each area. The study demonstrated that in some categories, e.g. earthquakes, floods, the minimum value of B/C ratios is very low, which make risk interventions very costly in areas where the probability of earthquakes and flood is low. However, overall the high median and mean values of benefit-cost ratio, indicates that risk management appears generally cost-effective across most of the disaster situations examined. This finding is further reinforced in Table 4.1, which summarises the results for earthquakes and flooding of the papers reviewed by Wethli (2014) and Mechler (2005). Looking at the values of B/C proposed in this table, it is clear that for earthquake is often higher than the benefit of mitigation actions; whereas for flood disasters the cost results much lower than the benefit in most of the cases. This implicates that the decision makers could prefer not to invest in mitigation actions for earthquake disasters. This low value of the BC could be due to the long return time of earthquake disasters.

Source	**Benefit-costs ratio**
Earthquakes	
Ghesquiere, Jamin and Mahul (2006)	Earthquake vulnerability reduction project in Colombia: BC ratio 1.6-2.5
Kunreuther and Michel Kerjan (2012)	Proposal to retro-fit schools to withstand damage from earthquakes (cross-country analysis): BC ratio 0.01-6.45
MMC (2005)	Average BC ratio across FEMA earthquake mitigation grants: 2.5
UN and World Bank (2010)	Case study on retrofitting homes to prevent damage from earthquakes in Istanbul: BC ratio 4.6
Flooding	

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Burton and Venton (2009)	Case studies on measures to reduce damage from flooding in the Philippines: BC ratio 0.7 (dykes), 4.9 (sea wall), 24 (footbridge)
Kull, Mechler and Hochrainer-Stigler (2013)	Structural measures to reduce flooding: Pakistan: BC ratio 1.3 (floodplain relocation), 1.9 (expressway), 9.3 (retention pond), 8.6 (river improvement), 25 (combined pond and river improvement) India: BC ratio 1.8 (embankment maintenance)
Kunreuther and Michel-Kerjan (2012)	Proposal to reduce damage from flooding (cross-country analysis): BC ratio 14.5 (elevating houses), 60.1 (community wall)
MMC (2005)	Average BC ratio across FEMA flood mitigation grants: 5.1
Woodruff (2008)	Measures to reduce flooding in Samoa: Floodwalls: BC ratio 0.11-0.64 Diversion channel: BC ratio 0.01-0.09 Flood proofing buildings by increasing floor height: BC ratio 0.53-8.07 (existing homes), 2.22-44.38 (new homes)
UN and World Bank 2010	Case study on flood-proofing a house: BC ratio 3.7 (Jakarta), 5.7 (India)
Dedeurwaerdere (1998)	Appraisal of different prevention measures against floods and lahars in the Philippines: BC ratio 3.5 – 30
Mechler (2004)	Prefeasibility appraisal of Polder system against flooding in Piura, Peru: BC ratio 3.8
Venton & Venton (2004)	Ex-post evaluations of implemented combined disaster mitigation and preparedness program in Bihar, India and Andhra Pradesh, India. Bihar: B/C ratio 3.76 Andhra Pradesh: B/C ratio 13.38

Table 4.1. Benefit-costs ratio of measures to limit damages from earthquakes and flooding.

Source: Wethli, (2014) and Mechler (2005).

4.12 In disaster mitigation, CBA is predominantly used to organise, appraise, and present the costs, benefits, and inherent trade-offs for mitigation investment projects seek to increase public

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welfare (Kopp et al., 1997; Kull and Melcher, 2013). CBA for disaster mitigation can also be used at various stages of the project life cycle, as a tool for options appraisal and prioritising mitigation actions and as a disaster mitigation planning tool to ensure robust mitigation solutions are identified and delivered (Mechler, 2005; Benson and Twigg, 2004). Again, these activities are compatible with the RAIF, developed earlier in the LIQUEFACT project (see Deliverables 1.3 and 1.4) to support the business case for soil liquefaction mitigation interventions. The question is whether investments in disaster prevention projects will provide the positive returns in avoided or reduced negative disaster impacts. According to limited evidence available in the literature: for every dollar invested we can expect to receive from two to four dollars in avoided losses (Mechler, 2005; World Bank and United Nations, 2010; Kull, 2013). However, Woo (2013) suggests that in earthquake disasters a wider range of ratio should be considered. This scholar suggests that five bands, <1, between 1 and 2; between 2 and 5; between 5 and 10; and above 10 should be used to support practical decision-making. This is in contradiction with the conclusions that were drawn from the table proposed by Melcher (2005) and Kull (2013).

4.13 White and Rorick (2010) discuss three theoretical approaches to community-based CBAs based on the main principle of comparison of the impact of disasters with and without DRR interventions. The first approach is a hypothetical approach. Under this approach either backward-looking or forward-looking methods can be used to assess the cost and benefits of DRR mitigation actions. The former suggests a comparison between the impact of a given disaster in a community with DRR mitigations and the hypothetical one without DDR mitigations. While the latter suggests a comparison of the realized impacts in a community without DRR mitigation to the hypothetical impacts with DRR mitigations (White and Rorick, 2010). Even though in many cases the hypothetical losses can be readily assessed, this approach relies on inferences of impact, which might be not accurate in dynamic, rapidly changing environments. The second approach is the comparative approach. Under this approach the impact of DRR mitigations are compared in two different communities stricken by disasters of the same magnitude. However, in practice, the magnitude of disasters vary across communities, and it is unrealistic to assume that this magnitude is the same; therefore, the 'effective disaster magnitude index', which is a relative measure of magnitude, is used for the comparison. Another limitation of this approach is the availability (or lack) of data. There is no guarantee that comparable datasets can be obtained for both communities to compare the impacts of DRR mitigation interventions. Finally, the third approach is the before-and-after approach. It suggests a comparison of the impact data from the same community for similar disasters occurring before and after a DRR mitigation programme. However, similar to comparative approach, data availability may become an issue. However, the more serious limitation of this approach would be the differences in methodologies of deriving, collecting

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and assessing the impacts of previous and current disaster events. This approach assumes that nothing has been changed in the community apart from DRR measures between events, and impacts on the community before and after are directly comparable. This assumption is unrealistic, given that economic, social and political factors are constantly changing the environment. Although there are serious limitations of this approach, it still has potential, if the best practices of collecting data on the impacts of disasters on communities have been adopted in the area, and a comprehensive database of the key variables has been created over time.

4.14 Thus, whilst there are concerns about applying CBA to disasters because it is not possible to quantify some benefits and the data collections are not always consistent, comparable and complete; they are the only way to compare mitigation plans against expected benefits in order to make decisions. As such, its application in the LIQUEFACT project would seem justified and in line with international best practice.

5. CBA applied to Earthquake Events

5.1 Cost-benefit analysis has been used to assess the effectiveness of mitigation interventions to reduce earthquake associated losses at both the individual building/assets and city/regional scale.

5.2 At the building/asset level Goda et al (2010) used CBA to investigate the efficiency of different types of seismic isolators to mitigate seismic risk applied to two identical buildings located in Vancouver against to their cost. Those scholars developed a simplified approach to assess the increased costs associated with potential seismic damage scenarios that could occur over the service life of the buildings. Their CBA model considered both the initial construction cost (both structural and non-structural) and the repair/re-construction costs associated with post event damage but did not include mortality or morbidity costs and as such represented only the tangible costs of earthquake events. By comparing the normalised building lifecycle costs with and without seismic isolators across a range of potential seismic scenarios Goda et al. (2010) concluded that expected lifecycle costs for buildings with seismic isolation would be reduced by 20%.

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5.3 Kappos and Dimitrakopoulo (2008) applied CBA to the assessment of the economic feasibility of retrofitting a portfolio of domestic buildings in the city of Thessaloniki. The CBA model used a series of hazard curves based on probabilistic models and vulnerability analyses resulting in fragility curves to examine the cost effectiveness of retrofitting actions to the urban pre-1959 reinforced concrete designed housing. The CBA model used local and international datasets to assess replacement and retrofit costs for the range of building typologies being investigated with the building damage being calculated as the product of the replacement cost times the area of the building times the mean damage factor derived from a damage probability matrix that describes the vulnerability of the building. In addition to the physical cost of damage to the buildings the CBA model also considered indirect losses including human fatality. When considering the reduced structural vulnerability available from a range of retrofit mitigation actions, the authors assumed (ibid) that all mitigation options would move the building from a "low-code" specification toward a "high-code" specification (0 – no retrofit; 1 – high code retrofit). The costs associated with human fatalities were calculated using the approach proposed by Coburn and Spence (2002, cited in Kappos and Dimitrakopouilos) to calculate the total number of casualties and by using a life reference value of €500,000 as an upper bound. Based on their portfolio wide analysis, Kappos and Dimitrakopoulo (ibid) found no economic arguments to justify the retrofitting of the building stock on purely physical grounds. However, they did not considered historical heritage buildings and factories treating or producing polluting substances. Across all building typologies, Kappos and Dimitrakopouilos (2010) found that the benefit to cost ratios were significantly lower than 1.0. However, when the impact of human losses were included in the calculation, two building typologies (low-rise and high-rise infill frame buildings) did have a benefit cost ratio greater than one. The significance of the impact that assumptions around human losses and the value assigned to a human life should not be underestimated as these dramatically change the benefit cost ratio in favour of retrofit mitigations.

5.4 Padgett et al. (2010) developed the risk-based seismic life-cycle CBA to evaluate alternative retrofit mitigations to non-seismically designed bridges as part of a seismic upgrade programme. Padgett et al's[2] approach uses probabilistic seismic hazard models combined with fragility curves of the as-built and retrofitted bridges across a range of damage scenarios and retrofit options to compare the expected costs of damage before and after a retrofit program. That CBA model considered the cost and benefits over the service life of the bridges (an assumption of 50 remaining year's life was used for all bridges) but did not include the

[2] Padgett, J. E., Dennemann, K. and Ghosh, J. (2010) 'Risk-based seismic life-cycle cost-benefit (LCC-B) analysis for bridge retrofit assessment', Journal of Structural Safety, vol 32, pp 165-173. Doi:10.1016/j.strusafe.2009.06.003

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costs of ongoing maintenance during the remaining service life period. The CBA model also assumed that the repair costs associated with each damage state could be represented as a fraction of the replacement cost, which in turn were based upon regional estimates for bridge construction costs. Also, the loss estimates used in the CBA model only included direct losses due to structural damage, with a an escalator of 13 (derived from literature) been used to account for indirect (intangible) losses. Finally, the cost of retrofit mitigations were derived for illustrative purposes only and the authors (ibid) noted that their results were highly sensitive to these retrofit mitigation cost assumptions. As such the authors recommended that they be refined if specific bridge applications are considered. This said, the authors concluded that their risk-based life-cycle cost-benefit model could be used to distinguish between different retrofit mitigation options and therefore could be used support effective decision-making; particularly moving the business discussion beyond a focus on upfront investment to the consideration of the avoidance of life-time expected losses.

5.5 Smyth et al. 2004 explained how to use CBA to evaluate the benefits of seismic retrofitting measures to avoid earthquake related damages to a concrete structure located in a suburb of Istanbul. Smyth et al (ibid) compared: The status quo, a braced retrofitted version of the structure, a partial shear wall retrofitted version of the structure, and a full shear wall retrofitted version of the structure. Smyth et al (ibid) used structural modelling software (SAP2000) to perform a probabilistic estimate of damages for all four types of mitigation measures at a given PGA (Peak Ground Acceleration). These were then used to calculate direct losses associated with damages to the buildings, losses due to loss of lives and fatalities over 50 years. Smyth et al (ibid) used a social discount rate within his analysis and used sensitivity analyses for the cost parameters used to value human life and cost of fatalities.

5.6 Whilst the previous examples have all involved the application of CBA in assessing the cost effectiveness of mitigation interventions to reduce failure at the structural serviceability and ultimate limit state of buildings/assets, in many modern 'intelligent' buildings (increasingly used by the healthcare, emergency responders, broadcast media, civic authorities etc.); consideration also needs to be given to reducing failure at the functional serviceability limit state, where lack of function or business performance can have a significant impact on total loss assessments Kanda and Shah (1997). However, whilst Kanda and Shah (ibid) identified the need to consider service level criteria focus was on the physical/technical performance of a building/asset at the service level rather than on the actual impact that this level of technical performance has on business performance (e.g. on the ability of the hospital to function in the aftermath of an earthquake event). This is an inherent weakness in the CBA approach. It is clear that estimate of building performance includes factors related to the business management, besides those structural; hence, a CBA for built assets is difficult to apply to categories of buildings, but needs to be customized to properly include peculiar aspects of the business related functionality of the asset.

5.7 Whilst each of the approaches described above adopts a different perspective to CBA modelling they all follow the basic approach:

- Identified the specific disaster impact being addressed and identify the specific improvements in performance (normally physical performance but could also include functional performance) required of any retrofit mitigation options.
- Identify the range of possible mitigation options that could be retrofitted to the building/asset to achieve the required improvements in performance.
- Evaluate the effectiveness of each mitigation option to achieve the desired improvements across a range of disaster impact scenarios.
- Develop a CBA model that relates the total damage costs (primarily tangible costs but should also include intangible costs where applicable) expected as a result of a future earthquake over the remaining service life of the building/asset.
- Compare the total damage costs with and without mitigation: if the former is less than the latter then the mitigation action is deemed cost-effective.

5.8 As before stated, this approach is widely used; however, it does have some significant weaknesses. Shreve and Kelman (2014) reviewed the application of CBA across a range of case studies of disaster risk reduction, including earthquake disasters. As result they identified a number of weaknesses including the lack of a standard approach to developing CBA models, the omission of sensitivity analyses in many of the models, and an evaluation of the duration of the potential benefits. In addition Shreve and Kelman (ibid) questioned the ability of CBA to address vulnerability, which they argued is the root cause of disasters, rather than natural or environmental hazards. Whilst CBA does generalise vulnerability into 4 broad categories (economic, environmental, physical, and social) the need for CBA to express the categories in numeric terms tends to favour the quantitative metrics (economic and physical) over the qualitative ones (environmental and social) (ibid) is a weakness. Whilst this weakness could be addressed by adopting a multi-criteria analysis approach, utilising democratic voting and expert opinion as part of the CBA model, such approaches are not yet generally used. As such, whilst CBA continues to rely on probabilistic models supported by fragility curves and loss assessment methodologies, its usefulness will be limited.

5.9 Even given the concerns of Shreve and Kelman (2014) quantitative loss assessment methodologies lie at the heart of CBA models. There are three general approaches to assessing losses associated with earthquake disasters: detailed loss assessments of actual losses carried

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out following an earthquake event; rapid loss assessment models; and probabilistic loss assessment models that can be used when developing CBA models.

5.10 There are a range of international schemes that gather data on losses and observed damage following an earthquake event and databases in which they are collected including:

- The Cambridge Earthquake Impacts Database (CEQID) which contains earthquake damage data assembled since the 1960s. The data contains damage to buildings and total recorded casualties (deaths, seriously and moderately injured) compiled from over 600 surveys following 51 separate earthquakes.

- GEM Earthquake Consequences Database is an open source database containing data related to:
    - Ground shaking damage to standard buildings (67 events);
    - Human casualty studies and statistics (26 events);
    - Ground shaking consequences on non-standard buildings, critical facilities, important infrastructure and lifelines (22 events);
    - Consequences due to secondary, induced hazards (landslides, liquefaction, tsunami and fire following) to all types of inventory classes (24 events, 13 of which are related to landslides); and
    - Socio-economic consequence and recovery data (18 events).

- The global CATDAT database is a privately owned database developed by James Daniell. The database contains data for damages from earthquakes and their secondary effects (tsunami, fire, landslides, liquefaction and fault rupture). The database includes seismological information, building damage, a range of social losses (deaths, injuries, homeless, and affected), and economic losses (direct, indirect, aid, and insured).
- The photographic dataset as been compiled by Earthquake Engineering Research Institute (EERI) and The Earthquake Engineering Field Investigation Team (EEFIT) from 6 earthquake events since 2008. That dataset has the ability to link geo-located photos and other images with MS Virtual earth maps to provide an online tool for viewing damage and other earthquake effects from a particular event.
- The Emergency Events Database (EM DAT) has been developed by the Centre of Research on Epidemiology of Disasters (CRED), Université Catholique de Louvain. This is a freely accessible database containing data on occurrences and effects, including economic effects, of over 18 000 mass disasters since 1990.
- The NatCatSERVICE is a global database of natural catastrophe data maintained by Munich Re (a global reinsurance company), which systematically records information

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on natural catastrophe loss events (including direct economic losses, insured losses and human losses) from around the world.
- The Sigma database is a proprietary database maintained by SwissRe (a global reinsurance company) that provides estimates of indirect losses.

5.11 In addition to databases described above, there are several rapid loss assessment and probabilistic models that can be used to assess earthquake losses, including:

- The HAZUS-MH Earthquake Model (HAZUS-MH 2003) and HAZUS MH software tool was developed by the United States Federal Emergency Management Agency (FEMA) to provide a nationally applicable methodology for earthquake loss estimates of damages and loss to buildings, essential facilities, transportation and utility lifelines, and the population.
- SELENA (Seismic Loss Estimation using a Logic Tree Approach) uses a similar methodology to HAZUS-MH, but it works independently of any Geographic Information System and allows the user to define weighted input parameters to account for epistemic uncertainties in all types of input data. SELENA is also available as a software tool.
- OpenQuake is a web-based risk assessment platform that offers an integrated environment for modelling, viewing, exploring and managing earthquake risk (Silva et al. 2013). The OpenQuake platform currently has five main calculators (Scenario Risk, Scenario Damage Assessment, Probabilistic Event Based Risk, Classical PSHA-based Risk and Benefit-Cost Ratio).
- PAGER (Prompt Assessment of Global Earthquakes for Response) provides fatality and economic loss impact estimates following significant earthquakes. The PAGER service is operated by the US Geological Survey and provides a wide range of tools on earthquake impact and loss assessment.

5.12 However, even the above data sources and models do not contain support the development of CBA models to assess the EILD events required in the LIQUEFACT project. As such, LIQUEFACT has developed its own CBA approach as part of the LIQUEFACT Resilience Assessment and Improvement Framework.

6. Cost-Benefit Analysis applied to EILD Events

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6.1 Earthquake-induced soil liquefaction occurs when soil strength and stiffness decrease as a consequence of an increase in pore water pressure in saturated cohesionless materials during, and following, seismic ground motion as a result of the applied stress; hence causing the soil to behave like a liquid. Earthquake-induced soil liquefaction phenomena result from the interaction of soil particles and porewater under the shear stress and shear strain reversals induced by earthquake shaking (National Academy of Sciences 2016). To the best of the authors knowledge CBA has never been used in liquefaction risk management. However, it must be recognized that Bird et al. (2004 a), Bird et al. (2004 b) and Bird and Bommer's (2004) works are an attempt to define cost assessment methods for earthquake-induced soil liquefaction disasters has been carried out. In this section a CBA methodology is developed to evaluate liquefaction risk management strategies at the community level and at single structure or infrastructure level.

6.2 Unlike disaster events that affect a wide geographical area, EILD event impacts are much localised, affecting individual sites and/or assets. As such, whilst the authors believe it is appropriate to apply the general principles of CBA to evaluating the potential impacts of liquefaction mitigation options to EILD events on the resilience of individual asset and the wider community, the processes of assessing hazard, exposure, vulnerability and risk will be different to those used for wide area impact events. The approach outlined in the LIQUEFACT RAIF (Deliverable D1.3) (Figure 2.1) envisages CBA being applied at two levels. Firstly, CBA will be used as part of the options appraisal process to identify the most appropriate liquefaction mitigation option at an individual asset (or collection of assets) at the site level. In this process the cost of the mitigation option will be set against the perceived benefit to the asset owner/operator in terms of avoiding the costs (both direct and indirect) associated with loss of performance or failure (full and/or partial loss of performance over time) of the asset following an EILD event. Secondly, the CBA for those individual assets within a region that are critical to support community resilience to an EILD event (see LIQUEFACT Deliverables 5.1 and 5.2) will be aggregated to provide an assessment of the overall CBA for the region of the mitigation interventions applied to the individual assets. Whilst the development of the full business model to support the assessment and implementation of EILD mitigation measures will be developed in LIQUEFACT Deliverable 5.4, the theoretical approach to evaluating CBA at the individual asset and community level is outlined here.

6.3 As discussed previously, there are both benefits and limitations of applying CBA to DRR scenarios.

- Benefits of CBA in DRR:
  - CBA can be instrumental in evaluating different alternative mitigation measures (Venton, 2010).

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  - CBA can be extremely valuable in enabling discussion between communities and stakeholder to think through the costs and benefits of different mitigation options, (Venton, 2010).
  - CBA works well for projects where the benefits can be identified in monetary terms as in the case of hard infrastructures (Mechler, 2016).
- Limitations of CBA in DRR:
  - Difficulty of accounting for non-market values (Mechler, 2005).
  - Difficulty in discounting benefits and costs (Mechler, 2005).
  - Difficulty in including disaster risk characteristics, such as low probability and high consequences of the events and shortness of planning horizons in administrations (Mechler, 2005).
  - Lack of consideration of environmental issues (Hanley, Spash 1993).
  - Lack of consideration on distributional impact: who are the beneficiaries and who are the payees? (Mechler, 2005; Kull 2008).
  - Ethical issues on associating a monetary value to life (May 1982, Mechler 2005).
  - Inability to quantify social and environmental benefits (Shreve and Kelman, 2014).
  - It is more difficult to apply in case of softer and more systematic interventions to define economic efficiency (Mechler, 2016).

6.4 To address the above the RAIF proposes the use of CBA at three stages of the built asset life-cycle:

- Project appraisal: as part of the formal evaluation of alternative liquefaction mitigation options by identifying and ranking the most efficient measures among alternatives.
- Post project evaluation: by measuring the actual costs and benefits of actual liquefaction mitigation options after completion (through data following an actual EILD event).
- Informational study as part of strategic built asset management plans: by providing a broad overview over costs and benefits to support a wider strategic debate of liquefaction mitigation options amongst stakeholders.

6.5 The main application of CBA to the evaluation of liquefaction mitigation options outlined in the RAIF follows the same generic four stage approach (Figure 6.1) as that suggested by Mechler et al. (2014).

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- Stage 1: Estimate the risk in the antecedent condition without soil liquefaction risk management strategies being implemented. This requires estimating and combining liquefaction hazard, exposure and vulnerability.
- Stage 2: Identify possible soil liquefaction risk reduction/mitigation measures and their associated costs, which, in case of hard infrastructure projects, consist of design, construction and maintenance.
- Stage 3: Analyse the risk reduction associated with each mitigation option: estimate the benefits of reducing liquefaction risk.
- Stage 4: Calculate the economic efficiency of the measures. A measure can be defined economically efficient if the benefits exceed costs.

6.6 The type of assessment to be conducted depends upon the objectives of the respective CBA as well as the data sources available to assess hazard, vulnerability (and then fragility), exposure and impacts. In order to operationalise the assessment of hazard, exposure, vulnerability, risk and risk reduction and considering data and resource limitations for conducting CBAs, two of Meckler's frameworks for quantitative analysis (Table 6.1) would appear to be the most applicable to evaluating liquefaction mitigation options. The forward-looking CBA framework (risk-based approach) combines data on hazard and vulnerability to assess antecedent risk and reduced risk after mitigation. Whilst this approach is mathematically rigorous, its application can be problematic in situations where data and resources available to undertake the assessment are limited. The approach is also less applicable to areas that are subject to multiple hazards or characterized by a large number of individual assets that have different vulnerabilities. In these situations it may be more pragmatic to use a Meckler's backward-looking framework (impact based-approach) where past damage to assets is used to assess the risks associated with the disaster event and quantify potential future damage states that history suggests would exist should such an event occur again. Both of these approaches are compatible with the six stage RAIF developed as part of the LIQUEFACT project.

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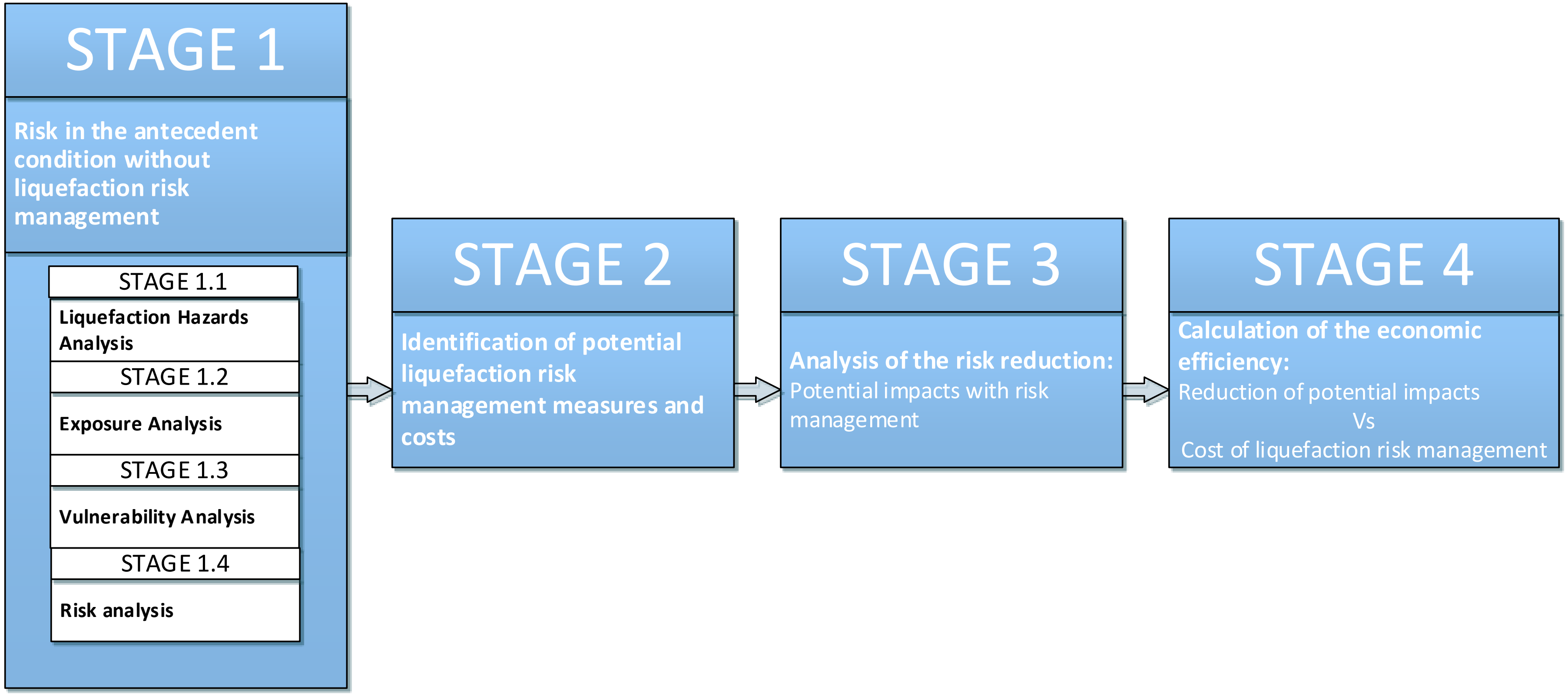

Figure 6.1: CBA for liquefaction risk management: adapted from (Mechler, Czajkowski et al. 2014)

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Type of Assessment	Methodology	Data Requirements	Cost and Applicability
Forward-looking assessment	* Assessment of damage at soil as soil liquefaction hazard measure * Assessment of vulnerability and consequent fragility of single built asset (with and without physical mitigation intervention) * Assessment of the exposure in terms of value of the built asset, community living there, value of the commercial or industrial activity (with and without mitigation) * Risk estimation	* Geotechnical data * Seismological data * Geological data * Hydrological data * Structural data of single built asset * Data on built asset users * Data on built asset owner * Technical data of physical mitigation intervention	* Accurate technical data * Computationally costly * Applicable mostly to single built assets
Backward - looking assessment	* Data collection of damages caused by EILDs in terms of economic and social losses * Adaptation of the risk assessment to the new conditions of the built asset and community	* Seismological data of past event * Hydrological condition at the soil liquefaction event occurred * Geological data * Geotechnical data * Structural damage of the built asset	* Time and resource consuming data collection of damages * Applicable only if a good database is available.

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		* Economic losses occurred * Social losses occurred * Policies for disaster management implemented	

Table 6.1 Types of assessment in CBA for Disaster Management modified from Melcher et al. (2014) and customized for IELDs

6.7 Forward-Looking Risk Based Liquefaction Assessment

6.7.1 STAGE 1: Antecedent Condition without Liquefaction Mitigation

6.7.2 STAGE 1.1: Liquefaction Hazard Analysis

The first stage in the forward-looking framework is to establish the risks of liquefaction at the asset/site level. This process involves analysing the liquefaction hazard across a city/region to identify the intensity and recurrence profile of the phenomenon. In the case of liquefaction, the hazard comprises of two phenomena; the occurrence of an earthquake event and the susceptibility of the soil to liquefy. As such, analysing the liquefaction hazard needs to combine an assessment of ground shaking and the impact this has on the local soil structure. There are a number of approaches to assessing liquefaction susceptibility (National Academy of Sciences 2016)).

6.7.3 They are generally difficult and costly to apply at scale and as such they are unlikely to prove cost-effective in assessing liquefaction susceptibility unless preconditions exist that suggest the likelihood of liquefaction at a particular location is high. The assessment of soil liquefaction potential can be obtained from macrozonation and microzonation maps (see WP2 for more details of approaches to developing macrozonation and microzonation maps for soil liquefaction). At different scales, such maps can be used for screening and planning purposes to identify liquefaction 'hotspots' within a region that require site-specific investigation comprising context specific tests and numerical modelling to quantify liquefaction susceptibility. A summary of the approaches that can be used to assess the susceptibility of a site to liquefaction are summarised in Figure 6.2

6.7.4 The figure shows that at the country/regional scale the analyses improve knowledge of the components that determine the seismic risk and provide selection criteria to prevent and reduce the risk. At the urban scale the analyses identify local seismic hazards and provide

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assessments of local vulnerability of the systems exposed to the hazard. At that level the analyses also identifies targets and priorities for the evaluation of mitigation interventions to reduce risks. In identifying targets and priorities, urban planning considers three components. The structural component identifies the critical components within the region that are at risk and identifies strategies and policies for reducing the risks. The operational component converts the structural component into a series of plans and actions to mitigate the risk. The implementation component develops the detailed analysis to support the implementation of the mitigation actions (Bramerini, et al., 2008).

6.7.5 The localised liquefaction analyses can be used to inform the design of new, or the retrofitting of, existing structures/infrastructures. For new structures the analyses provide guidance to support design and construction whilst for existing structures/infrastructures the analyses identify the specific in-depth investigations that should be carried out to support the design and retrofit of mitigation actions (Bramerini, et al. 2008).

6.7.6 The localised liquefaction analysis can also be used in emergency planning to inform both the design of the emergency plans and to test the resilience to an EILD event. At the design stage the analyses can ensure that the location of critical emergency structures and assets are located in non-liquefy above areas. At the resilience testing stage the analyses can help identify 'critical' features of the transport and service infrastructures that are susceptible to liquefaction and that have a major impact on community resilience.

6.7.7 Within LIQUEFACT, the data on liquefaction hazard mapping generated in WP2 and integrated in the LRG software will be used to develop a Susceptibility Matrix (Bartolucci and Jones, 2016) that relates characteristic earthquake to ground characterization in order to identify the level of hazard of the asset. This analysis will provide asset managers and other stakeholders with an assessment of the range of exposures that their asset(s) is/are likely to be susceptible to.

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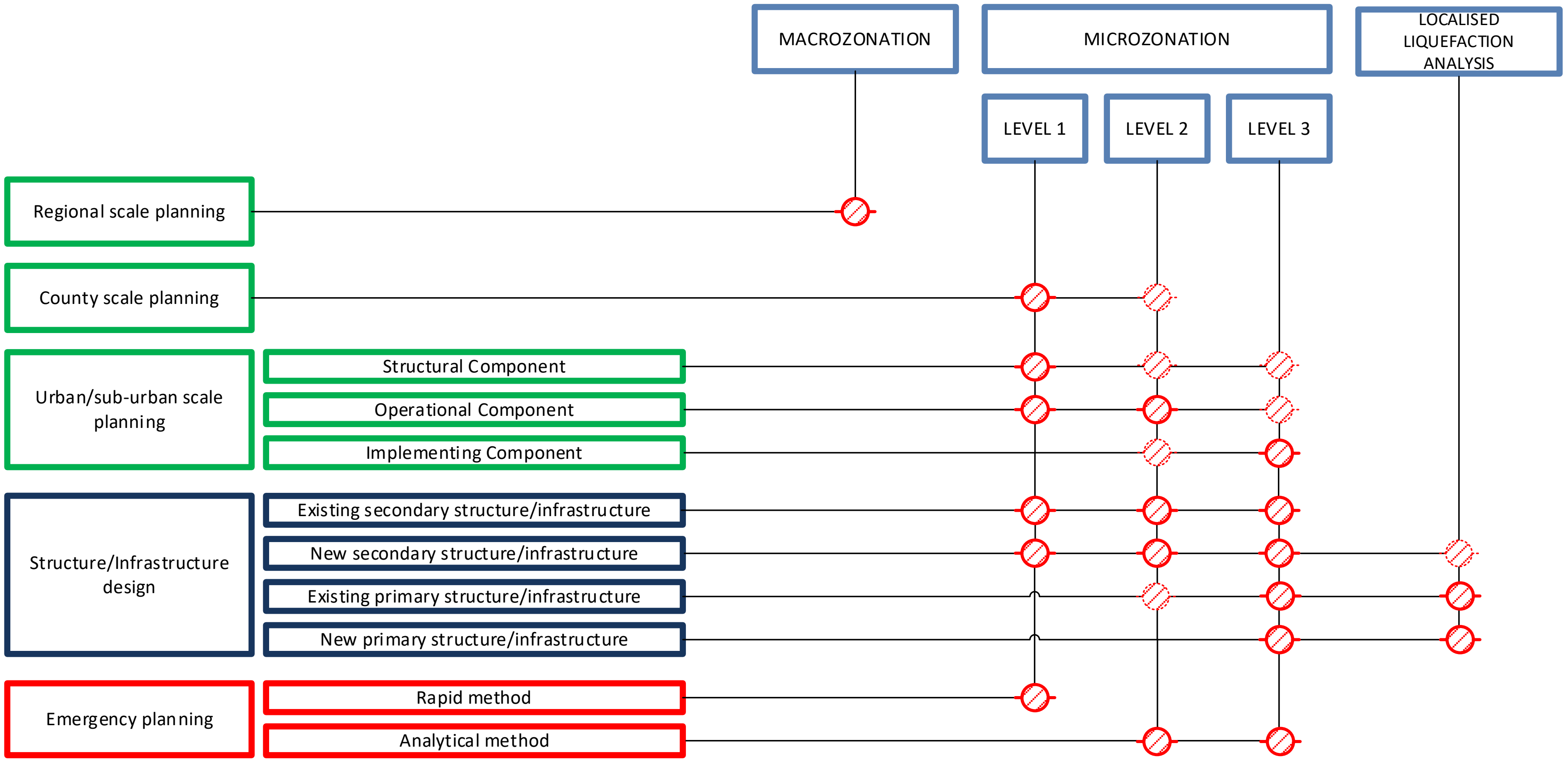

Figure 6.2 - Use of Macro And Microzonation maps and local field test

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6.7.8 STAGE 1.2: Liquefaction Exposure analysis

6.7.9 Liquefaction exposure analysis aims to identify the people, property, systems, or other elements present in zones susceptible to liquefaction and that are thereby subject to potential losses. The exposure analysis needs to consider not only the current socio-economic and land-use patterns, but also anticipate future changes as these could significantly affect losses in the future (Kull, 2008). When assessing current socio-economic parameters consideration needs to be given to measuring the population demographic living in an area, as well as the number and value (unit values) of built assets (e.g. private houses, public buildings, factories, business premises etc.). Predictions of future population demographics can be based upon expected annual growth rates obtained from local statistics and/or reports. A similar approach can be applied for built assets, where their growth is assumed proportional to population growth (Mechler, 2005). In LIQUEFACT exposure analysis will be undertaken through reference to the national and local statistics that are available to the research team.

6.7.10 STAGE 1.3: Vulnerability analysis:

6.7.11 Vulnerability analysis involve the identification of the potential consequences of the exposure of a building/asset to an EILD event. It is usually expressed in terms of fragility. The latest is the probability of the structure or structural component to overcome a performance limit state when subjected to a range of seismic action. . The limit states can range from minor cosmetic damage to total collapse through serviceability disruption. Whilst the limit states are better defined for superstructures than for geotechnical systems, (National Academy of Sciences, 2016) neither actually relates the physical damage to an asset with the operational performance of that asset. This, combined with the uncertainties associated with the levels of damage to the structure and the corresponding consequences in terms of direct repair costs, loss of functionality, and human injuries (Bradley et al., 2010) causes difficulties in assessing the potential loss profiles. In the author's opinion, this is a major weakness with this approach.

6.7.12 Bartolucci and Jones (Liquefact Deliverable 1.3) suggested identifying a building/infrastructure typology using an integrated classification system (likely to be a combination of construction and foundation type), and through the combination with the associated pre-defined vulnerability model (ground shaking and liquefaction fragility curves), assessing the potential level of damage and vulnerability to the ground condition scenarios can be evaluated. For asset managers and other stakeholders, the result of this analysis of damage and vulnerability will be provided in the form of a classification using qualitative labels.

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6.7.13 STAGE 1.4: Risk analysis

6.7.14 Risk is defined as the combination of the probability of an event and its negative consequences (UNISDR, 2009). Combining hazard, exposure and vulnerability leads to risk (UNDRO, 1979). "A tool often that can be used to achieve a quantitative estimation of risk and potential damages as well as benefits of reducing damages, is the loss exceedance function indicating the probability of an event not exceeding a certain level of damages" (Mechler, 2005). A loss exceedance curve is developed that "indicates the probability of an event exceeding a certain loss level. While a 100-year event could occur two or more times in a century, the probability of this happening is of course lower than that of a single 100-year event. To avoid misinterpretation (a common problem with return periods), the exceedance probability often is the better conceptual representation as compared to the cumulative loss distribution; however, the information that both present is actually the same. The area under a loss exceedance curve represents the expected annual value of losses, that is, the average annual amount of losses that can be expected over a long (infinite) time horizon. This concept helps in translating infrequent events and damage values into an annual number that can be used for planning purposes" (Kull, et al. 2013). Loss exceedance curves have been developed in case of liquefaction in order to assess various repair methods for bridges (Kramer et al., 2009), and to described a loss-level evaluation of a pile-supported bridge (Bradley et al., 2010).

6.7.15 Natural disasters result in a range of impacts that affect social, economic, environmental, and heritage elements. Further, the impacts can occur as a direct result of the disaster event or over time as indirect or macroeconomic effects (Mechler, 2005). However, because soil liquefaction is a very localised phenomenon the variety and range of impacts is generally limited when compared to that which would be expected from large-scale events (Tokimatsu et al., 2018), which in case of that phenomenon is an earthquake occurrence. This said, the impacts associated with an EILD event still occur across the social, economic, environmental and heritage elements. The expected range of direct (immediate effect of liquefaction) and indirect (medium-long term effect, occurring as a result of the direct impacts) impacts associated with EILD events are summarised in Table 6.2. Table 6.2 also identifies where it is possible to assign monetary values to these impacts. The table includes also impacts associated to both earthquake and soil liquefaction occurrences, because it is not possible to clearly distinguish the impact due to an earthquake event from that related to the consequent EILD event. However, EILD causes additional impacts across the social, economic, environmental and heritage elements that in Table 4 are indicated with an additional (L). The impacts identified in Table 4 will be validated as part of the CBA model developed in the LIQUEFACT case studies to assess the potential mitigation options.

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Costs					
	Potential Sectors involved	Monetary		Non-Monetary	
		Direct	**Indirect**	**Direct**	**Indirect**
Social	Households	- Furnishings and fittings	- Provisional housing -Provision of alternative services - Increase of housing cost (Chang-Richards et al., 2016) - Loss of wages, reduced purchasing power	- Mortality and morbidity rates (Chang-Richards et al., 2017) - Loss of public services - Loss of private service	- Stress related disorders - Reduction of living standard - Increased poverty
Economic	Households	- Value of housing damaged or destroyed	- Cleaning up of sand ejecta (Villemure et al., 2012) (L) - Post-event survey' - Repair, and reconstruction		
	Infrastructure: Education Healthcare Power Transport Water and sewage	- Assets destroyed or damaged - Repair, and reconstruction	- Loss of critical infrastructure services - Cleaning up of sand ejecta (Villemure et al., 2012)(L) - post-event survey'		

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	Industry Commerce Services	- Assets destroyed or damaged - Repair, and reconstruction	- Loss due to reduced production or complete closure (Chang-Richards et al., 2016) - Post-event survey - Employment of alternative chain suppliers (Chang-Richards et al., 2016) - Unemployment		- Lack of skilled manpower (Chang-Richards et al., 2016)
Environmental	Pollution		- Decontamination of soil and water-bearing stratum (L)		- Leakage of sewage in water-bearing stratum due to damage to drain failure (L) - Increase of special waste due to demolition of structures and cleaning up of sand ejecta (L)
Heritage	Cultural		- Loss due to Business closure - Post-event survey'	- Cultural and historical assets damaged or destroyed	
	Natural			- Impact on natural habit (L)	- Effects on biodiversity (L)

Table 6.2: Impacts and costs associated to liquefaction

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6.7.16 Within LIQUEFACT, the overall performance and functionality of the asset is evaluated on a case by case basis using expert knowledge to interpret the impact the damage and risk (from Stage 3) will have on functionality and performance.

6.8 STAGE 2: Identification of potential liquefaction risk management measures and costs

6.8.1 LIQUEFACT has identified two approaches to mitigate liquefaction: modifying the seismic demand by reducing the site susceptibility to liquefaction; and enhancing the capacity of structures to reduce the damage caused by liquefaction. The first category includes techniques that improve the liquefaction strength of the soil (ground improvement techniques), usually by one or more of the following factors:

- Densification of the liquefiable soil (to be achieved with any kind of compaction)
- Stabilization of soil skeleton (to be achieved by different actions)
- Dissipation of increased excess pore pressure (e.g. by improving drainage capacity)
- Desaturation of the liquefiable soil.

6.8.2 Appropriate mitigation to enhance the capacity of structures to reduce the damage caused by liquefaction differ depending upon the type of structure and the potential impacts that liquefaction could have (e.g. lateral displacement, differential settlement, etc.). The most common strategies for liquefaction retrofit of existing structures are:

- restriction or change of use,
- partial demolition and/or mass reduction,
- removal or lessening of existing irregularities and discontinuities,
- addition of new lateral load resisting systems,
- local or global modification of elements and systems,

6.8.3 Once the mitigation alternatives have been identified for a specific built asset then the costs of retrofitting the mitigation options can be calculated using the standard cost-benefit approach outlined in Figure 6.1. Within LIQUEFACT, once the level of loss of performance and functionality of individual building/infrastructure assets and the impact on the resilience of a community following an EILD event has been established, end-users will be directed to develop a customized mitigation measure. Based on the outcomes of the hazard-risk analysis (from Stage 1.1 to 1.4), a range of mitigation actions will be identified, and the effect of each on the level of performance of individual buildings/infrastructure assets will be evaluated. Two types of mitigation actions need to be considered: those that seek to reduce a building/infrastructure asset's vulnerability/increase its resilience; and those that seek to reduce the hazard level. Mitigation options (Table: 6.3) will be ranked according to their

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impact on the sub-system level and on their contribution to improving overall community resilience. Table 6.3 includes also a list of mitigations based on land use planning and economic incentives, which are not investigated within LIQUEFACT project. Further details of direct soil liquefaction mitigation methods, their cost-benefit relations and their implementation into the LIQUEFACT LRG software toolbox are given in Appendix A.

Mitigation actions		Application sectors	Benefits	
			Direct	Indirect
Direct on built assets	Strengthening of existing buildings	Household	- Reduction of structural damages - Reduction of costs for cleaning up of sand ejecta	- Reduction of loss of furnishings and fittings - Reduction of costs for provisional housing - Reduction of morbidity and mortality rates - Reduction of repair and reconstruction costs - Reduction of (clinical treatment for) stress related disorders - Stability of living standard - Stability of housing cost - Economic stability
		Infrastructure	- Reduction of damages to critical infrastructures - Reduction of costs for cleaning up of sand ejecta	- Reduction of morbidity rate - Reduction of costs for provision of alternative services - Reduction of repair and reconstruction costs

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				- Reduction of cost for soil and water bearing stratum decontamination costs - Reduction of special waste due to demolition and cleaning up of sand ejecta - Preservation of natural habitat and biodiversity
		Industry, commerce and services	- Reduction of loss of private service	- Enough skilled manpower - Reduction of soil decontamination costs - Economic stability - Continuity of service - Stability of employment rate
		Cultural heritage	- Reduction of loss of cultural and historical assets	- Reduction of economic loss due to business closure
	New standards for design	Household	- Reduction of structural damages - Reduction of costs for cleaning up of sand ejecta	- Reduction of loss of furnishings and fittings - Reduction of costs for provisional housing - Reduction of morbidity and mortality rates - Reduction of costs for repair and reconstruction

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				- Reduction of (clinical treatment for) stress related disorders - Stability of living standard - Stability of housing cost - Economic stability
		Infrastructure	- Reduction of damages to critical infrastructures - Reduction of repair and reconstruction costs - Reduction of costs for cleaning up of sand ejecta	- Reduction of morbidity rate - Reduction of costs for provision of alternative services - Reduction of cost for soil and water bearing stratum decontamination costs - Reduction of special waste due to demolition and cleaning up of sand ejecta - Preservation of natural habitat and biodiversity
		Industry, commerce and services	- Reduction of loss of private service	- Enough skilled manpower - Reduction of soil decontamination costs - Economic stability - Continuity of service - Stability of employment rate

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Land use planning †	Household	- Reduction of structural damages - Reduction of costs for cleaning up of sand ejecta - Reduction of cost of cleaning up of sand ejecta	- Reduction of loss of furnishings and fittings - Reduction of costs for provisional housing - Reduction of morbidity and mortality rates - Reduction of costs for repair and reconstruction - Reduction of (clinical treatment for) stress related disorders - Stability of living standard - Economic stability
	Infrastructure	- Reduction of damages to critical infrastructures - Reduction of costs for cleaning up of sand ejecta	- Reduction of morbidity rate - Reduction of costs for provision of alternative services - Reduction of cost for soil and water bearing stratum decontamination costs - Reduction of special waste due to demolition and cleaning up of sand ejecta - Preservation of natural habitat and biodiversity
	Industry, commerce and services	- Reduction of loss of private service	- Enough skilled manpower

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		- Reduction of cost of cleaning up of sand ejecta	- Reduction of soil decontamination costs - Economic stability - Continuity of service - Stability of employment rate
Economic incentives †	Household	- Economic stability	- Stability of cost of housing
	Infrastructure		- Economic stability
	Industry, commerce and services	- Economic stability	- Stability of employment rate

Table 6.3: Potential Mitigation Actions

† Not investigated in LIQUEFACT

6.9 STAGE 3: Analysis of the risk reduction: Potential impacts with risk management

6.9.1 The benefits of reducing risk through mitigation risk can be estimated by summing the avoided direct, indirect (Table 6.3) and macroeconomic costs (Loss of GPD) associated with the antecedent condition (i.e. those costs that would be expected from a EILD event where no mitigation was present) with those that would be expected following the introduction of mitigation actions. This would introduce a quantification of tangible benefits, i.e. a quantification of the potential impacts of mitigations in the risk management.

6.9.2 STAGE 4: Estimating efficiency of liquefaction risk management measures

6.9.3 The final step in a CBA is to compare costs and quantified benefits and calculate the efficiency of the analysed options. There are two steps for doing this. First the benefits arising over time need to be discounted to current value and then compared to the costs (also discounted to current value). The comparison can be made using a number of standard efficiency measures: net present value (NPV); Benefit/Cost Ratio and/or the internal rate of return (IRR) (Mechler, 2005).

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6.9.4 Backward-looking framework (impact-based approach)

6.9.5 The main difference between the forward -looking framework and the backward-looking framework is that data from past EILD events is used as the basis for understanding risk and assessing the impact of potential damage rather than hazard and vulnerability models (Mechler, 2005). The backward-looking framework can again be described as a four stage model (Figure 6.2).

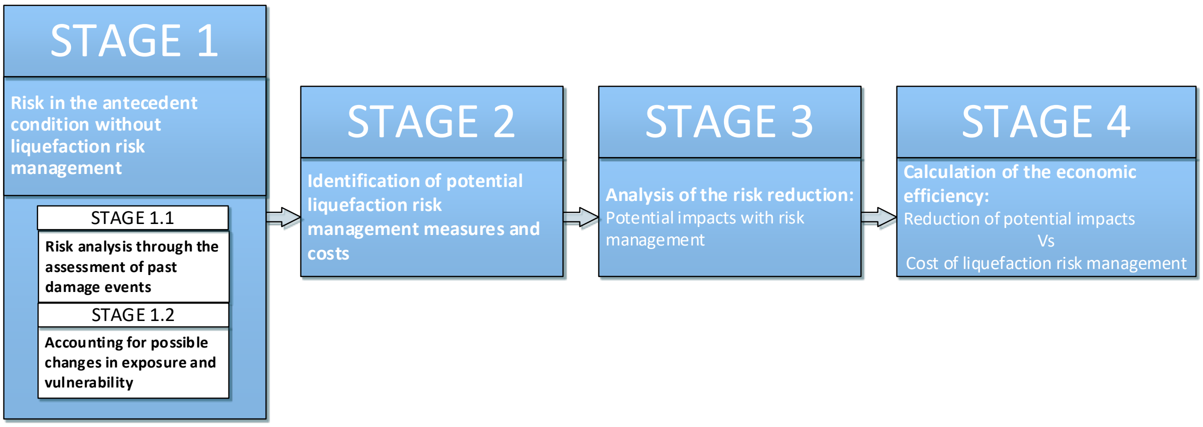

Figure 6.2 Backward-looking framework for CBA in liquefaction risk management: adapted from Mechler et al. (2014)

6.9.6 STAGE 1.1: Assessment of past damage events and recurrence of events

According to Mechler (2005) the outcome of this assessment is to identify risk in the past as demonstrated by occurred damages/impacts. "In order to assess damages in monetary terms based on reported impacts of past disasters, relevant indicators of impacts need to be identified. Generally, the prime source for past-disaster impacts are loss-assessments conducted by local, regional and national governments, industry and commercial groups and disaster management authorities. Another source of information are standardised databases on disaster losses. Mostly these sources will cover the direct economic impacts and the immediate social health consequences (in non-monetary terms). Conventionally, the indirect effects should be assessed during a 5 year time period after an event, whereby the major ones occur during the first two years. In theory, these effects should be counted "throughout the period required to achieve the partial or total recovery of the affected production capacity" (ECLAC 2003)" (Mechler, 2005). ""Indirect effects can be estimated after an event by:

- Conducting surveys post event: bottom-up;

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- Examining statistical information on the performance of affected sectors after the event in top-down manner;
- Deriving simple relationships" (Mechler, 2005).

In LIQUEFACT assessments of the damages caused by past liquefaction events will be used to derive direct economic indicators of the impacts of recent past liquefaction events. Whilst the LIQUEFACT project team know of the existence of such analyses it is not clear and present as to the extent of the economic data they contain. It is anticipated that the data sources will cover the direct economic impacts and the immediate social health consequences. The LIQUEFACT project will evaluate the availability of the above datasets in the case study region to evaluate the appropriateness of using the backward-looking framework as part of the CBA model.

6.9.7 In the backward-looking framework the hazard return period is calculated by considering the time period between previous hazard events to determine frequency and probability values for return periods such as 10, 20 and 100 years. Whilst there will be considerable uncertainty as to the exact return periods, for CBA purposes such information can be used as long as the uncertainty is acknowledged in the final estimates.

6.9.8 STAGE 1.2: Accounting for possible dynamics in exposure and vulnerability

The outcome of this assessment is to update the risk to current conditions. Whilst liquefaction may not be considered as a dynamic hazard, vulnerability and exposure can change over time. For example, assets may have increased due to population increase, migration into area and increased economic activity; whilst vulnerability may be reduced due the implementation of new design guidelines. This can be achieved by shifting the original loss-frequency curve representing risk of potential damages downwards by implementing risk management measures decreasing damages associated with a certain probability; or by shifting the curve upwards in case the risk may increase as increases in population, assets and economic activity may increase exposure. In LIQUEFACT this approach will require an assessment of changes in terms of population profile, the number and value of physical assets, land use etc. that has occurred since the last EILD event. Again, whilst the LIQUEFACT team believe this data exists, quality and value of this data to the backward-looking CBA model has not been assessed.

6.9.9 STAGE 2, 3 and 4

Stages, 2, 3 and 4 of the backward -looking framework are calculated in the same way as for the forward-looking framework.

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6.9.10 The CBA model outlined in this section to assess the potential of mitigation interventions to improve community and critical infrastructure resilience to EILD events is a theoretical model developed by considering best practice in CBA modelling of disaster events and the specific constraints associated with the impact of soil liquefaction. The theoretical model forms part of the LIQUEFACT RAIF and LRG tools which will be integrated into a Built Asset Management (BAM) framework currently being developed by the LIQUEFACT project. Once the BAM framework is complete (expected December 2018) it, along with the CBA model described in this section, and the LIQUEFACT toolbox being developed by WP2, WP3 and WP4 will be tested through the case studies (WP7). The final versions of the LIQUEFACT RAIF, the LRG, the CBA model, and the LIQUEFACT toolbox will be developed once the case study analysis is complete.

7. Improving Community Resilience to Earthquake Disaster Events: Integrating CBA into the RAIF

7.1 The CBA is an integral part of the RAIF. The RAIF is a decision support tool, developed by the LIQUEFACT project, for built asset owners and/or managers to assess the impact of an EILD event on individual buildings/infrastructure assets, multiple buildings/infrastructure assets on a single site, or portfolios of buildings/infrastructure assets across multiple sites. The RAIF can also be used by international, EU, regional and local decision makers to assess the impact of an EILD event on community support systems (e.g. healthcare, public transportation, etc.). The RAIF (Figure 2.1) provides a mechanism to assess the potential improvements to the resilience of built assets and community systems that can be achieved from a range of mitigation actions.

7.2 The RAIF is supported by the LIQUEFACT toolbox (a range of specific tools being developed in the LIQUEFACT work packages) and by the LIQUAFACT Reference Guide (LRG) being developed in work package 6 (see Appendix B). The logic behind the RAIF and the LRG, and the ability of the individual tools to support it were tested in a 'sprint test' workshop held in Rome on the 17th November 2017. The sprint test workshop (see Appendix C) confirmed the appropriateness of the LIQUEAFCT tools and the business model framework within which the RAIF and LRG would be used. The business framework, including the role of CBA, is outlined below.

7.3 The 6 stage model underpinning the application of the RAIF is similar to the 4 stage model outlined in Section 6 for the development of disaster mitigation CBA models. The 6 stages of the RAIF are:

Stage 1 - Antecedent Condition Analysis: examine the hazard risk to the buildings and critical infrastructure within the geographical area under investigation (e.g. individual

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building/infrastructure asset, portfolio of buildings/distributed infrastructure assets, town/city wide buildings/infrastructure, regional wide buildings/infrastructure, state wide buildings/infrastructure assets etc.). The hazard risk assessment needs to consider both direct and indirect impacts of the hazard on the community. The hazard risk assessment will use multi-criteria analysis to define inherent vulnerabilities at the physical, social, environmental and economic level.

Stage 2 - Impact Assessment: develop a matrix of vulnerabilities against hazard impacts. The matrix needs to consider each impact separately (e.g. physical system, social system etc.) and identify the ability of each sub-system component (e.g. building, infrastructure, employment etc.) to cope with and recover from the impact. For each sub-system component that has a high vulnerability and a low coping capacity, possibly mitigation interventions to either reduce vulnerability; improve coping capacity; or achieve both need to be identified.

Stage 3 - Scenario Condition Analysis: model the effect of the interventions identified in stage 2 at the sub-system component level using a multi-criteria methodology at the system level to establish the overall effect of the mitigation interventions on inherent system vulnerability. The scenario condition analysis will also require inter-actions between systems (e.g. physical, social etc.) to be modelled to identify the collective impact of each of the sub-system component interventions on the overall resilience of the community.

Stage 4 - Mitigation Options: identify a series of specific (sub-system component level) mitigation interventions that can be specified at the level of detail required to allow initial options appraisal to be carried out. The specification should describe explicitly the improvement in performance required at the sub-system component level and the methods that will be used to measure whether this performance is achieved in practice.

Stage 5 - Improvement Framework: develop CBA models for each specific sub-system component. The CBA will need to consider both direct and indirect costs (e.g. physical, loss of revenue during refurbishment period, etc.) and benefits (e.g. to the organisation, community, etc.) and extend these analysis across geographical and temporal scales (e.g. consider the inter-relationships between multiple similar assets, consider the implications of delaying refurbishment until later in a building/infrastructure life cycle). Once the CBA has been completed for all sub-system components interventions consideration will need to be given setting intervention priorities and sequencing of work. The adaptive capacity of all stakeholder groups to fund and manage the retrofitting of mitigation interventions will need to be assessed (e.g. availability of capital, governance requirement, legislation etc.) and priorities set for both the mitigation interventions to be enacted (it is very unlikely that sufficient adaptive capacity will be available to adopt all the mitigation actions suggested by the multi-criteria model) and the timescales over which they will be programmed (e.g. retrofitting of buildings/infrastructure mitigation interventions are likely to be programmed periodically over the assets normal refurbishment cycle – up to 30 years in some cases).

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Stage 6 - Built Asset Management Planning: once priorities have been set, detailed built asset management plans can be developed. These plans require detailed design solutions to be developed for each mitigation intervention and all financial and legal conditions to be addressed before contracts are let. Once implemented, the performance of the mitigation intervention against the performance specification detailed in stage 4 is monitored through detailed simulation or in response to an EILD event.

7.4 Stage 5 of the above requires the development of CBA models to prioritise the mitigation interventions. The CBA models will be developed at the individual building/asset level and then combined to produce portfolio, city, and if required regional level models. The CBA models will be developed using a variant of the forward-looking and backward-looking frameworks outlined by Mechler (2005) and customised by the LIQUEFACT team to reflect the specific circumstances associated with EILD events. The proposed application process of these frameworks within the RAIF is outlined in Table 7.1 The application process will be validated through case study scenarios undertaken as part of WP7.

7.5 A sprint test workshop was held with members of the LIQUEFACT team to validate the above. The sprint test used a hypothetical hospital scenario was used to test the logic and availability of data from within the LIQUEFACT project to test the RAIF, LRG and CBA. The sprint test, in conformity with the LIQUEFACT Deliverable 4.1, used a discrete matrix approach to demonstrate the RAIF. The PowerPoint slides supporting this workshop are given in Appendix C.

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Step	Activity	Data Source
1	Define the geographical area under investigation. This could be a site, town, city or region. Define the key objectives (in terms of resilience improvements) required from the study. This could be at the organisation, town, city or regional level and could involve specific operational improvements or more general community resilience improvements.	The key stakeholder commissioning the study.
2	Identify the general susceptibility of foundation soil of critical buildings/assets located in the region under investigation to EILD events. This will involve the use of macrozonation and microzonation analyses.	European macrozonation map and microzonation guidelines and microzonation studies for Liquefact WP2 case studies are available from WP2. The macrozonation map and the guidelines for microzonation studies will be given in the final version of the LRG.
3	For each critical infrastructure and building/asset relevant to the community and located in area susceptible to soil liquefaction commission a detailed geotechnical investigations (site investigations, physical modelling, computer modelling etc.) to further understand the potential susceptibility of the site to earthquake induced liquefaction.	Guidelines for commissioning a detailed geotechnical investigation at the site level are being developed in WP4 and will be available in the LRG.
4	For those sites where the detailed geotechnical investigations confirm their susceptibility to earthquake induced liquefaction, identify the specific impacts (in terms of vulnerability and fragility) that a liquefaction event would have on the buildings/infrastructure on the site.	Fragility curves for a range of typical buildings/infrastructure are being developed in WP3 and the potential impacts of soil liquefaction on buildings/infrastructure is being developed in WP4. The outputs from WP3 and WP4 will be available through the LRG.

5	For those buildings at risk of physical damage as a result of soil liquefaction assess the effect that such damage would have on the performance of the buildings/assets (in terms of the impact that loss or reduced functionality at the serviceability and ultimate limit states) has a potential impact on the society. The loss of functionality (performance) will be made on a case by case basis using the expert knowledge of the facilities manager and building users to interpret the impact that any given level of risk (a qualitative score ranging from very high to very low) will have on service functionality and performance.	A combination of the outputs from WP2, WP3 and WP4 will be used to categorise the level of risk. The built asset management plan to be developed in WP5 will provide the guidelines for linking damage to buildings to loss of performance. All of the above will be available through the LRG. The community resilience model to be developed following the case study analyses (in WP7) will be used to assess the potential impact that a loss of performance of individual buildings and assets will have on overall community resilience.
6	A range of mitigation actions will be identified (both physical and operational) for each building/asset identified as at high risk and whose impact has an adverse effect on community resilience. Two types of mitigation actions will be considered: those that seek to reduce a building/infrastructure assets vulnerability/increase its resilience; and those that seek to reduce the hazard level. The former are likely to be building level interventions; the latter are likely to be ground level interventions.	A combination of the outputs from WP3 and WP4 will be used to identify a range of technical building and ground level mitigations. Operational mitigations will be developed in WP5. The mitigation options will provide sufficient detail on reduced physical impact to allow post mitigation service level performance to be assessed.

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7	Once the mitigation options have been identified a CBA will be performed for each specific sub-system component. The cost/benefit analysis will consider both direct and indirect costs (e.g. physical, loss of revenue during refurbishment period, etc.) and benefits (e.g. to the organisation, community, etc.) and extend the analysis across geographical and temporal scales (e.g. consider the inter-relationships between multiple similar assets, consider the implications of delaying refurbishment until later in a building/infrastructure life cycle). A hybrid version of the forward-looking-and backward-looking frameworks developed by Mechler (2005) and customised for EILD events will be used.	
8	The cost (capital and operating) of implementing each mitigation option will be derived from building cost databases (for rebuild and repair) and where necessary supplemented from historic accounts and contractor's estimates. The cost of operational mitigations will be derived through discussions with the building/asset owners/FM.	Cost databases, historic records, contractor's estimates, and the building/asset owners/Facility Manager (FM).

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9	The benefits in terms of avoided losses without mitigation at the organisation level will consider both tangible and intangible losses. Tangible losses include: repair and rebuilding of buildings/assets; replacement of fixtures and fittings; clean-up and decontamination; loss of business; loss of income. Intangible losses include: loss of reputation; loss of market share; disruption to the supply chain, including additional costs associated with substitute services; etc. additional operating costs; additional human resources costs, including disruption to the workforce and availability of skilled labour; increased insurance costs; etc. The additional intangible losses without mitigation at the community level additionally include: increased mortality and morbidity rates; costs of temporary substitute services; loss of wages; increased poverty; increased levels of stress; reduce standards of living; economic stability; destruction of habitat/biodiversity; etc.	The total tangible costs will be calculated against a range EILD scenarios. Tangible direct losses will be derived from cost databases, historic records, contractor's estimates, and the building/asset owners/FM. Direct intangible losses will be derived from discussions with the organisation owners/FM and the use of the LIQUEFACT CI Resilience Scorecard. The additional intangible losses at the community level will be calculated with reference to historic datasets, discussions with community level representatives and the use of the LIQUEFACT Community Resilience Scorecard.
10	A loss-frequency curve will be developed that assesses the loss profile that could be expected over the remaining service life of the asset. The loss profile will take account of the likelihood of an EILD event affecting the organisation's buildings/assets and of the estimated losses should such an event occur. The loss-frequency curve will be calculated using both a forward-looking (risk-based) framework and a backward-looking (impact-based) framework. The loss frequency curve will only consider tangible losses. All losses will be discounted to the present value to allow direct comparison with current costs (Mechler, 2005). The impact of intangible losses will be assessed using a multi-criteria model to be developed as part of the LIQUEFACT RAIF.	The loss frequency profile will be derived through discussions with the building/assets owners/FM. The effect of intangible losses will be modelled using a multi-criteria model that combines the outputs from the LIQUEFACT CI Scorecard and the LIQUEFACT Community Resilience Scorecard with the aggregated results of the loss frequency curves for all the critical assets identified by the stakeholders who commissioned the study.

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11	The benefits in terms of avoided losses with mitigation at the organisation level will consider both tangible and intangible losses. Tangible losses include: repair and rebuilding of buildings/assets; replacement of fixtures and fittings; clean-up and decontamination; loss of business; loss of income. Intangible losses include: loss of reputation; loss of market share; disruption to the supply chain, including additional costs associated with substitute services; etc. additional operating costs; additional human resources costs, including disruption to the workforce and availability of skilled labour; increased insurance costs; etc. The additional intangible losses without mitigation at the community level additionally include: increased mortality and morbidity rates; costs of temporary substitute services; loss of wages; increased poverty; increased levels of stress; reduce standards of living; economic stability; destruction of habitat/biodiversity; etc.	The total tangible costs will be calculated against a range EILD scenarios. Tangible direct losses will be derived from cost databases, historic records, contractor's estimates, and the building/asset owners/FM. Direct intangible losses will be derived from discussions with the organisation owners/FM and the use of the LIQUEFACT CI Resilience Scorecard. The additional intangible losses at the community level will be calculated with reference to historic datasets, discussions with community level representatives and the use of the LIQUEFACT Community Resilience Scorecard.
12	A loss-frequency curve will be developed that assesses the loss profile that could be expected over the remaining service life of the asset. The loss profile will take account of the likelihood of an EILD event affecting the organisation's buildings/assets and of the estimated losses should such an event occur. The loss-frequency curve will be calculated using both a forward-looking (risk-based) framework and a backward-looking (impact-based) framework. The loss frequency curve will only consider tangible losses. All losses will be discounted to the present value to allow direct comparison with current costs (Mechler, 2005). The impact of intangible losses will be assessed using a multi-criteria model to be developed as part of the LIQUEFACT RAIF. (e) - life-cycle losses with mitigation.	The loss frequency profile will be derived through discussions with the building/assets owners/FM. The effect of intangible losses will be modelled using a multi-criteria model that combines the outputs from the LIQUEFACT CI Scorecard and the LIQUEFACT Community Resilience Scorecard with the aggregated results of the loss frequency curves for all the critical assets identified by the stakeholders who commissioned the study.

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13	Calculate the benefit/cost ratio by evaluating the area between the loss frequency curve without mitigation and the loss frequency curve with mitigation and discounting these values to current day. If the benefit/cost ratio is greater than one then implementing the mitigation action is economically cost-effective (Mechler, 2005). The benefit/cost ratio at the community level will be calculated by aggregating the benefit/cost ratios for all the critical assets identified by the stakeholder commission the study.	From step 10 and step 12 above.
14	Compare the economic (quantitative) and social (quantitative) performance of each mitigation interventions against the business needs of the organisation and prioritise their inclusion into the Built Asset Management life-cycle. Mitigation interventions would be programmed to occur at some future point in the remaining service life of the asset. The timing of future mitigation interventions will depend on the remaining residual value of the asset and on where the asset currently sits in terms of the organisations maintenance and refurbishment cycle.	WP5 will provide a generic built asset management plan for the programming of EILD event mitigation interventions.

Table 7.1 Proposed application process for evaluating the CBA risk based model of mitigation options.

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8 Summary and Next Steps

8.1 The LIQUEFACT project aims to develop a more comprehensive and holistic understanding of the earthquake soil liquefaction phenomenon and the effectiveness of mitigation techniques to protect structural and non-structural systems and components from its effects. The LIQUEFACT project will evaluate the mitigation techniques against the potential improvements that could accrue to community resilience in regions prone to EILD events. This report provides an introduction to CBA as it is applied to the valuation of mitigation interventions that seek to reduce the impact of disaster events on individual buildings/assets and the wider community. The report outlines the basic principles of a CBA and draws attention to the issues that need to be considered when assessing both the costs and benefits associated with a mitigation intervention. The report also considers the role of CBA in the project development cycle and reviews alternative theoretical approaches that have been developed by researchers studying disaster management and disaster risk reduction mitigation. In reviewing these theoretical approaches the report considers both the benefits and limitations of applying CBA in disaster management and disaster risks reduction and, whilst it acknowledges that the limitations are significant, concludes the benefits of using CBA to inform business decisions, outweighs the limitations. The report then proceeds to develop a bespoke hybrid LIQUEFACT CBA framework that can be applied to the evaluation of alternative mitigation interventions that seek to reduce the impact that EILD events have on individual buildings/assets and the wider community. In developing the LIQUEFACT CBA framework the report considers the specific characteristics of the earthquake induced liquefaction phenomenon and explains how these are addressed within the LIQUEFACT CBA framework. The report also explains how the LIQUEFACT CBA framework is integrated into the LIQUEFACT RAIF and supported by the LIQUEFACT toolbox and LRG. Finally, the report outlines a 14 step model that will be used to validate the LIQUEFACT CBA, RAIF, LRG and toolbox through the case study analyses being undertaken in WP7.

8.2 As the work described in this report is ongoing, this report should be considered a work in progress that will be amended and added to as the LIQUEFACT project progresses. The primary audience for this report are the LIQUEFACT partners and researchers.

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Appendix A

LIQUEFACT: The effectivity of Liquefaction mitigation measures, their cost-benefit relations and their implementation into the LIQUEFACT LRG software toolbox

DRAFT REPORT - SUBJECT TO CHANGE

LIQUEFACT

The effectivity of Liquefaction mitigation measures, their cost-benefit relations and their implementation into the LIQUEFACT LRG software toolbox

Contribution to WP4:
Mitigation Measures against Liquefaction Damage – State-of-the-art report

University of Naples “Federico II”
Department of Civil, Architectural and Environmental Engineering

Carla Santosuosso
Grazia Scarpato

NORSAR

Summary

1 Preliminary considerations

The evaluation of earthquake-induced liquefaction have become a routine part of geotechnical engineering design. PHRI (1997), Cooke and Mitchell (1999), Cook (2000), and Youd (1998) provide a complete process to assess the need of remediation for soil liquefaction. The process involves all key elements in soil liquefaction engineering:

1. assessment of liquefaction potential,
2. assessment of post-liquefaction strength and ground deformation,
3. evaluation of liquefaction-induced consequence and damage by comparing to the design specifications,
4. implementation (evaluation) of engineered mitigation if the consequence estimated in step (3) is unacceptable.

Mitigation efforts may consist of relocation of the construction site, removing native liquefiable soils, bypassing the liquefiable soils with deep foundations, structurally accommodating the deformations or strength loss caused by liquefaction, or preventing the onset of liquefaction through ground improvement. Within the scope of this study, ground improvement is assumed as the preferable remedial measure.

Saturated granular materials, such as sands and silts, subjected to cyclic loading will exhibit a tendency to contract or densify, and thus will generate excess pore pressure if their drainage is too slow. Depending on their initial density and cyclic stress history, these soils may develop excess pore pressure high enough to cause complete loss of shear strength and stiffness at essentially zero effective stress (the historical definition of liquefaction), or cause excess deformations (defined as cyclic mobility or liquefaction with limited strain potential). Both phenomena are particularly severe for native loose soil deposits such as those developed during conventional reclamation work (i.e., hydraulic placed fills). The reduction in strength (and stiffness) can result in permanent deformation ranging from a few meters (i.e., lateral spreading) to hundreds of meters (i.e., flow failure). In addition, loss of bearing capacity to shallow foundations and the floating or sinking of structures frequently occurs. It is clearly understood that the phenomenon of soil liquefaction depends on three principal factors: excess pore pressure, shear strength, and shear strain/deformation of the soil, many of these definitions are based solely on one factor instead of all three.

In 1978, the Committee on Soil Dynamics of the Geotechnical Engineering Division of ASCE recommended the definition of liquefaction as: “The act or process of transforming any substance into a liquid”. It was hoped that this definition was general enough to be universally applicable and error free. Unfortunately, it avoided any quantitative criteria, and thus was too loose to be useful for either researchers or practitioners. In the meantime, the U.C. Berkeley definition of soil liquefaction continues to be “significant reduction of strength and stiffness of a soil, principally as a result of pore pressure increase and corresponding reduction in effective stress.” This definition is deliberately broad and general, and avoids taking a stand with regard to the specific question regarding how much strength and stiffness reduction is required to satisfy the “significant” threshold.

Most other definitions that have been used in academic research and engineering practice are based on either pore pressure criteria, shear strength, or shear strain/deformation criteria.

Modern studies of liquefaction with initial emphasis on liquefaction trigging mechanisms started in 1960s after the Niigata and Alaska earthquakes, such as Seed and Lee (1966), Seed and Idriss (1967, 1971), Martin et al. (1975) and Peck (1979). Experimental testing of liquefiable soils (especially on loose saturated granular soils) coupled with field observations provided the first understanding of the triggering mechanisms. The triggering mechanism has been studied extensively through both experimental testing and numerical methods.

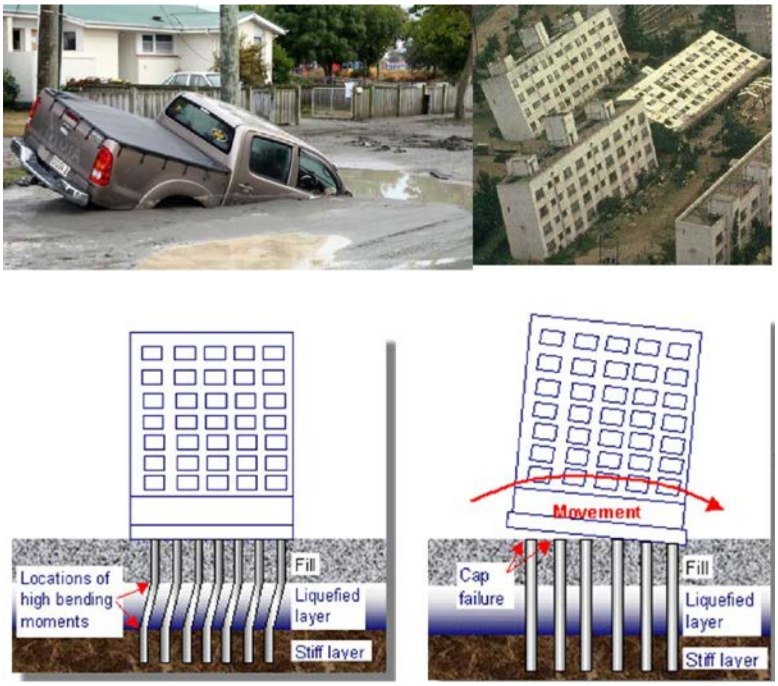

Figure 1-Liquefaction induced deformations and failure modes (After PHRI, 1997).

2 Liquefaction potential evaluation

Once a particular soil is found to be susceptible to liquefaction on the basis of various susceptibility criteria as mentioned in Kramer (1996), it is necessary to control the liquefaction potential of saturated cohesion-less soil based on the intensity and duration of earthquake shaking and the density and effective confining pressure of the soil. Three methods for evaluating liquefaction potential are described briefly in the following subsections:

1. Energy-based approach
2. The cyclic stress-based approach
3. The cyclic strain-based approach
4.

2.1 Energy-based approach

The energy-based approach is theoretically very much appropriate for liquefaction potential evaluation, as the dissipated energy reflects both cyclic stress and strain amplitudes. When a dry soil is cyclically loaded it causes densification at the expense of energy as energy is required to rearrange the individual soil particles. For a saturated soil densification causes an increase in pore water pressure under un-drained condition as the amount of energy required to rearrange soil grains decreases due to decrease in contact forces. Using this principle Davis and Berrill (1982) developed energy based formulation, in which the dissipated seismic energy at a site is considered responsible for the progressive development of pore water pressure and also presented an expression as a criterion for liquefaction. Berrill and Davis (1985) revised their earlier formulation and developed an expression for the pore pressure increase by taking into account a non-linear relationship between the pore pressure increase and dissipated energy, effect of natural attenuation and reassessing the magnitude total radiated energy relationship:

$$\frac{\Delta u}{\sigma_v'} = \frac{120 A^{0.5} 10^{0.75M}}{r N_1^{1.5} {\sigma_0'}^{0.75}} \qquad (1)$$

where

- Δu= increase in pore water pressure;
- σ'_v= effective vertical stress at depth of interest;
- N_1= corrected standard penetration value of the site soil layer under investigation;
- A =material attenuation factor;
- M= earthquake magnitude on the Richter scale;
- r = distance of the site from the centre of energy release.

Law et al. (1990) used the above energy principles and developed a criterion for liquefaction occurrence in sands as given below.

$$\frac{10^{1.5M}}{2.28\times10^{-10}N_1^{1.5}r^{4.3}} \geq 1.0 \qquad (2)$$

Several other investigators have established relationships between the pore pressure development and the dissipated energy during ground shaking (Figueroa et al. 1994; Ostadan et al. 1996). The liquefaction triggering can be formulated by comparing the calculated unit energy from the time series record of a design earthquake with the resistance to liquefaction in terms of energy based on in-situ soil properties (Lianget al. 1995; Dief 2000). The energy based methods, however, is less commonly used due to non-availability of quality data for calibration of these methods.

2.2 Cyclic strain-based approach

The cyclic strain-based approach to evaluate of liquefaction potential is based on experimental evidence that shows densification of dry sands is effectively controlled by cyclic strain rather than cyclic stress and there exist a threshold volumetric strain below, which densification does not occur. Since there are tendencies of sand to density when dry, this is directly related to its tendency to develop excess pore pressure when saturated. This shows that pore pressure generation is more fundamentally related to cyclic strains than cyclic stress. In this approach earthquake induced loading is expressed in terms cyclic strains. The time history of the cyclic shear strain can be estimated from the ground response analysis. As it is difficult to predict cyclic strain accurately, Dorby et al.(1982) developed a simplified method for estimating uniform cyclic strain (γ_{cyc}) from the amplitude of the uniform cyclic stress as originally proposed by Seed and Idriss (1971). Once γ_{cy} is calculated it is compared with threshold shear strain (γ_t):

- If $\gamma_{cyc} < \gamma_t$, no pore water pressure will be generated and thus liquefaction cannot be initiated.
- If $\gamma_{cyc}>\gamma_t$, the occurrence of liquefaction is possible.

Liquefaction potential can be evaluated in this approach by comparing the earthquake induced cyclic loading in terms of the amplitude of a series of an equivalent number of uniform strain cycles with liquefaction resistance, which is expressed in terms of the cyclic strain amplitude required to initiate liquefaction in the same number of cycles. Liquefaction can be triggered at depths where loading exceeds the liquefaction resistance. Dorby et al.(1984) developed a torsional tri-axial test for measurement of liquefaction resistance by imposing cyclic strains under un-drained conditions on a cylindrical tri-axial specimen by strain controlled cyclic torsion. The developed cyclic shear strain induces excess pore pressure in the specimen. Unlike cyclic stress approach, cyclic strain approach is not commonly used as cyclic strain; amplitudes can to be predicted as accurately as cyclic stress amplitude and the cyclic strain-controlled testing equipment is less readily available than the cyclic stress-controlled testing equipment (Kramer and Elgamal, 2001). Thus, the focus of this chapter is on the evaluation of liquefaction potential using the cyclic stress-based methods.

2.3 Cyclic stress-based approach

In this approach the earthquake induced loading is expressed in terms of cyclic shear stress, which is compared with the liquefaction resistance of soil expressed also in terms of cyclic shear stress. The location at which the loading exceeds the resistance of the soil liquefaction is expected to occur. The earthquake loading can be estimated in two ways:

1. by a detailed ground response analysis
2. by the simplified method as originally proposed by Seed and Idriss (1971) and its subsequent modifications.

The simplified methods are widely used than the first method. The uniform cyclic shear stress amplitude due to earthquake loading for level (or gently sloping) ground can be evaluated as per the simplified model developed by Seed and Idriss (1971), which is presented below.

$$\tau_{av} = 0.65 \frac{a_{max}}{g} \sigma_v r_d \qquad (3)$$

where:
- τ_{av} = the average equivalent uniform shear stress;
- σ_v = total vertical stress at the depth under consideration;
- a_{max} = the peak horizontal ground surface acceleration,
- g = acceleration due to gravity
- r_d = the value of a stress reduction factor at the depth of interest that accounts for the flexibility of soil column (e.g.; r_d = 1 corresponds to the rigid body behaviour) and can be presented as

$$r_d = \frac{(\tau_{max})_d}{(\tau_{max})_r} \qquad (4)$$

The $(\tau_{max})_d$ is the maximum shear stress on soil element considering it as a deformable body whereas, $(\tau_{max})_r$ is the maximum shear stress on soil element considering it as a rigid body. The factor 0.65 is used to convert the peak cyclic shear stress ratio to a cyclic stress ratio that is representative of the most significant cycles over the full duration of loading.

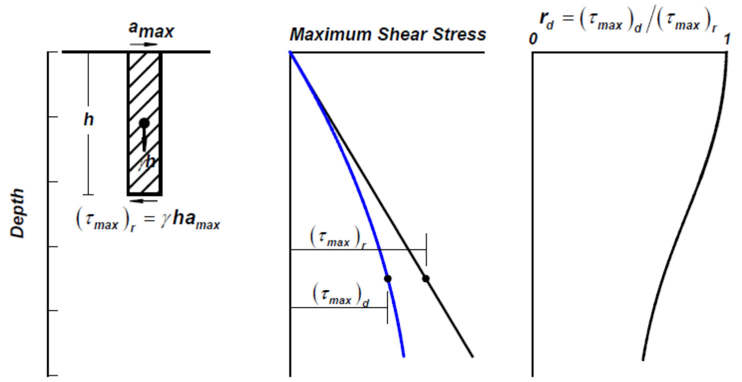

Figure 2-Schematic for determining maximum shear stress, τ_{max}, and the stress reduction coefficient r_d (Seed and Idris 1971).

The liquefaction resistance of an element of soil depends on how close the initial state of soil is to the state corresponding to "failure" and also the nature of loading required to move the soil element from the initial state to failure state. Cyclic stress based approach is widely used and two types of methods under this approach are available for assessing liquefaction potential. One is by means of laboratory testing of undisturbed samples, and the other is based on empirical relationships that relate the field behavior with the in-situ tests.

2.3.1 Laboratory test-based methods

Liquefaction resistance can be determined generally by two types of laboratory testing of undisturbed samples: (i) cyclic tri-axial test and (ii) cyclic simple shear test. In these test liquefaction failures is defined as the point at which initial liquefaction is reached or at which some limiting cyclic strain amplitude is reached. Laboratory tests show that number of loading cycles required to produce liquefaction failure decreases with increase of shear stress amplitude and with the decrease of density of soil. Cyclic strength is normalized by initial effective overburden pressure to produce cyclic stress ratio (CSR). For cyclic simple shear test CSR is taken as the ratio of cyclic shear stress to the initial vertical effective stress i.e. $(CSR)_{SS} = \tau_{cyc}/\sigma'_{V}$. For cyclic tri-axial test it is taken as the ratio of maximum cyclic shear stress to the initial effective confining pressure and can be given as $(CSR)tx = \sigma_{dc}/2\ \sigma'_{3C}$.
where σ_{dc} is cyclic deviator stress and σ'_{3c} is the effective confining pressure. The CSR of the above two tests are not equivalent as they impose quite different loading. The CSR values of both tests are related as $(CSR)_{ss} = c_r\ (CSR)_{tx}$, where c is a correction factor.
Seed and Lee (1966) defined initial liquefaction as the point at which the increase in pore pressure is equal to the initial effective confining pressure from their study of liquefaction of saturated sands during cyclic loading. Seed and Idriss (1967) developed an empirical procedure to evaluate the liquefaction potential of soil deposits by combining the development of pore water pressure obtained from laboratory results with the shear stress time history determined from the seismic response calculations. Seed et al. (1975) developed a model to determine the number of uniform stress cycles, N_{eq} (at an amplitude of 65% of the peak cyclic shear stress i.e, $t_{avg} = 0.65t_{max}$) that would produce an increase in pore pressure equivalent to that of irregular time history by applying weighting procedure to a set of shear stress time histories from the recorded strong ground motions. Ishihara and Koseki (1989) showed that when the plasticity indices were below 10 the fines have little effect on liquefaction resistance. Chern and Chang (1995) developed a mathematical model for the evaluation of liquefaction characters of soil subjected to earthquake induced cyclic loading based on cyclic triaxial test results. Using the developed model and commonly used physical properties of soil the cyclic shear strength, number of cycles required to cause liquefaction and generation of excess pore water pressure can be evaluated without resorting to the complex laboratory cyclic shear test. Bray and Sancio (2006) confirmed through cyclic testing of a wide range of soils, which were found to liquefy in Adapazari during the 1999 Kocaeli earthquake, that these fine-grained soils are susceptible to liquefaction. Gratchev et al. (2006) examined the validity of the plasticity index (PI) as a criterion for estimating the
liquefaction potential of clayey soils under cyclic loading. They found that an increase in PI decreased the soil potential to liquefy, and soil with PI>15 seemed to be non-liquefiable, a finding that is in agreement with the results of other researchers.
Though, evaluation of liquefaction potential based on laboratory test yields good results many engineers prefer to adopt the field performance correlation-based approach because of great difficulty and cost involved in obtaining undisturbed samples from cohesion-less soil deposits. Here in this study focus is on in-situ test-based available methods for liquefaction potential evaluation.

2.3.2 In-situ Test based methods:

Soil liquefaction potential can be determined by using in-situ tests such as:

1. standard penetration test (SPT)

2. cone penetration test (CPT)
3. shear wave velocity (V) measurement
4. Becker penetration test (BPT).

Due to difficulties in obtaining high quality undisturbed samples and subsequent high quality laboratory testing of granular soils, use of in-situ tests along with case histories- calibrated empirical relationships are generally resorted by the geotechnical engineers for the assessment of liquefaction potential of soils. The simplified procedure pioneered by Seed and Idris (1971) mostly depend on a boundary curve, which presents a limit state and separates liquefaction cases from the non-liquefaction cases basing on field observations of soil in earthquakes at the sites where in situ data are available. The boundary is usually drawn conservatively such that all cases in which liquefaction has been observed lie above it. In this approach the CSR is usually used as earthquake loading parameters and the cyclic resistance ratio (CRR) is represented by in-situ test parameters that reflect the density and pore pressure generation properties of soil. Out of the various in-situ methods as mentioned above SPT and CPT-based methods are widely used for liquefaction susceptibility analysis of soil.

SPT-based method

It is the most widely used methods among the available in-situ test methods as discussed above for evaluation of resistance of soil against the occurrence of liquefaction. Whitman (1971) first proposed to use liquefaction case histories to characterize liquefaction resistance in terms of measured in situ test parameters. Seed and Idriss (1971) did a pioneer work in developing a simplified empirical model, using laboratory tests and post liquefaction field observations in earthquakes, which presents a limit state function separating liquefied cases from the non-liquefied cases on the basis of SPT data. Seed et al. (1983) extended their previous work in developing a modified model in which used CSR (τ_{av}/σ'_v) instead of peak ground acceleration (a_{max}) as a measure of seismic action and overburden pressure corrected SPT value (N_1) instead of relative density (Dr) as the site parameter representing its resistance to liquefaction. However, it has been addressed by many researchers that the SPT has been conventionally conducted by using different kinds of hammers in different parts of the world, with different energy delivery systems, which also have varying degrees of efficiency. Moreover, the borehole diameters and the sampling techniques also differ significantly, which in turn cause a large variability in the measured values depending on the combinations of actual test procedures and equipment used.
Seed et al. (1985) expressed the measured penetration resistance (N_m) in terms of $N_{1,60}$ where the driving energy in the drill rod is considered to be 60% of the free fall energy and correction for overburden effect is applied. Liquefaction resistance curves for sands with different fines contents are proposed, which is considered to be more reliable than the previous curves expressed in terms of mean grain size. Cyclic stress ratio, CSR, as proposed by Seed and Idriss (1971) and its subsequent modifications in Seed et. al.(1983), Seed et al.(1985), Youd et al. (2001), is defined as the average cyclic shear stress, τ_{av}, developed on the horizontal surface of soil layers due to vertically propagating shear waves normalized by the initial vertical effective stress, σ'_v, to incorporate the increase in shear strength due to increase in effective stress and is presented as follows:

$$\mathrm{CSR} = \frac{\tau_{av}}{\sigma'_v} = 0.65\ \frac{a_{max}\sigma_v}{g\sigma'_v} r_d \qquad (5)$$

where σ'_v = effective vertical stress at the depth under consideration.
The value of CSR is corrected to an earthquake magnitude of 7.5, using the magnitude correction proposed by Seed et al. (1985). Seed et al.(1985) proposed a standard blow count N_{60} as given below

$$N_{60} = N_m\left(\frac{ER}{60}\%\right) \qquad (6)$$

where

- ER= percentage of the theoretical free-fall energy (i.e., estimated rod energy ratio expressed in percentage)
- N_m = measured SPT blow count corresponding to the ER.

The value of N_{60} is corrected to an effective stress of 100 kPa. Thus, the overburden stress and energy corrected SPT value, $N_{1,60}$ is obtained by using the following relation:

$$N_1 = C_N \times N_{60} \qquad (7)$$

where C_N is the effective stress correction factor and is calculated from the following relation:

$$C_N = \frac{2.2}{(1.2+\frac{\sigma'_v}{P_a})} \qquad (8)$$

where P_a = 1atm of pressure in the same units used for σ'_v.
The figure 3 is a graph of calculating CSR and corresponding $N_{1,60}$ data from sites where liquefaction was or was not observed following past earthquakes with magnitudes of approximately 7.5. Liquefaction and non liquefaction data were separated by Cyclic Resistance Ratio (CRR) curves. Curves were developed for granular soils with the fines content of 5% or less, 15%, and 35%.
Figure 3 is only applicable for magnitude of 7.5 earthquakes.
Juang et al. (2000) proposed an artificial neural network (ANN) -based CRR model based on SPT dataset and used Bayesian mapping function approach to relate factor of safety against the occurrence of liquefaction, F_s with probability of occurrence of liquefaction, P_L. Youd et al. (2001) published a summary paper of 1996 and 1998, NCEER workshop in which the updates and augmentations to the original "simplified procedure" of Seed and Idriss (1971); Seed et al.1983; and Seed et al (1985) for evaluation of liquefaction potential, are recommended using SPT-based methods and is still followed as the current state of the art on the subject of liquefaction potential evaluation. Cetin (2000) and Cetin et al. (2004) proposed new correlations for assessment of liquefaction triggering in soil.

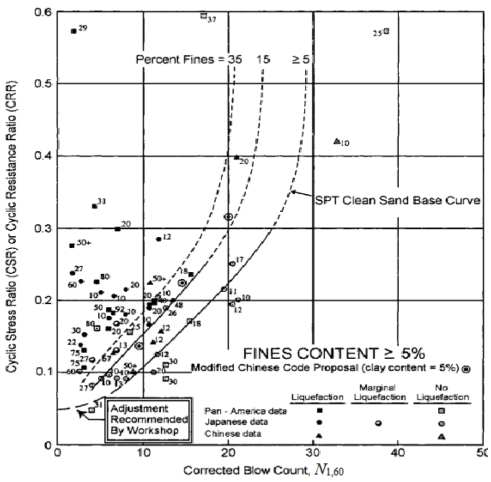

Figure 3-SPT –based limit state boundary curves for Magnitude 7.5 earthquakes with data from liquefaction stories (modified from Youd et al.2001)

These correlations are developed on the basis of an expanded and reassessed post liquefaction SPT database after making screening of field data case histories on a quality/uncertainty basis, incorporating

improved knowledge and understanding of factors affecting interpretation of SPT data, using improved understanding of factors affecting site specific earthquake ground motion, implementing improved methods for assessment of in situ CSR and using higher order probabilistic tools, Bayesian updating technique. The resulting correlations reduce the uncertainty associated with the liquefaction potential evaluation with respect to the existing models and also resolve controversial issues like magnitude-correlated duration weighting factors, adjustment of fines content and corrections for overburden stress in the context of assessment of CSR. Idriss and Boulanger (2004) and
Idriss and Boulanger (2006) re-examined the existing semi-empirical procedures for evaluating the liquefaction potential of saturated cohesion-less soils during earthquakes and recommended revised correlations for use in practice. In this paper the authors discussed about the parameters, which contribute to the CSR formulation like stress reduction factor, earthquake magnitude scaling factor, overburden correction factor, and also the overburden normalization factor for penetration resistances and presented the modified relations for these parameters.

CPT-based method

Although, the above SPT-based method remains an important tool for evaluating liquefaction resistance, it has some drawbacks, primarily due to the variable nature of the SPT (Robertson and Campanella, 1985; Skempton, 1986), nowadays the cone penetration test (CPT) is becoming more acceptable as it is consistent, repeatable and able to identify a continuous soil profile. Thus, CPT is being used as a valuable tool for assessing various soil properties, including liquefaction potential of soil. A typical CPT involves pushing a 35.7mm diameter conical penetrometer into the ground at a standard rate of 2cm/sec, while electronic transducers record (generally at 2cm or 5cm intervals) the force on the conical tip, the drag force on a short sleeve section behind the tip, pore water pressure behind the tip (or sometimes at other locations). The tip force is divided by the cross sectional area of the penetrometer to determine the tip resistance, q_c and the sleeve drag force divided by the sleeve surface area to determine the sleeve friction, f_s. The main advantages of the CPT are that it provides a continuous record of penetration resistance and is less vulnerable to operator error than the SPT. The main disadvantages of the CPT are the difficulty in penetrating layers that have gravels or very high penetration resistance and need to perform companion borings or soundings to obtain actual soil samples.
Zhou (1980) first published liquefaction correlation directly based on case history CPT database of the 1978 Tangshan earthquake. He presented the critical value of cone penetration resistance separating liquefiable from non-liquefiable conditions to a depth of 15m. Seed and Idriss (1981) as well as Douglas et al. (1981) proposed the use of correlations between the SPT and CPT to convert the available SPT-based charts for use with the CPT data. Robertson and Campanella (1985) developed a CPT- based method for evaluation of liquefaction potential, which is a conversion from SPT-based method using empirical correlation of SPT-CPT data and follows the same stress-based approach of Seed and Idriss
(1971). This method has been revised and updated by many researchers and need the knowledge of mean particle size (D_{50}) and fines content (FC) which cannot be obtained from CPT measurements alone. For determining D_{50} and FC additional boreholes are required for collecting samples. Ishihara (1993) suggested that in case of liquefaction resistance evaluated by using CPT value for silty sands (>5% fines), the effects of fines could be estimated by adding some tip resistance increments to the measured tip resistance to obtain an equivalent clean sand tip resistance.
For evaluating liquefaction potential only from CPT measurements, Olsen (1997) developed a CRR model using the parameters: qc, σ'_v and friction ratio (R_f). Robertson and Wride (1998) proposed a separate method using soil behaviour type index, I_c , which was recommended for use by the 1998, National Center for Earthquake Engineering Research (NCEER) workshop and is also presented in the summary paper of Youd et al. (2001).

Juang et al. (2003) also developed an ANN-based simplified method using soil type index (I_c) for evaluation of CRR of soil using post liquefaction CPT database and also used Bayesian mapping function approach to relate Fs with P_L.

Moss (2003) and Moss et al. (2005) presented a CPT-based probabilistic model for evaluation of liquefaction potential using reliability approach and a Bayesian updating technique. Juang et al. (2006) used first order reliability method (FORM) for probabilistic assessment of soil liquefaction potential.

Shear wave velocity (V_s)-based methods

The use of shear wave velocity (V_S)as a in-situ test index of liquefaction resistance of soil is very well accepted because both V_S and CRR are similar, but not proportional, influenced by void ratio, effective confining stresses, stress history, and geologic age.

The followings are the main advantages of using Vs for evaluation of liquefaction potential:

- V_S measurements are possible in soils that are difficult to penetrate with SPT and CPT or difficult to extract undisturbed samples, such as sandy and gravelly soils, and at sites where borings or soundings may not be permitted;
- V_S is a basic mechanical property of soil materials, directly related to small-strain shear modulus;
- the small-strain shear modulus is a parameter required in analytical procedures for estimating dynamic soil response and soil structure interaction analyses.

But, the following disadvantages are also there when V_s is used for liquefaction resistance evaluations:

- seismic wave velocity measurements are made at small strains, whereas pore-water pressure build up and the liquefaction triggering are medium- to high-strain phenomena;
- seismic testing does not provide samples for classification of soils and identification of non-liquefiable soft clay-rich soils;
- low V_s strata may not be detected if the measurement interval is too large.

Therefore, it is preferred to drill sufficient boreholes and conduct in-situ tests (SPT or CPT) to detect and demarcate thin liquefiable strata, non-liquefiable clay-rich soils, and silty soils above the ground water table that might become liquefiable should the water table rise. Few shear wave velocity - based methods have been developed and are in use. But as Vs method is of recent origin and has not been verified with the historical post liquefaction database, Vs – based method is not that popular like SPT and CPT–based method.

BPT-based methods

Liquefaction resistance of non-gravelly soils has been assessed mostly through SPT and CPT, with rare V_s measurements. Several investigators have employed large-diameter penetrometers to overcome these difficulties; the Becker penetration test (BPT) in particular has become one of the more effective and widely used larger tools. The BPT was developed in Canada in the late 1950s and consists of a168-mm diameter, 3-m-long double-walled casing driven into the ground with a double-acting diesel-driven pile hammer. The hammer impacts are applied at the top of the casing and the penetration is continuous. The Becker penetration resistance is defined as the number of blows required to drive the casing through an increment of 300 mm. The BPT has not been standardized, and several different types of equipment and procedures have been used. There is currently very few liquefaction sites from which BPT data have been obtained. Thus the BPT cannot be directly correlated with field behaviour, but rather through estimating equivalent SPT N_m values from BPT data and then applying evaluation procedures based on the SPT. This indirect method introduces substantial additional uncertainty into the calculate CRR.

Very few BPT-based simplified methods (Harder and Seed 1986 and Youd et al. 2001) have been developed primarily as it is only suitable for gravelly soil. A summary of in-situ techniques used for liquefaction potential assessment is provided in the table 1.

Features	Test type			
	SPT	CPT	Vs	BPT
Past measurements at liquefaction sites	Abundant	Abundant	Limited	Sparse
Type of stress-strain behaviour influencing test	Partially drained, large strain	Drained, large strain	Small strain	Partially drained, large strain
Quality control and repeatability	Poor to good	Very good	Good	Poor
Detection of variability of soil deposits	Good for closely spaced	Very good	Fair	Fair
Recommended soil type	Non-gravel	Non-gravel	All	Primarily gravel
Obtained soil sample	Yes	No	No	No
Measures index or engineering property	Index	Index	Engineering property	Index

Table 1-Comparisons of in-situ techniques for liquefaction potential evaluation (Youd et al., 2001)

2.4 Liquefaction ground deformations

Earthquake shaking may trigger the liquefaction of a saturated sandy soil in the ground. During past major earthquakes, enormous damage to engineered structures and lifelines has been caused by liquefaction-induced ground failures (e.g., Hamada and O'Rourke 1992). Generally, liquefaction-induced ground failures include flow slides, lateral spreads, ground settlements, ground oscillation, and sand boils. Several methods have been proposed to estimate liquefaction induced lateral ground displacements including numerical models, laboratory tests, and field-test-based methods. Challenges associated with sampling loose sandy soils limit the applications of numerical and laboratory testing approaches in routine practice. Field-test-based methods are likely best suited to provide simple direct methods to estimate liquefaction-induced ground deformations for low-to medium-risk projects and to provide preliminary estimates for high-risk projects.For estimating lateral displacements, empirical methods are used.

Methods using standard penetration test (SPT) data are available for estimating lateral displacements in a liquefaction-induced lateral spread. Even though the cone penetration test (CPT) has greater repeatability and reliability, and provides a continuous profile compared with other field tests, no CPT-based method to estimate liquefaction-induced lateral displacements is currently available.

The approach combines available SPT- or CPT-based methods to estimate liquefaction potential with laboratory test results for clean sand to estimate the potential maximum cyclic shear strains for saturated sandy soils under seismic loading.

Case history data are used to develop empirical correlations for lateral displacement for:

- gently sloping ground without a free face,
- level ground with a free face,
- gently sloping ground with a free face.

2.4.1 Lateral spreading

Liquefaction-induced lateral spreading is defined as the finite, lateral displacement of gently sloping ground as a result of pore pressure build-up or liquefaction in a shallow underlying deposit during an earthquake (Rauch, 1997). A three dimensional description of the lateral spreading is illustrated in Fig.4 (Varnes 1978, Rauch 1997).

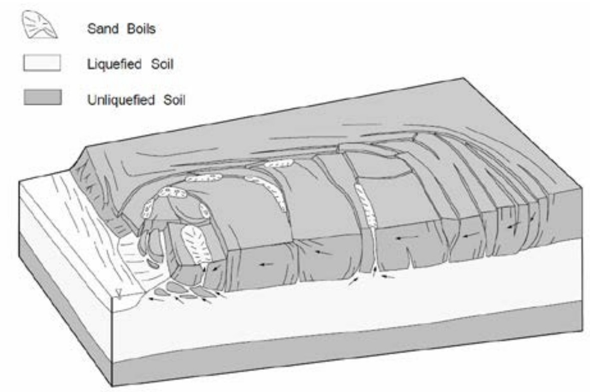

Figure 4-Schematic depiction of a lateral spreading resulting from liquefaction in an earthquake (after Rauch 1997, originally from Varnes 1978).

Fig. 5 shows two typical patterns of soil liquefaction and the induced lateral spreading. The geologic conditions for increased susceptibility to liquefaction-induced lateral spreading are: 1) shallow water table, 2) presence of unconsolidated loose sandy alluvium, typically Holocene in age; 3) strong ground shaking, and 4) constant initial shear stress resulting from a gently sloping ground. The first three conditions are the conditions required for liquefaction to occur. The last is the additional condition for liquefaction-induced lateral spreading to occur.

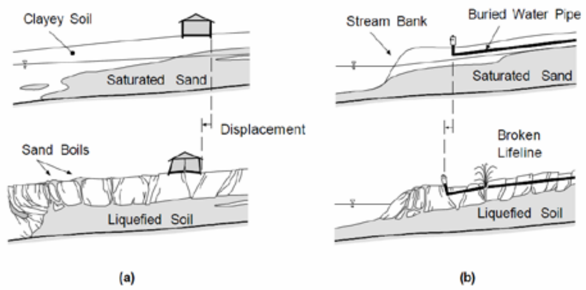

Figure 5-Soil liquefaction and lateral spreading of (a) gently sloping ground and b)toward a free face (after Rauch 1997).

Among liquefaction mechanism, lateral spreading can be more hazardous (Youd *et al.,* 2002). A number of approaches have been proposed for prediction of the magnitude of lateral ground displacements under different conditions. All of them can be categorized into Figure 6.

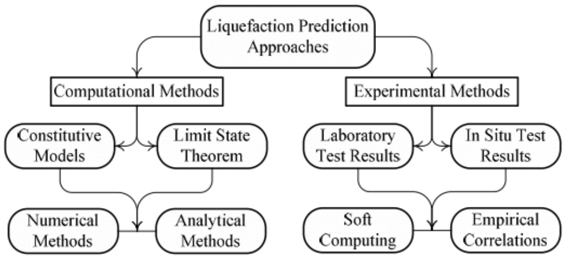

Figure 6-Schematic description of liquefaction-induced lateral ground spreading, and the associated destructive effects (After Rauch, 1997).

However, all predictions based on any of the above-mentioned approaches require determination of input parameters, which are prone to uncertainties and inaccuracies. The effect of any inaccuracies of input data in the numerical and analytical approach may be studied by a sensitivity analysis of the predictions on various input data. However, due to versatility, empirical and semi-empirical correlations remain at the centre of practice.

2.4.2 Flow Failure

The simple definition of flow type slide found after Varnes (1978) is: "When a stable mass of soil resting on sloped impervious started moving down like fluid due to its static liquefaction of the ingredient soil particles at its base of the strata then, it may be called as flow landslide".

After slide, the dump down sliding mass has no longer can retain its original shape or block of mass even for two particles together at a time. In general, flows are down slope movement of earth that resembles viscous fluid. They differ from general form of landslides in that sense there are no well-defined blocks moving along shear surfaces. Instead, the mass flows down hills with shear strains present everywhere. After the flow ceases, its products have a clearly fluidized appearance, as shown in Figure 7.

In terms of geotechnical expression found after Bayan,when the undrained steady state shear strength of a particular soil mass of the strata resting on an inclined impervious ground started losing its strength suddenly without any strain deformation and used to move down like a fluid flow then such type of landslide is called flow type landslide

It is recognised through geotechnical investigations that the particular soil layer which generates flow slide is known as colluvial soil mass only with presents of angular soil grains (Bayan, 2008 & 2010). The external force which makes it happen to be occurred is the heavy and long durational rainfall only. No over loading nor any dynamic forces like earthquake shocks can generate flow type landslide. Due to the action of sudden increase of water load among the voids of the soil particles of the colluvium strata pertaining to the discrete boundary, the soil mass gets saturated along the inter layer boundary and generates static liquefaction(Bayan, 2008). Static liquefaction in the soil mass is a local phenomenon in very small scale. However, the accumulated result of such innumerable static liquefactions yield large scale land mass to flow down as fluid flow, which normally use to term flowslide or flow type landslide. Flow failures occur in sloping areas inclined at 5 percent or greater.

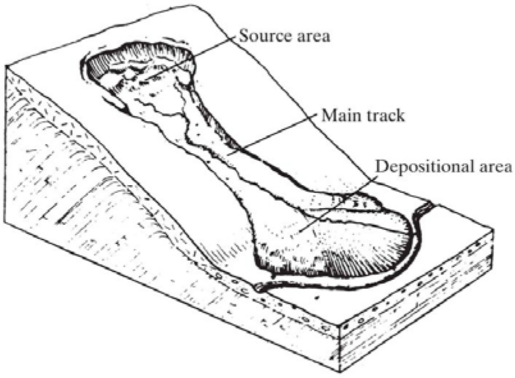

Figure 7-Flow failure (Varnes,1978).

Several flow landslides have occurred in the estuary section of the Dutch province of Zeeland, including some that have caused disastrous breaches in the dikes of that region. Dike slopes before failure were typically about 36 percent (20°), with banks as high as 40 m (130 ft). After failure, many slopes were 7 percent (4°) or less. Liquefaction of a loose granular layer underlying the slides areas was the apparent cause of the flow failures. A flow failure occurred in the Fort Peck Dam, Montana, during construction of a hydraulic-fill embankment. A 250 m (1,700 ft), long section of the upstream shell failed and moved 460 m (1,500 ft) upstream in about 4 min. Before failure, the slope of the upstream embankment was about 25 percent (14°); after failure slopes on the failed mass were generally less than 5 percent (3°). The cause of failure is believed to have been liquefaction of a sand zone in the shell and possibly a natural granular layer beneath the dam.

2.4.3 Vertical Settlement

Liquefaction-induced ground settlement, resulting from the dissipation of excess pore water pressure during earthquakes, is of great engineering significance. There are many mechanisms that can result in liquefaction-induced ground settlements, as shown in figure 8.

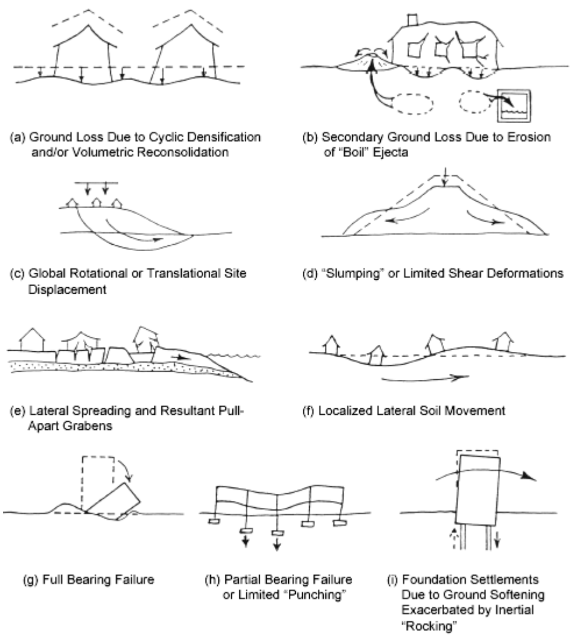

Figure 8-Schematic illustration of liquefaction-induced ground vertical displacement mechanism (Seed et al.,2001)

The settlement of the ground is considered to be one of the measures for evaluating the level of damage due to liquefaction. Thus, an assessment of the ground settlements, particularly for level grounds, is the first step to examining the various kinds of associated damage. Attempts to evaluate ground settlements were made by Tokimasu and Seed (1987), Ishihara and Yoshimine (1992), Tsukamoto et al. (2004), Shamoto and Zhang (1998) and Shamoto et al. (1998). Their methods, however, were based on limited data from field observations and laboratory tests. In recent times, significant progress has been made in recovering high-quality undisturbed samples of sands and testing them in the laboratory. In addition, a diversity of field case studies has been made for settlements by in-situ surveys of liquefied grounds at the time of the Great East Japan Earthquake in 2011.

2.4.4 Bearing capacity

When the soil supporting a structure liquefies and loses strength, large deformations can occur, leading to large settlements and/or tilting of structures. Loss of soil bearing capacity may also occur when liquefaction that has initially developed in a sand layer a few meters below a footing propagates upward through overlying sand layers and subsequently weakens the soil supporting the structure.

Isolated column footings, strip footings, mat footings, and even pile foundations all may fail during seismic events. The reason for the seismic settlements of these foundations seems to be that the bearing capacity was reduced. An example of such a failure occurred during the Miyagihen-Oki earthquake of magnitude 7.8 on June 12, 1978 northeast of Sendai (Japan), where the foundations of several oil-storage tanks suffered from bearing-capacity failure and excessive settlements (Okamoto 1978). The subsoil for the oil-storage tanks consisted of a fine sand 65 m thick that had been consolidated by vibrofloatation prior to the construction of the tanks. In the United States, the settlement at the Jensen Filtration Plant during the San Fernando earthquake is a well-documented example of large seismic settlement (about 100 mm) experienced by a compacted material (Whitman and Bielak 1980).

The seismic bearing capacity of shallow foundations resting on partially liquefiable sands did not receive the deserved attention in the literature.

Many researchers investigated the problem of the seismic bearing capacity of liquefiable sand overlain by a non-liquefiable clay crust. Naesgaard et al. (1998) assumed that punching shear failure will occur in the clay crust along with a wedge-type failure within the liquefiable sand layer, and developed an equation to calculate the factor of safety against bearing capacity failure. Cascone and Bouckovalas (1998) investigated the same problem, and proposed a formula to compute the degraded internal friction angle of the liquefiable sand due to excess pore pressure generation. Bouckovalas, Valsamis and Andrianopoulos (2005) suggested a modification to the expression of the degraded internal friction angle by Cascone and Bouckovalas (1998) to include the effect of the residual shear strength of the liquefied sand. Karamitros, Bouckovalas, Chaloulos and Andrianopoulos (2013) conducted an effective stress finite difference analysis to study the bearing capacity degradation of liquefiable sand overlain by a clay crust.

Karamitros et al. (2013) presented a relationship between the clay crust thickness and the footing width in order to avoid the hazard of soil liquefaction.

Day (2010) derived an expression to calculate the seismic bearing capacity of shallow foundations resting directly on partially liquefiable saturated sand. According to Day (2010), partially liquefiable sand corresponds to F in the range of 1.0–2.0. Day (2010) assumed that the likely mode of failure is unching shear. Hence, the q-term of the bearing capacity equation is omitted, as shown in equation

$$q_{ult} = \frac{1}{2} B \gamma_{sub} N_{\gamma} (1 - r_u) \qquad (9)$$

where B is the footing width, γ_{sub} is the buoyant unit weight of the soil beneath the footing, N_γ is the bearing capacity factor, which was suggested by Day (2010) to be taken equal to the static value, and r_u is the pore pressure ratio (the ratio between the excess pore pressure and the effective stress).
Day (2010) recommended determining the pore pressure ratio from the charts developed by Marcuson and Hynes (1990).
The most adverse consequence of partially liquefiable sands is the value of the permanent settlement after the end of an earthquake episode. Several researchers indicated that post-earthquake deformations are actually driven by static forces rather than dynamic forces (Finn, Yogendrakumar, Lo and Yoshida 1988; Law *et al.* 2005; Kourkoulis, Anastasopoulos, Gelagoti and Gazetas 2010; US Bureau of Reclamation, 2012).
USBR (2012) pointed out that many observations of embankment instability from seismic loading indicated that most of the deformation occurs after the end of the seismic episode.
Therefore, the permanent deformation of saturated sands should be calculated by redistributing the unbalanced stresses during the earthquake using a static numerical analysis.

3 Ground improvement methods used as remedial countermeasures

Efficient remediation of soils susceptible to liquefaction-induced damage is one of the most challenging problems in geotechnical earthquake engineering. The design guidance for liquefaction mitigation by ground improvement under certain earthquake conditions has been developed semi-empirically based on the in-situ observations of case histories, and physical and numerical testing results. Based on these studies, the improved performance is not always satisfactory but, in generally, fairly acceptable. However, in the majority of the analyzed case histories involving liquefaction mitigation by ground improvement, the design earthquake magnitudes were greater than the actual recorded magnitudes, such as the 1964 Niigata earthquake and the 1989 Loma Prieta earthquake. This may indicate that the seismic design could be unconservative.
For earthquake-induced soil liquefaction, based on the sequences of following several phenomena or failure mechanisms, excessively large cyclic loading occurs to in-situ soils which could cause the collapse of the soil skeleton, by which excess pore water pressure is generated without rapid and sufficient drainage, and this could lead to the reduction of effective stress and occurrence of soil liquefaction. Ground improvement as remedial measures can be implemented to prevent the above failure mechanisms or reduce their influence on the stability of the soil skeleton. Theoretically, any ground improvement methods that can impede the failure mechanisms can be used for liquefaction mitigation. According to NRC (1985), PHRI (1998), and DFI (2013), there are mainly four fundamental mitigation mechanisms usually involved:

1. densification,
2. reinforcement,
3. drainage
4. solidification

These methods are briefly discussed in the following sections.

3.1 Densification

As is well known, a soil's resistance to liquefaction is largely a function of relative density (D_r). Hence, improving soil relative density (increasing Cyclic Resistance Ratio "CRR" in the "simplified procedure") can be achieved by a substantial number of ground improvement methods (e.g., vibro-compaction, vibro-stone columns, dynamic compaction, compaction grouting, etc.).

Vibro-compaction methods such as sand compaction piles, have been used to increase the density of granular soils.
The use of vibro-replacement methods such as stone columns for densification, drainage and strengthening, are popular in the US.
Through densification, liquefaction will not occur or the induced deformation may be controlled during an earthquake. Densification is attractive because the methods are relatively simple and practical and improvement can be easily verified using in-situ penetration techniques.
The imparted energy changes the loose sand into a more dense state. The more dense soil has increased strength and bearing resistance and increased resistance to liquefaction. In cohesive soils increasing the density is accomplished through consolidation processes that remove water from the void spaces thus reducing the amount of settlement that will occur when loads are applied to the soil.
The design and constructions of densification primarily depends on fines content, elevation of ground water table and disturbance to adjacent structures.

3.2 Reinforcement

Inducing stiffer elements as shear reinforcement within a soil mass can reduce the cyclic shear stress ratio (CSR) applied to liquefiable soil and provide an improved axial stiffness. Soil reinforcement options include: full soil treatment (via permeation grouting, chemical grouting and bio-cementation), cellular or panel reinforcement (using jet grouting or slurry wall systems), or individual column elements (using jet grout columns, mechanically mixed columns, stone columns, aggregate piers, grout columns, etc.). Design of reinforcement is mainly based on the principle of strain compatibility between reinforcing elements and the enclosed soil even though its effectiveness could be significantly overestimated. Unlike densification methods for mitigation, the effectiveness of reinforcement cannot be verified based on post-treatment. Engineers must utilize theoretical analyses for their design and effectiveness verification. Comparing the individual columnar elements, underground wall panels or lattice shaped elements are more effective. In brief, numerical simulation in optimization of seismic design involving stiffer elements is more reliable than the semi-empirical methods.

3.3 Drainage

Generation of excess pore water pressure in liquefiable soil is the primary triggering mechanism leading to liquefaction. Hence, liquefaction can be mitigated and the reduction of effective stress in liquefiable soil under seismic excitation may be controlled if the development of high excess pore water pressure can be prevented through rapid drainage. The current design methods for drainage in practice are mainly semi-empirical or analytical (Seed and Booker, 1977; Pestana et al., 1997). The rule of thumb in most drainage designs for liquefaction is to find the installed drainage spacing to limit the excess pore water pressure ratio lower than 0.6 in soil to minimize deformation. Although drains can successfully mitigate liquefaction, the improvement of the ductility cannot be expected and the volume of water drained during seismic event is still approximately equal to the amount of deformation observed at the surface of drain treated ground (Iai, 1988). Therefore, the use of drainage is rarely relied on as the sole mechanism for mitigating liquefaction in the U.S. In addition, the effectiveness of an earthquake drain installation cannot be verified through conventional in-situ penetration techniques.

3.4 Solidification method

Increase in liquefaction resistance can also be obtained through a more stable skeleton of soil particles. This can be achieved by solidification methods, which involve filling the voids with cementitious materials resulting in the soil particles being bound together.
Typical methods include deep mixing and pre-mixing. The deep mixing method achieves liquefaction remediation by mixing a stabilising material such as cement in sandy soil and solidifying the soil. Such a method is applicable even for foundation ground beneath an existing structure. The pre-mixing method, on the other hand, involves adding stabilising material to the soil in advance and placing the treated soil at the site. This is recommended for constructing new reclaimed land. References on the application of these methods include Francis and Gorski (1998), Bruce (2000) and CDIT (2002).
Chemical grouting is a technique that transforms granular soils into sandstone-like masses, by permeation with a low viscosity grout. Typically, a sleeve port pipe is first grouted into a predrilled hole. The grout is injected under pressure through the ports located along the length of the pipe. The grout permeates the soil and solidifies it into a sandstone-like mass. The grouted soil has increased strength and stiffness, and reduced permeability.

4 Seismic designs of ground improvement measures

Seismic designs of ground improvement remedial measures can be highly diverse and case-specific. As a rule of thumb, the area to be improved to reduce the effects of liquefaction is the part of the ground which significantly affects the stability of structures (PHRI, 1998) and enough to cover the influence (i.e., migration of excess pore water pressure; liquid-like response) of surrounding unimproved zone. According to Haulser (2002), the critical influencing factors on improved ground performance are the improved zone depth and lateral extent. The majority of design guidelines are developed semi-empirically based on case histories and experimental testing data. Cooke and Mitchell (1999) and PHRI (1997) recommended that the improved depth should be equal to thickness of liquefiable soil depths determined by in-situ techniques by following the “simplified procedure”. PHRI (1997) also recommended extending the improvement depth through the full liquefiable soil zone thickness with the maximum improvement depth up to 20 m. In terms of the specifications on lateral extent of improved zone, Iai et al. (1988&1990) and Mitchell et al. (1995) recommended extending the improvement distance in the lateral direction equal to the depth of the soil zone being improved. PHRI (1997) recommended an improved distance greater than or equal to two-thirds of the improvement depth. For light or small structures, the lateral extent of the treatment zone should be at least one half of the improvement depth. The improvement zone width to depth should always be greater than 1.5 to prevent the migration of excess pore water pressure from surrounding unimproved zones and also keep the maximum pore water pressure ratio in the treated zone less than 0.5 (DFI, 2013).
In brief, an adequate improvement zone should provide sufficient protection and resistance to the influence exposed from surrounding unimproved zones..
The effectiveness of ground improvement for liquefaction mitigation has been assessed and interpreted based on:

- observations of the improved performances in case histories from previous earthquakes
- implementation of physical modeling (i.e., shanking table and centrifuge testing)
- numerical modeling to compare the unimproved and improved performance of structures through measured engineering parameters.

As indicated in PHRI (1997), if remedial measures are to be considered, the simplified method may not be sufficient and more advanced analytical or experimental methods are recommended.
Mitchell et al. (1995), Hayden et al. (1994) and PHRI (1997) assessed the great effectiveness of implementing ground improvement for liquefaction mitigation based on field observations from previous earthquakes. Their results showed the improved areas normally performed better and suffered less damage than the surrounding unimproved areas or structures.

The magnitudes of these seismic events were generally less than their ground improvement earthquake magnitudes, hence, the influence of shaking intensity and duration on improved performance remains uncertain. This drawback also highlights the importance of physical and numerical testing. Based on case histories collected in Japan, PHRI (1997) indicated the SPT $(N_1)_{60}$ value, required as part of QA/QC process, was most frequently used as an index for evaluating the effectiveness of improvement for the densification method. The target N-values are normally taken to be at 14 to 16. The maximum pore water pressure ratio employed as a target value for drainage method is commonly set at 0.5. For solidification or grouting methods, the unconfined compressive strengths are usually taken to be from 0.9 to 6.0 kgf/cm^2 (14.2 psi) for the mixed materials. All the above target values were determined based on the factor of safety against liquefaction occurrence. Therefore, the improvement effectiveness or design specifications of the improvement design are simply verified as part of QA/QC program in the field.

Yasuda et al. (1996) analyzed the effectiveness of several ground improvement methods based on the post-earthquake ground settlements observed in Kobe earthquake in 1995. Based on their results, sand compaction piles were regarded as the most effective, followed by densification methods which were more effective than drainage methods. The other evaluation criteria also included the in-situ SPT blow count measured before and after earthquake occurred.

The main conclusions include:

1. The average unacceptable performance rate for buildings with shallow foundations and piles was about 15% to 20%, which was slightly higher than tanks with shallow foundations and embankments.
2. Using sand compaction piles as the remedial measure, the unacceptable performance rate was 20%, which was slightly higher than the cases involving other remedial measures.
3. The unacceptable performance rate using vibro-compaction in this earthquake was surprisingly about 50%, which may indicate the seismic design method of the vibro-compaction method for liquefaction mitigation in Japan may be unconservative.
4. No clear trend between the building settlement and improved thickness was found based on the data obtained from this case history.

Another data source on improvement effectiveness evaluation is based on experimental testing, which include laboratory tests (i.e., cyclic triaxial tests, hollow cylinder torsional test, or cyclic simple shear test) on “undisturbed improved samples” to obtain the improved strength under cyclic loading or full scale tests such as shaking table and centrifuge test. The emphasis herein is on the full scale experiment tests because they can predict the improved performance in greater details and accuracy.

For the full scale experimental tests, extensive measurements (i.e., acceleration, excess pore water pressure generation within the soil mass, structure deformation, etc.) to discover the insights of the failure model and improvement mechanisms from physical testing can be collected and future used to optimize the seismic design of ground improvement to ensure satisfactory improved performance under the design earthquake. In general, centrifuge test can generally provide more accurate results than the shaking table test (Adalier, 1996; Haulser, 2002).

Adalier (1996) performed a series of centrifuge tests and shaking table tests to evaluate the effectiveness of various ground improvement methods in liquefiable soil.

The primary measurements included:

- time histories of acceleration response,
- time histories of excess PWP ratio,
- time histories of deformation,
- shear stress and strain distribution within the soil mass.

These data can be measured at numerous selected critical locations within the modeled structure-soil system without and with the implementation of ground improvement. Based on their results, the following conclusions were drawn:

1. Increasing over consolidation ratio of liquefiable soils can reduce excess pore water pressure generation and densification efforts.
2. The effectiveness of vacuum-suction for liquefaction mitigation was proved and although a reliable design has not yet been developed.
3. Densification methods were effective in mitigating liquefaction risk, and the island soil effects should be accounted in remediation design. Reinforcing effects of stiff grid shape metallic as stiffer elements may not be effective in eliminating the liquefaction potential.
4. For earth embankments, compaction and sheet piles can minimize excessive deformation and prevent catastrophic failure from occurring without completely eliminating the risk of liquefaction.

Like the experimental testing, well calibrated and verified numerical modeling also has great benefits in evaluating the improved performance and optimizing the remedial design. Three reasons explain why using numerical modeling for improvement effectiveness evaluation:

First, numerical modeling can account for influences of complex and simultaneous phenomena in seismic design. For instance, the improved part of the ground will be affected by migration of excess pore water pressure and liquid-like response of the liquefied part of ground surrounding the improved part. However, to evaluate this change and the effects on the stability of structures, which cannot be captured using simplified procedures, requires detailed and advanced investigation. Hence, well-calibrated numerical analyses can provide valuable insights on this issue.

Second, numerical modeling can capture multiple types of improved response data. Cooke and Mitchell (2000) conducted a comprehensive numerical study to determine the effectiveness of grouting methods on an existing highway bridge stub abutment supported on soil embankment underlying by liquefiable zone. Using an effective stress analysis technique in FLAC (Itasca, 2007), the effectiveness of three remedial measures (chemical grouting, jet grouting and compaction grouting) were examined quantitatively based on a series of improved numerical engineering parameters (reduction of excess pore water pressure within the liquefiable zone, the reduced displacement of the sub abutment and deformation of the soil embankment) under seismic excitation. Similar to physical modeling, the effectiveness evaluation and comparison based on numerical testing was based on the time histories of soil or structure deformations and/or responses under seismic excitation.

Third, numerical modeling can be used to examine different remedial measures, regardless of the improvement mechanisms. For instance, in-situ penetration techniques are unable to be used to verify the improvement effectiveness of soil mixing method, such as lattice or wall shaped mixed elements by grouting methods within liquefiable soil. Rourke et al. (1997), Namikawa et al. (2007) and Nguyen et al. (2012) evaluated the effectiveness of deep soil mixed elements in liquefiable soil under seismic loading. In these studies, the primary performance evaluation parameters and comparison criteria were

- unimproved and improved cyclic stress ratio “CSR”,
- dynamic performance (i.e., stress-strain correlations, failure models) of the induced stiffer elements,
- unimproved and improved shear strain ratio distribution,
- reduction of the peak shear stress ,
- excess pore water pressure ratio distribution in the enclosed soil.

Similar studies on columnar type remedial measures are also conducted by Green et al. (2008) and Olgun and Martin (2008).

As indicated by Mitchell (2008), with the increasing development and application of numerical modeling and computational power, implementation of appropriate and calibrated constitutive models should be encouraged in effectiveness evaluation and optimization of the liquefaction mitigation design. Beyond the effectiveness evaluation, there is still considerable room for future developments in perusing an accurate and efficient design framework for liquefaction mitigation by ground improvement. This is also the primary objective of this research.

5 Development of Liquefaction LRG software

The proper selection of ground improvement technology(s) under site and project-specific conditions is an important first step to ensure an adequate, sufficient and cost-efficient mitigation treatment plan. Inappropriate selection and application of a remedial technology may lead to unacceptable, structure damage, and/or high cost.

Liquefaction mitigation using ground improvement technologies has been widely applied and has proven to be effective in reducing liquefaction induced damage. Design and remedial practice is normally different from country to country due to local engineering practice. However, before developing an adequate, economical, and effective remedial design using ground improvement technologies, engineers always encounter the first key step, which is to select the proper technology(s) based on site or project-specific characteristics.

Previous studies, including Youd (1998) and Mitchell et al. (1995) indicated that the inappropriate use of ground improvement technologies could result in an unexpected high construction cost and unsatisfactory improved ground performance. The factors leading to the use of ground improvement technologies are diverse, reflecting both a number of “hard” concerns (including geotechnical, logistical, accessibility, environmental, cost, schedule, and performance factors) as well as a number of less tangible issues (including national issues, the degree of logistical, accessibility, environmental, cost, schedule and performance factors) as well as a number of less tangible issues.

The selection of suitable technology(s) to mitigate liquefaction potential is also based on integration of available engineering knowledge, experiences, technology related features, and site or project-specific features. Moreover, several of these factors, such as experience and site characteristics, cannot be evaluated and described quantitatively. Application of ground improvement is often a combination of engineering and art (Chu et al., 2009). Therefore, the primary challenge in development of the proposed selection system is to take those influencing factors into consideration in the selection process. The suitability of ground improvement technology subjected to various site- and project-specific characteristics are first evaluated based on information from case histories.

The proposed tool is an accessible guide, with the goal of providing an informative reference and summary of well-accepted rules from the literature for liquefaction mitigation using ground improvement technologies.

The ground improvement technology candidates involved in this system are all commonly used for liquefaction mitigation in practice, and are shown in the tab.2 The primary supplementary information supporting this technology selection system is the elimination process of ground improvement technologies. The first step to eliminate some ground improvement technologies is to define the limits of their applicability, splitting them if the site is in a free field conditions or if there are existing buildings or infrastructures. The other involved factors include depth of liquefiable zone, type of soil to be treated, foundation type, ground water table and potential failure type.

A generalized summary of the evaluation process for use with the technologies can be summarized as:

1. Identify project conditions which could require ground modification or geoconstruction technologies, such as projects that encounter:
    a) Poor ground conditions which will not provide adequate support for the transportation related structure. Poor ground conditions are typically characterized by soft or loose foundation soils, which, under load, would cause long-term vertical and/or lateral deformations, or cause construction or post-construction instability.
    b) Project constraints which require retaining walls or steep slopes. Pavement foundations which require improvement.
    c) Need for of a working platform or access road.
2. Identify or establish performance requirements.

3. Identify and assess any time, space or environmental constraints.
4. Assess the site conditions.
5. Assess project constraints.
6. Identify limitations on the use of ground modification technologies.
7. Consider alternatives to the use of ground modification technologies.

Free field conditions			**Existing buildings**		
Category	**Technologies**	**Applicability**	**Category**	**Technologies**	**Applicability**
Vertical Drains and Accelerated Consolidation	Earthquake drains	Compressible clays, saturated low strength clays	Grouting	Compaction Grouting	Cohesionless granular soils, collapsible soils, and unsaturated fine grained soils; may be used to fill voids in sinkholes or abandoned mine shafts; can arrest settlement under a structure and lift foundations that have settled
Deep Compaction	Deep Dynamic Compaction	Loose pervious and semi-pervious soils with fines contents less than 15%, materials containing large voids, spoils and waste areas		Jet Grouting	Wide range of soil types and groundwater conditions
	Vibro Compaction	Cohesionless soils, clean sands with less than 15% silts and/or less than 2% clay	Soil Mixing	Deep soil mixing	Suitable in large range of soils, ones that can be stabilized with cement, lime, slag, or other binders
	Blasting Compaction	Broad applicability; no geologic or geometric limitations			
Aggregate Columns	Vibro replacements	Clays, silts, loose silty sands, and uncompacted fill			

Table 2-General applicability of technologies

The suitability of a technology subjected to different evaluation criteria or an “elimination engine” is discussed in the following sections.

5.1 Evaluation criteria

5.1.1 Failure type

In most cases, more than one type of failure can occur; therefore, the primary failure type is defined as the one with largest magnitude. The applied remedial technology is also not unique; the successful liquefaction mitigation can be achieved using a different mechanism provided by different types of remedial methods.

Technologies	**Failure type**			
	Lateral spreading	Flow failure	Vertical settlement	Bearing capacity
Earthquake drains	✓	✓	✓	
Deep Dynamic Compaction	✓	✓*		✓
Vibro Compaction	✓	✓		✓
Blasting Compaction			✓	
Vibro replacements	✓	✓	✓	✓
Compaction Grouting	✓	✓*		✓
Jet Grouting	✓	✓		✓
Deep soil mixing	✓	✓		✓

Table 3-Ground improvement technologies for different failure types

5.1.2 Soil type

Fully or partially saturated loose cohesionless and slightly cohesive soils are susceptible to liquefaction in earthquake events. The Unified Soil Classification System (USCS) has drawn up a soil classification based on identifying soils according to their textural and plasticity qualities and on their grouping with respect to behaviour. Soils seldom exist in nature separately as sand, gravel, or any other single component. They are usually found as mixtures with varying proportions of particles of different sizes; each component part contributes its characteristics to the soil mixture. The USCS is based on those characteristics of the soil that indicate how it will behave as an engineering construction material. The following properties have been found most useful for this purpose and form the basis of soil identification. They can be determined by simple tests and, with experience, can be estimated with some accuracy.

- Percentages of gravel, sand, and fines (fraction passing the No. 200 sieve).
- Shape of the grain-size-distribution curve.
- Plasticity and compressibility characteristics. In the USCS, the soil is given a descriptive name and a letter symbol indicating its principal characteristics.

In the first column of the table 4, soils are primarily identified as coarse grained, fine grained, and highly organic. On a textural basis, coarse-grained soils are those that have 50 percent or more by weight of the overall soil sample retained on the No. 200 sieve; fine-grained soils are those that have more than 50 percent by weight passing the No. 200 sieve. Highly-organic soils are, in general, readily identified by visual examination. The coarse-grained soils are subdivided into gravel and gravelly soils (G) and sands and sandy soils (S). Fine-grained soils are subdivided on the basis of their LL and plasticity properties; the symbol L is used for soils with LLs of 50 and less and the symbol H for soils with LLs in excess of 50. Peat and other highly organic soils are designated by the symbol Pt and are not

[*] The indicated methods could be used to prevent flow failure secondary.

subdivided. In general practice there is no clear-cut boundary between gravelly soils and sandy soils and, as far as behaviour is concerned, the exact point of division is relatively unimportant. For identification purposes, coarse-grained soils are classified as G if the greater percentage of the coarse fraction (that which is retained on the No. 200 sieve) is larger than the No. 4 sieve. They are classed as S if the greater portion of the coarse fraction is finer than the No. 4 sieve. Borderline cases may be classified as belonging to both groups. The G and S groups are each divided into four secondary groups as follows:

- Well-graded material with little or no fines—symbol W, groups GW and SW.
- Poorly graded material with little or no fines—symbol P, groups GP and SP.
- Coarse material with non plastic fines or fines with low plasticity— symbol M, groups GM and SM.
- Coarse material with plastic fines—symbol C, groups GC and SC.

The fine-grained soils are subdivided into groups based on whether they have a relatively low (L) or high (H) LL. These two groups are further subdivided as follows:

- Inorganic silts and very fine sandy soils, silty or clayey fine sands, micaceous and diatomaceous soils, and elastic silts—symbol M, groups ML and MH.
- Inorganic clays—symbol C, groups CL and CH.
- Organic silts and clays—symbol O, groups OL and OH.

COARSE-GRAINED SOILS More than half the material (by weight) is individual grains visible to the naked eye	GRAVELLY SOILS More than half of coarse fraction is larger than 4.75 mm	CLEAN GRAVELS Will not leave a stain on a wet pal	Substantial amounts of all grain particle size	GW
			Predominantly one size or range of sizes with some intermediate sizes missing	GP
		DIRTY GRAVELS Will leave a stain on a wet palm	Non-plastic fines (to identify, see ML below)	GM
			Plastic fines (to identify, see CL below)	GC
	SANDY SOILS More than half of coarse fraction is smaller than 4.75 mm	CLEAN SANDS Will not leave a stain on a wet palm	Wide range in grain size and substantial amounts of all grain particle sizes.	SW
			Predominantly one size or a range of sizes with some intermediate sizes missing	SP
		DIRTY SANDS Will leave a stain on a wet palm	Non-plastic fines (to identify, see ML below)	SM
			Plastic fines (to identify, see CL below)	SC

FINE-GRAINED SOILS More than half the material (by weight) is individual grains	**Ribbon**	**Liquid Limit**	**Dry Crushing Strength**	**Dilatancy Reaction**	**Toughness**	**Stickness**	
	None	<50	None to slight	Rapid	Low	None	ML
	Weak	<50	Medium to high	None to very slow	Medium to high	Medium	CL
	Strong	>50	Slight to Medium	Slow to none	Medium	Low	MH

not visible to the naked eye (<0.074 mm)	Very strong	>50	High to very high	None	High	Very high	CH
HIGHLY ORGANIC SOILS	Readily identified by colour, odour, spongy feel and frequently by fibrous texture						OL OH Pt

Table 4-Summary of field identification tests

Table 5 summarizes the liquefaction potential of various types of soil subjected to earthquake loading. In general, any ground improvement technologies that can effectively improve the shear and compression resistance of liquefiable soil can be used for liquefaction mitigation, but each remedial technology has its own suitable soil type to which it should be applied. Based on well-proven guidelines about the applications of ground improvement technologies in liquefiable soils (e.g. Mitchell et al., 1995; Mitchell, 2008; Chu et al., 2009; Mitchell, 2013), the suitability of ground improvement technologies subjected to various types of liquefiable soils is evaluated and presented in table below.

Technologies	Suitable soil type	Symbol (USCS)
Earthquake drains	Most soils susceptible to liquefaction	G-, S-, ML-
Deep Dynamic Compaction	Saturated sands; silty sands; partly saturated materials	S-
Vibro Compaction	Sands, silty sands, clayey silts, gravelly sands with fines < 20%, or combined with vertical drains	G-, S-
Blasting Compaction	Saturated clean sands and gravels, granular soils	G-, S-
Vibro replacements	Silty sands, clayey silts, or combined with vertical drains	S-, ML-
Compaction Grouting	Any rapidly consolidating, compressible soils	G-, S-
Jet Grouting	Most soils susceptible to liquefaction	G-, S-
Deep soil mixing	Most soils susceptible to liquefaction	G-, S-

Table 5-Ground improvement technology for liquefaction mitigation for various soil types.

In the table below, it is developed a sort of matrix to eliminate or filter the technologies less-applicable than others in terms of soil type.

Technologies	Soil type		
	G	S	ML
Earthquake drains	✓	✓	✓
Deep Dynamic Compaction		✓	
Vibro Compaction	✓	✓	
Blasting Compaction	✓	✓	
Vibro replacements		✓	✓
Compaction Grouting	✓	✓	
Jet Grouting	✓	✓	
Deep soil mixing	✓	✓	

Table 6-Evaluation criteria of soil type.

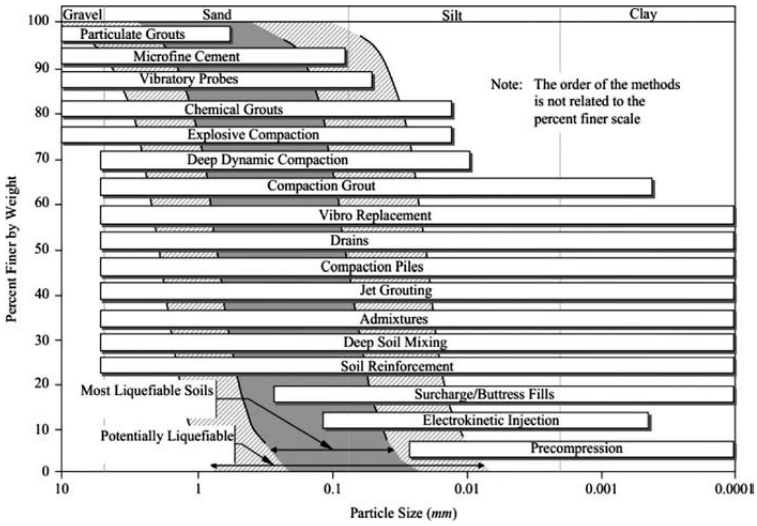

Figure 9-Applicable grain size ranges for liquefiable soil improvement methods (Mitchell and Gallagher 1998).

5.1.3 Depth of the groundwater table

Fully or partially loose saturated soils in shallow depth (less than 12 to 15 m or 40 to 50 ft) are always an important indication of liquefaction in a seismic region. Also, a high ground water table can critically influence the improvement effectiveness of ground improvement technologies. The records on depth of the ground water table in case histories are insufficient to make a general conclusion on the influence of the water table on technologies selection. Based on well-accepted rules found in the literature (e.g., PHRI, 1998; Towhata, 2006; Mitchell, 2013),the suitability evaluation of ground improvement technologies subjected to various depths of ground water table is presented in Table 7.

Technologies	**Depth of groundwater table [m]**			
	0-3	3-6	6-12	>12
Earthquake drains	✓	✓	✓	✓
Deep Dynamic Compaction		✓	✓	✓
Vibro Compaction			✓	✓
Blasting Compaction	✓	✓	✓	
Vibro replacements	✓	✓	✓	✓
Compaction Grouting	✓	✓	✓	✓
Jet Grouting	✓	✓	✓	✓
Deep soil mixing	✓	✓	✓	

Table 7-Suitability evaluation of technologies subjected to various ground water depths.

5.1.4 Depth of the treatment zone

Youd et al. (2001) indicated that liquefaction is not likely to occur when soil depth exceeds up to 25 m (80 ft) based on extensive review on past earthquakes. Several studies have empirically specified the minimum improved depth: (1) Mitchell et al. (1995) recommend improving the full thickness of liquefiable materials beneath the structures; (2) PHRI (1997) recommends a minimum improved depth of 15 m (50 ft) or up to the bottom of liquefiable layer encountered. Therefore, the specified improved depth is primarily governed by the maximum value of :

1. foundation depth,
2. the critical soil depth influencing foundation seismic performance,
3. the depth of liquefiable layer.

Technologies	Depth of treatment zone [m]					
	< 3	3-12	12-18	18-25	25-40	> 40
Earthquake drains					✓	✓
Deep Dynamic Compaction	✓	✓				
Vibro Compaction		✓	✓	✓	✓	✓
Blasting Compaction						✓
Vibro replacements		✓	✓	✓	✓	✓
Compaction Grouting	✓	✓	✓	✓	✓	✓
Jet Grouting	✓	✓	✓	✓	✓	✓
Deep soil mixing	✓	✓	✓	✓		

Table 8-Effective improved depth of ground improvement technologies in selection system.

Table 8 can be used as a screening criterion to eliminate the technologies if their effective treatment depths are less than the specified depth of liquefiable soil depth based on user's input. Since the effective improved depth could vary significantly under different cases, the lower bound values based on literature review are used for conservative purposes. Finally, the suitability of ground improvement technologies subjected to various liquefiable soil depths is determined.

5.1.5 Size of area

In some cases it is not possible to define a range, because the size of treated area could be influenced by other factors.
For the blasting compaction, it needs to know the size of the charge, which is related to the nature of the material and the depth of burial of the charge.
Dynamic compaction is generally completed over an area larger than the plan area of the embankment or the loaded area. This is to induce densification of the below ground area that will be subjected to stress increase due to the pressure distribution resulting from the new loading. On many projects, dynamic compaction is undertaken beyond the edge of the loaded area for a distance equal to the depth of the weak deposit.
The limits proposed in the below table to define the economic size of treated area have been established in accordance to the case histories found in literature.
As seen, different ratings are applied to describe the suitability of a certain technology subjected to different conditions:

1. Small area ("S"): indicates an area less than 1000 m^2
2. Medium area ("M"): indicates an area between 1000 and 5000 m^2
3. High area ("H"): indicates an area more than 5000 m^2

Technologies	Economic size of the treated area [m]		
	S	M	H
Earthquake drains	✓	✓	✓
Deep Dynamic Compaction	✓		
Vibro Compaction			✓
Blasting Compaction		✓	✓
Vibro replacements	✓	✓	✓
Compaction Grouting	✓	✓	
Jet Grouting		✓	✓
Deep soil mixing	✓		

Table 9-Economic size of area of ground improvement technologies.

5.1.6 Site conditions

Sometimes the site could be characterized by project constraints such as traffic flow/interruption and congestion, weather and environmental factors, traffic patterns and sensitive equipments.
For project constraints, it means, also, the construction influences or if some methods could damage the adjacent structures.
As criteria to select a ground improvement technologies based on the project constraints criteria, it is possible to identify different cases:

1) Low overhead clearance ("Case 1") means the not accessibility of the equipment to reach the site.
2) Adjacent structures ("Case 2") means that it is possible to use some technologies even if there are structures near the improved area. Adjacent buildings and structures, sometimes, must be monitored when using some technologies.
3) Existing utilities ("Case 3") means that a technology may be acceptable if there are some existing utilities. Sometimes it is necessary to show on the drawings these utilities because they could affect the operations.

Environmental constraints may include the disposal of spoils from the particular ground modification technology, the disposal of waste materials encountered on the site, protection of the site from erosion, protection of surface and ground waters from pollution, and the effects of construction vibrations, noise and dust.
Site conditions means, also, assessment of the subsurface conditions. The level of detail regarding the assessment of subsurface conditions will vary significantly across the wide range of transportation related projects and the type of ground modification selected.
Regardless of the project type, the soils which will affect the performance requirements must be identified and the necessary engineering properties established to perform a preliminary design for the project. At a minimum, the type, depth, and extent of needed treatment must be determined, as well as the location of the groundwater table.
For sites with poor ground conditions, it is also valuable to have at least a preliminary assessment of the shear strength, compressibility, and organic content of the identified poor soils. Additionally, assessment of subsurface obstructions in terms of cobbles, boulders, or construction debris, water bearing sands, organic layers, and very stiff surface deposits can affect the selection of appropriate technologies. The availability of materials for construction such as sand, gravel, and water are also important site considerations.

Technologies	**Subsurface obstructions**		**Project constraints**		
	Yes	No	Case 1	Case 2	Case 3
Earthquake drains		✓		✓	✓
Deep Dynamic Compaction		✓			✓
Vibro Compaction		✓		✓	
Blasting Compaction	✓				✓
Vibro replacements		✓		✓	
Compaction Grouting		✓	✓	✓	✓
Jet Grouting	✓		✓	✓	✓
Deep soil mixing		✓	✓		✓

Table 10-Ground improvement for different project constrains.

5.1.7 Foundation type

The selection of foundation type allows sorting of technologies based on the usefulness of the geoconstruction technology to the specific foundation type. However, no well-accepted rules have been established on this issue. Densification methods have been widely used for different foundation types; reinforcing column methods are more popular for deep foundations; dewatering or drainage methods are always used together with other technologies.

Technologies	Foundation type	
	Shallow foundations	Deep foundations
Earthquake drains		✓
Deep Dynamic Compaction	✓	
Vibro Compaction		✓
Blasting Compaction		✓
Vibro replacements	✓	
Compaction Grouting		✓
Jet Grouting		✓
Deep soil mixing		✓

Table 11-Ground improvement technologies for different foundations

5.1.8 Environmental regulatory requirements

Some geoconstruction technologies such as deep mixing method or grouting methods can improve the in-situ ground by introducing chemicals or contaminates into the soils, which can be a critical environmental issues in some cases.

6 Cost analysis

The purpose of this section is to present an overall approach that may be used to develop a cost estimate for any ground modification project. The cost estimate method described is similar to that commonly used in engineering to develop a conceptual cost estimate. That is, sufficient detail is used to identify the cost components which may have the largest effect on total project cost, however specific bid quantities and potential bidders are not necessarily known.

The method requires that a baseline cost for the particular ground modification technology be known. Over the past several decades, numerous ground modification projects have been completed in the United States and typical costs have varied during this time period primarily due to the business cycle, the experience of specialty ground modification contractors, advances in ground modification equipment, and the amount of competition from more conventional solutions. Due to the wide variety in types of ground modification technologies, there is no typical "average" price for "ground modification." Prior to describing a suggested cost estimating method for ground modification technologies, a discussion on the key factors which affect ground improvement costs is presented.

The primary cost items for any ground modification or geo-construction technology consist of costs for mobilization, materials, labor, and the equipment for the particular technology.

Added to this must be the cost of quality assurance including instrumentation and/or load tests that might be conducted as part of the QA process. There are many factors which can affect the cost of a specific project including project type, application, geoconstruction technology, soil type, labor rates, utility conflicts, location, weather, competition, etc.

Identifying and understanding how these variables impact cost can be beneficial when evaluating the applicability of a ground modification or geo-construction solution.

Often the costs of the primary cost items described above are rolled into prices that are quoted in lineal feet or square feet of installed product.

Technologies	Unit Cost
Earthquake drains	1.5-11.4 €/m
Deep Dynamic Compaction	10-30.5 €/m^2
Vibro Compaction	14-25.7 €/m
Vibro replacements	43-170€/m
Compaction Grouting	84.2-843/m^3
Jet Grouting	80-200 €/m^3
Deep soil mixing	170-357/m

Table 12-Comparative unit costs by ground modification technologies

Note that mobilization and demobilization of equipment to the project site is generally a lump sum item and can vary from a few hundred dollars to more than $100,000 depending on the technology. The costs for the original site investigation, quality assurance testing, and instrumentation are not included in the technology unit prices or mobilization costs.

Two or more technologies may be identified which are potential technical solutions; when this occurs, engineers typically consider the initial cost of a solution as part of the selection process. It is important to note that while initial cost is a consideration when selecting a solution, it should not be the driving force; performance, construction time, life cycle costs, risks and safety should be factored into the evaluation of alternative geotechnical solutions.

There are many factors which can affect cost for a specific project (i.e. soil type, labor rates,utility conflicts, etc.); identifying and understanding how these variables impact cost can be beneficial when evaluating the applicability of a geotechnical solution.

To develop a preliminary cost estimate for the use of a ground modification technology information about the site conditions must be known. This information also is necessary for the preliminary design of the particular ground modification technique. This preliminary design information can then be used to develop specific quantities of materials, equipment, etc. that can used to compute a preliminary cost estimate.

7 Example interface of the proposed technology selection system

The process to identify potential liquefiable ground conditions, the need for ground modification and the selection of appropriate technologies have to use a logical sequence.

The steps involved are summed up in forms of questions to help the implementation of this tool in the software and with the goal of making geotechnical solutions more accessible to the users.

To make it easier, the technologies are identified with a code shown in the table below.

Technologies	Tech ID
Earthquake drains	T1
Deep Dynamic Compaction	T2
Vibro Compaction	T3
Blasting Compaction	T4
Vibro replacements	T5
Compaction Grouting	T6
Jet Grouting	T7
Deep soil mixing	T8

Table 13-ID for different technologies

Q1. Which is the primary condition of the site?

❓*Selection of site characteristics allows for a distinction in site requirements to identify viable geoconstruction technologies. Few practical methods are currently available for liquefaction mitigation beneath existing structures.*

- R1.1. Free field
    1) T1; T2; T3; T4; T5; T6; T7; T8
- R1.2. Existing buildings
    2) T6; T7; T8

Q2. Which types of failure do you want to prevent?

❓*Understanding of the primary failure mechanism is important to select the effective mitigation mechanism, which can be achieved by different technologies. However, no guidelines or well-accepted empirical rules have been established on this issue.*

Select at least one.

- R2.1. Flow failure
    1) T1; T2; T3; T5; T6; T7; T8
- R2.2. Lateral spreading
    2) T1; T2; T3; T5; T6; T7; T8
- R2.3. Vertical settlement
    3) T1; T4; T5
- R2.4. Bearing Capacity
    4) T2;T3;T5;T6;T7;T8
- R2.5. Flow failure; Lateral spreading
    5) T1; T2; T3; T5; T6; T7; T8
- R2.6. Flow failure; Vertical settlement
    6) T1;T5
- R2.7. Flow failure; Bearing capacity
    7) T2;T3;T5;T6;T7;T8
- R2.8. Lateral spreading; Vertical settlement
    8) T1;T5
- R2.9. Lateral spreading; Bearing capacity
    9) T2;T3;T5;T6;T7;T8
- R2.10. Vertical settlement; Bearing capacity
    10) T4;T5
- R2.11. Flow failure; Lateral spreading; Vertical settlement; Bearing capacity
    11) T5

Q3. Project constraints (e.g., construction influences/damage to the adjacent structures, site access, traffic patterns, mobilization, sensitive equipment):

❓*If present, select the project constraint to sort the suitable technologies.*

You may leave all checkboxes blank if none of the listed constraints apply.

- R3.1. Low overhead clearance
    1) T6; T7; T8
- R3.2. Adjacent structures
    2) T1;T3; T5; T6; T7
- R3.3. Existing utilities
    3) T1; T2; T4; T6; T7; T8

- R3.4. Low overhead clearance; Adjacent structures
    4) T6; T7
- R3.5. Low overhead clearance; Existing utilities
    5) T6; T7; T8
- R3.6. Adjacent structures; Existing utilities
    6) T1; T6; T7
- R3.7. Low overhead clearance; Adjacent structures; Existing utilities
    7) T6; T7

Q4. Depth of groundwater table

Sometimes if there is a high water table, the technologies could not be applicable. In practice, ground water table should be at least 1-2 m lower than the treated depth improved by applied technology. Otherwise, dewatering action is required.

- R4.1. 0-3 m
    1) T1; T4; T5; T6; T7; T8
- R4.2. 3-6 m
    2) T1; T2; T4; T5; T6; T7; T8
- R4.3. 6-12 m
    3) T1; T2; T3; T4; T5; T6; T7; T8
- R4.4. >12 m
    4) T1; T2; T3; T5; T6; T7

Q5. Which is the treated soil type?

Some types of soil are susceptible to liquefaction as fully or partially saturated loose cohesionless and slightly cohesive soils. The improvement effectiveness is influenced by the treated soil characteristics such as fines content, density, and Atterberg limit. The Unified Soil Classification System (USCS) has drawn up a soil classification based on identifying soils according to their textural and plasticity qualities and on their grouping with respect to behaviour.

- R5.1. G – Gravel soils (more than half of coarse fraction is larger than 4.75 mm)
    1) T1; T3; T4; T6; T7; T8
- R5.2. S – Sandy soils (more than half of coarse fraction is smaller than 4.75 mm)
    2) T1; T2; T3; T4; T5; T6; T7; T8
- R5.3. ML – Inorganic silts, clays silts of low to medium plasticity
    3) T1; T5

Q6. Are present any subsurface obstructions?

"Yes" will remove technologies which have difficulties penetrating subsurface obstructions.

- R6.1. Yes
    1) T4; T7
- R6.2. No
    2) T1; T2; T3; T5; T6; T8

Q7. Depth of the treatment zone based on case histories

The effective treatment depths should be greater than the depth of liquefiable soil layer to be mitigated. Soils with depth greater than 12m normally tend to not liquefy.

- R7.1. <3 m
    1) T2; T6; T7; T8
- R7.2. 3-12 m
    2) T2; T3; T5; T6; T7; T8
- R7.3. 12-18 m

3) T3; T5; T6; T7; T8
- R7.4. 18-25 m
   4) T3; T5; T6; T7; T8
- R7.5. 25-40 m
   5) T1; T3; T5; T6; T7
- R7.6. >40 m
   6) T1; T3; T4; T5; T6; T7

Q8. Size of area to be improved

❓*Some technologies are more economical and practical for small areas while other technologies are more economical and practical for larger one.*

- R8.1. Small (area smaller than 1000 m^2)
   1) T1; T2; T5; T6; T8
- R8.2. Medium (area between 1000 m^2 and 5000 m^2)
   2) T1; T4; T5; T6; T7
- R8.3. High (area larger than 5000 m^2)
   3) T1; T3; T4; T5; T7

Q9. Select the improved foundation type

❓*Densification methods have been widely used for different foundation types; reinforcing column methods are more popular for deep foundations; dewatering or drainage methods are always used together with other technologies. No well-accepted rules have been established on this issue.*

- R9.1. Shallow foundations
   1) T2; T5
- R9.2. Deep foundations
   2) T1; T3; T4; T6; T7; T8

Q10. Are there any environmental issues that may affect the project?

❓*Some technologies such as deep mixing method or grouting methods can improve the in-situ ground by introducing chemicals or contaminates into the soils, which can be a critical environmental issues in some cases. This selection removes the environmentally-unfriendly technologies from the list.*

- R10.1. Yes
   1) T1; T2; T3; T4; T5
- R10.2. No
   2) T1; T2; T3; T4; T5; T6; T7; T8

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NORSAR

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Appendix B

LIQUEFACT Deliverable 6: The LIQUEFACT LRG toolbox

DRAFT REPORT - SUBJECT TO CHANGE

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LIQUEFACT

Assessment and mitigation of liquefaction potential across Europe: a holistic approach to protect structures/infrastructure for improved resilience to earthquake-induced liquefaction disasters

H2020-DRA-2015

GA no. 700748

DELIVERABLE D6.1

LRG Software Toolbox for Liquefaction Mitigation Planning and Decision Support

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UNIPV	Universita degli Studi di Pavia	Italy
UPORTO	Universidade do Porto	Portugal
UNINA	Universita degli Studi di Napoli Federico II.	Italy
ULJ	Univerza v Ljubljani	Slovenia
UNICAS	Universita degli Studi di Cassino e del Lazio Meridionale	Italy
Istan-Uni	Istanbul Universitesi	Turkey

GLOSSARY

Acronym	Description
LRG	LIQUEFACT Reference Guide
EILDs	Earthquake-Induced Liquefaction Disaster

This project has received funding from the European Union's Horizon 2020 research and innovation programme under grant agreement No. 700748

LSN	Liquefaction Severity Number
LPI	Liquefaction Potential Index
LP	Liquefaction Probability

This project has received funding from the European Union's Horizon 2020 research and innovation programme under grant agreement No. 700748

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EXECUTIVE SUMMARY

The aim of this report is to outline the concepts and protocols that form the basis of the LIQUEFACT Reference Guide (LRG) software, a toolbox for liquefaction mitigation planning and decision support that will help end-users to assess liquefaction risk and develop mitigation plans in order to reduce the effects of earthquake-induced liquefaction damage. To this end, the report briefly describes the process that has been undergone, within Work Package 6, for the design and the development of the LRG software, and the connections with other work packages (WPs) of the LIQUEFACT consortium. It also describes the various protocols, the concept and philosophy of the analysis and processing features characterising the software, and the technology used for the software development.

SCOPE AND PURPOSE

The review presented in the present report should be considered a work in progress which will be amended and modified throughout the duration of the LIQUEFACT project to reflect emerging issues identified by project partners and any location-specific characteristics of the four case study sites identified by the external stakeholder and expert advisory groups.

TARGET AUDIENCE

This report is primarily an internal document intended for the LIQUEFACT project partners and researchers.

This project has received funding from the European Union's Horizon 2020 research and innovation programme under grant agreement No. 700748

LRG Software Toolbox for Liquefaction Mitigation Planning and Decision Support

INTRODUCTION

The LIQUEFACT project aims to develop a more comprehensive and holistic understanding of the earthquake soil liquefaction phenomenon and the effectiveness of mitigation techniques to protect structural and non-structural systems and components from its effects. The LIQUEFACT project will evaluate the mitigation techniques against the potential improvements that could accrue to community resilience in regions prone to EILD events. One of the key outputs from the LIQUEFACT project will be the LIQUEFACT Reference Guide (LRG) software, a toolbox for liquefaction mitigation planning and decision support, able to estimate and predict the likely consequences of EILD to the most vulnerable region of Europe.

The development of the LRG software, which is designed and developed during Work Package 6 (WP6), has been undergoing in two main phases: the first phase of the development process involves reviewing the Deliverables from WP2, WP3, WP4 and WP5 and getting familiar with the various methodologies and procedures that are developed and suggested by the WPs partners. This activity also includes the examination of various potential challenges and issues in integrating the methodologies and procedures into a software toolbox. This has allowed to establish a common understanding of how the software will be used in practice and define specifications of the data, tools and models to be developed in subsequent work packages in order to ensure their successful integration into the LRG software.

The second phase of the development process involves the design and development of protocols and modules were the various outputs from the LIQUEFACT consortium partners are integrated into the software toolbox that will provide civil engineers and relevant stakeholders guidance in making informed assessments on the feasibility and cost-benefit of applying certain liquefaction mitigation techniques for a given earthquake-induced liquefaction threat.

This report provides a description on the development process of the LRG software, illustrating the various steps and activities that have been undertaken in designing and developing an easy-to-use software toolbox that can provide guidance during the building design/assessment and implementation but also during the planning process at local and regional level. The report provides insights on how the various methodologies and different forms of data, provided by the other work packages, have been integrated, and illustrates the interaction between the various protocols of the hazard, risk and mitigation analysis, as well as the concept and the philosophy of analysis process that characterise the software.

CONNECTIONS WITH OTHER WORK PACKAGES

The development of the LRG software involves a process of review and integration of the knowledge and methodologies that have been developed and analysed in WP2, WP3, WP4, and WP5 (see Figure 1). Specifically, the integration of procedures and regression models for liquefaction hazard map

(WP2), the methodologies of liquefaction vulnerability analysis of critical infrastructures (WP3), the mitigation measures (WP4) and the socio-economic loss computation (WP5). The applicability of the software toolbox will be tested in the widest possible range of situations and addressed to selected sample cases representative of the European different characteristics (WP7). Finally, both the LRG software and the validation will be used to support and guide the technical and non-technical decision maker during the planning process and in the development of the Built Assess Management Plan.

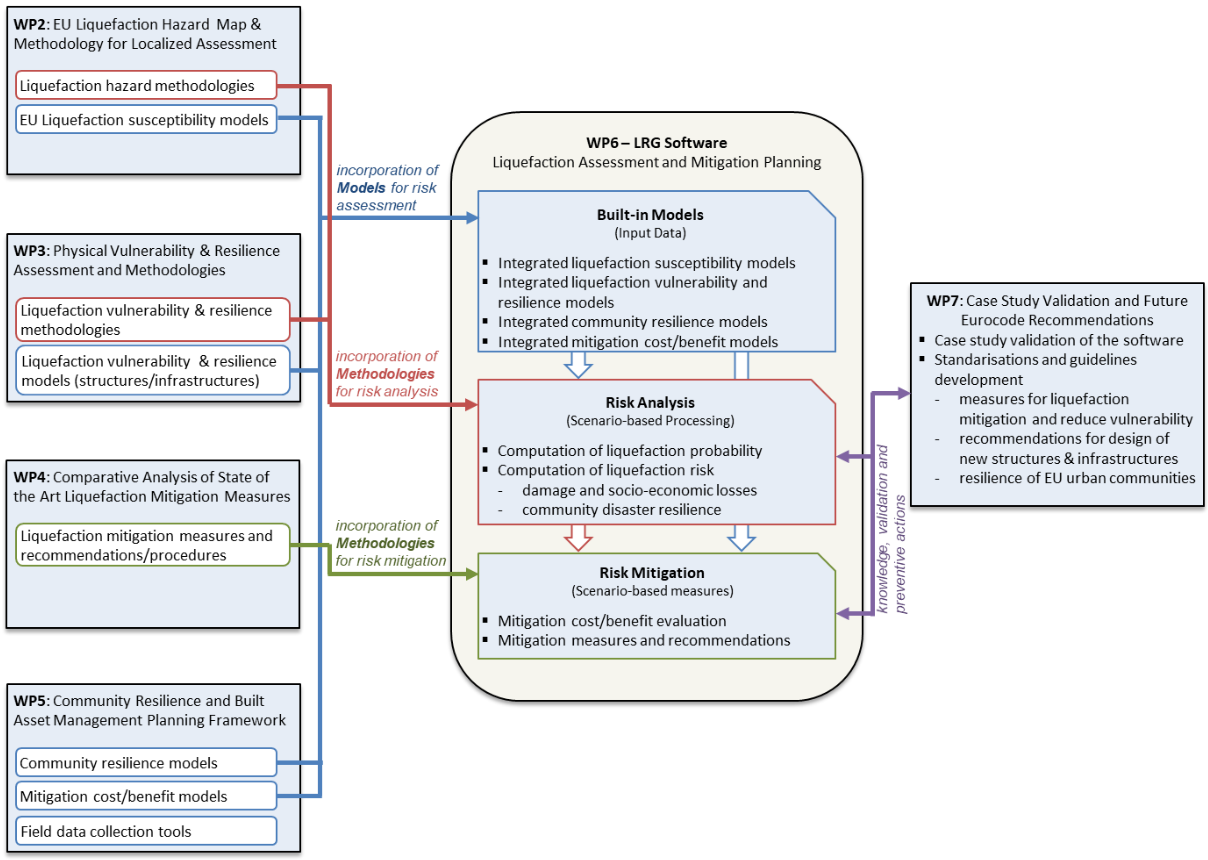

Figure 1. The integration of knowledge and methodologies from WPs into the LRG software

LRG SOFTWARE PROCESSING CONCEPT

Earthquake-induced liquefaction damage assessment is a multi-process analysis that requires different types and forms of input data related to geology and seismology of the site, geotechnical data, and structure-foundation system characteristics of the asset under risk (Table 1). To this end, the LRG software has been designed in a way that EILD assessment is conducted at three independent protocol of analysis to provide more flexibility to the end-user's requirements with respect to the level of analysis to be implemented and type of input data that are available.

The three-independent protocol of analysis implemented in the LRG software are: the protocol for liquefaction hazard analysis, the protocol for risk analysis, and the protocol for mitigation analysis (see

Figure 3 and Figure 2). At the stage of liquefaction hazard, the end-user can conduct qualitative analyses to identify how likely an asset (e.g. individual building/CI asset, portfolio of buildings/distributed infrastructure assets, etc.) is susceptible to liquefaction. If the end-user wants to conduct a risk analysis as well, which is aimed to estimate the level of impact of the potential liquefaction threat on the asset and evaluate the performance, then a quantitative analysis of the liquefaction potential is required (in order to evaluate quantitatively the level of the threat) followed by structural response and damage analysis, and performance evaluation. For the Mitigation Analysis, the end-user can develop a customized mitigation framework based on the outcome of the risk analysis.

Table 1. LRG software Concept with respect to the type of analysis and level of data requirement

Type of Analysis	Data Requirement
hazard analysis – liquefaction susceptibility (qualitative analysis)	liquefaction hazard/susceptibility map
hazard analysis – probability and ground deformation (quantitative analysis)	geological and geotechnical data
EILD risk impact on the asset	structural characteristics-related data and vulnerability models
mitigation and cost-benefit analysis	library of liquefaction mitigation measures and cost-benefit data

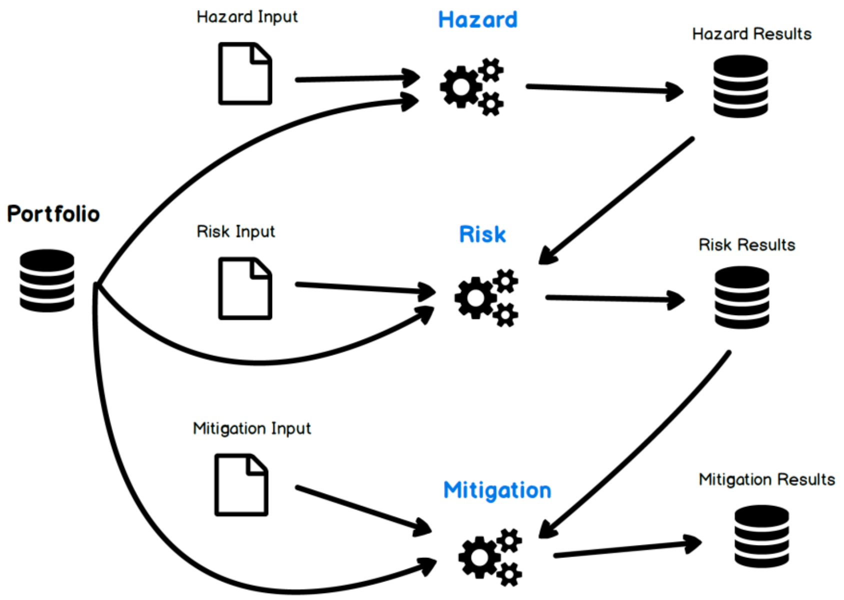

Figure 2. Protocol analysis processes in the LRG software

Analysis Processes

The LRG software has the following specifications/characteristics which are addressed through the various WPs associated with the LIQUEFACT project, (see Figure 3):

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Stage 1: for the evaluation of liquefaction susceptibility and probability of liquefaction for a given susceptible category at specified level of ground shaking intensity;
Stage 2: for the evaluation of the liquefaction threat on the asset, where different approaches can be used in order to correlate the liquefaction-induced ground deformation (e.g. in terms of LSN intensity measure) with the built/infrastructure asset response/damage;
Stage 3: assess the impact of the EILD event (damage probabilities) on the asset due to the occurrence of ground failure liquefaction and ground shaking;
Stage 4: evaluate the impact (i.e. the damage and risk) on the functionality and performance of the asset;
Stage 5: evaluate the adoption of mitigation measures in terms of soil-structural improvements, cost and prioritize the mitigation measures.

Stage 1: Assess liquefaction susceptibility

At the stage of the liquefaction susceptibility analysis, the assessment is conducted to identify whether an asset (e.g. individual building/CI asset, portfolio of buildings/distributed infrastructure assets, etc.) is located in a geographical area likely to be affected by an EILD event. The level of hazard is evaluated by considering the probability of an earthquake hazard and the susceptibility of the ground to liquefaction. The data on liquefaction hazard mapping generated in WP2 and integrated in the LRG software will be used to develop a Susceptibility Matrix (Bartolucci and Jones 2016) that relates earthquake characteristic to ground characterization in order to identify the level of hazard of the asset. The level of hazard will be classified using qualitative labels ranging from "Very Low" to "Very High" that express the level of likelihood of the ground below the asset to liquefaction for any given earthquake characteristic. This analysis will provide asset managers and other stakeholders with an assessment of the range of exposures that their asset(s) is/are likely to be susceptible to.

Stage 2: Evaluation of the level of liquefaction threat to be correlated with the asset response/damage

To assess how the built/infrastructure asset is likely to be affected by an EILD event, liquefaction-induced ground deformation is correlated, by using intensity measure such as LSN, with the asset response. This computation can be implemented using various approaches that were developed during WP2 and integrated into the LRG software, depending on what type of soil profile data are available.

Stage 3: Impact of the EILD on the built/infrastructure asset

Through identifying the building/infrastructure typology using an integrated classification system (is likely to be a combination of construction and foundation type), and through the combination with the associated pre-defined vulnerability model (ground shaking and liquefaction fragility curves), the potential level of damage and vulnerability to the ground condition scenarios, identified in stage 2, are evaluated. For asset managers and other stakeholders, the result of this analysis of damage and vulnerability will be provided in form of a classification using qualitative labels ranging from "Very Low" to "Very High" (Bartolucci and Jones 2016).

Stage 4: Evaluate performance and functionality of the asset

The overall performance and functionality of the asset is evaluated on a case by case basis using the expert knowledge of the facilities manager and building users to interpret the impact the damage and risk (from *Stage 3*) will have on functionality and performance. The loss of functionality will be categorised using qualitative labels ranging from "minor non-structural damage" to "major structural damage" with the loss of performance being a further qualitative statement contextualising the impact of the loss of functionality.

Stage 4: Impact of the EILD on the built/infrastructure asset

Once the level of loss of performance and functionality of individual building/infrastructure assets and the impact on the resilience of a community following an EILD event has been established, end-users will be directed to develop a customized mitigation measure. Based on the outcomes of the hazard-risk analysis (from Stage 1 to 4), a range of mitigation actions are to be identified, and the effect of each on the level of performance of individual buildings/infrastructure assets has to be evaluated. Two types of mitigation actions need to be considered: those that seek to reduce a building/infrastructure asset's vulnerability/increase its resilience; and those that seek to reduce the hazard level. Mitigation options will be ranked according to their impact on the sub-system level and on their contribution to improving overall community resilience.

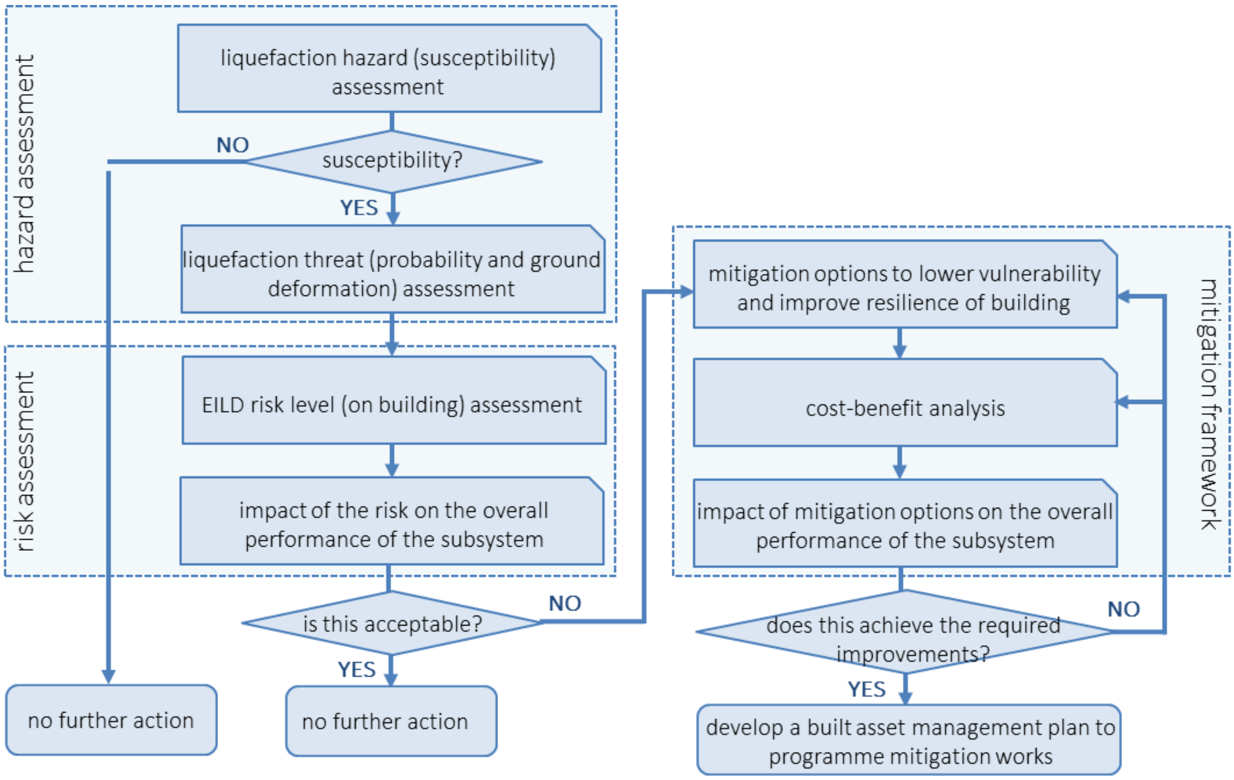

Figure 3. LRG software processing and analysis concept

Options and Alternatives

Taking into account the aspects described above, the LRG software is designed and developed to provide up to nine options/alternatives of analysis processing, as illustrated in Figure 4, offering more flexibility to end-users in conducting an assessment with respect to how detailed the input data are, the availability of the data, and what type of assessment and result the end-users want to obtain.

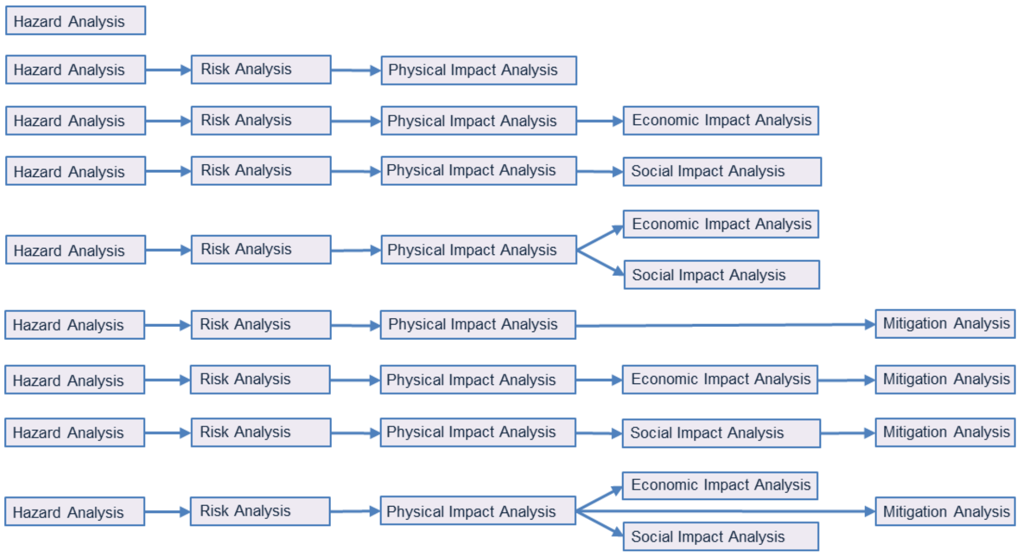

Figure 4. Alternatives offered in the LRG software for EILD related analysis

TECHNOLOGY IN THE LRG SOFTWARE DEVELOPMENT

The LRG software is entirely written in the C++ programming language (Stroustrup 2012) and is developed exploiting object-oriented methodologies and tools, a number of dependences on open-source libraries, and implementation of a wide set of state-of-the-art algorithms. The aim of the various choices undertaken was to ensure the creation of features that would allow end-users a better flexibility and control in terms of selection of assessment/analysis procedure to be implemented, how to view the various results of analyses, type of inputs that can be used/accepted by the software and type of outputs that can be obtained from the software allowing an easy reading and understanding for non-technical user (see Figure 5 and Figure 6).

Graphical User Interface

The user interface has been developed using C++-based tools such as Qt (Rischplater 2014) which is a cross-platform application framework that is used for developing application software that can be run on various software and hardware platforms with little or no change in the underlying codebase, while still being a native application with native capabilities and speed.
With respect to user interaction in maps (e.g. adding information to maps…etc), the development of the software has involved the use of the Qt Modeling Language, QML (Rischplater 2014; Zhi 2018),

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which is a user interface markup and declarative language for designing user interface–centric applications. QML is also used for describing a 3D scene and a frame-graph rendering methodology. QML modules shipped with Qt include primitive graphical building blocks (e.g. Rectangle, Image), modeling components (e.g. FolderListModel, XmlListModel), behavioral components (e.g. TapHandler, DragHandler, State, Transition, Animation), and more complex controls (e.g. Button, Slider, Drawer, Menu). These elements can be combined to build components ranging in complexity from simple buttons and sliders, to complete internet-enabled programs.

Table 2 LRG software development tools

Tools	Description
C++ and QML	code written in C++ 11 and QML (Qt 5.10) for Windows
Qt 5.10	used for the development of the User Interface
Qt and Qwt	used for plotting curves and graphs.
Git and Bitbucket	development of control system for Tracking changes and coordinating Work
OpenStreetMaps (embedded in Qt)	GIS visual view and interactive mapping system
Help and Manual	online help system and user manual
NSIS	easy installation with NSIS (Nullsoft Scriptable Install System) installer
FlexNet	Using FlexNet licensing system (www.flexera.com). Host name locked free license.

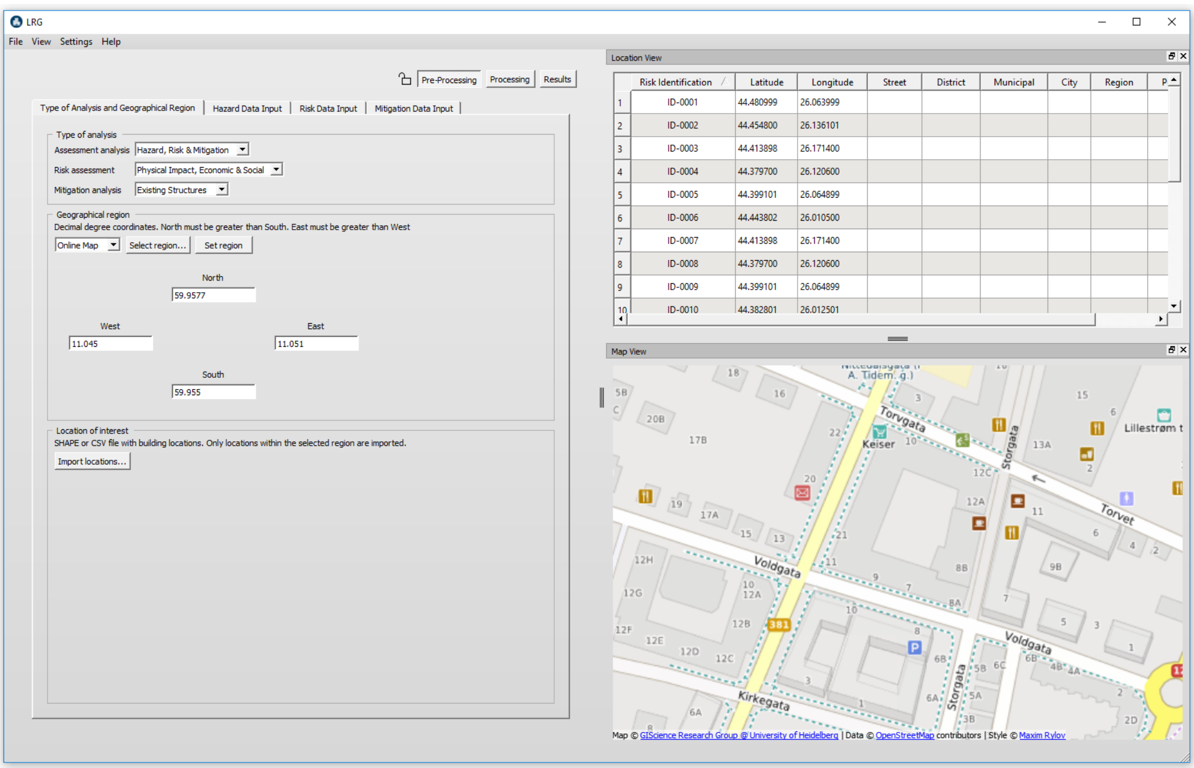

Figure 5. LRG Software - graphical user interface

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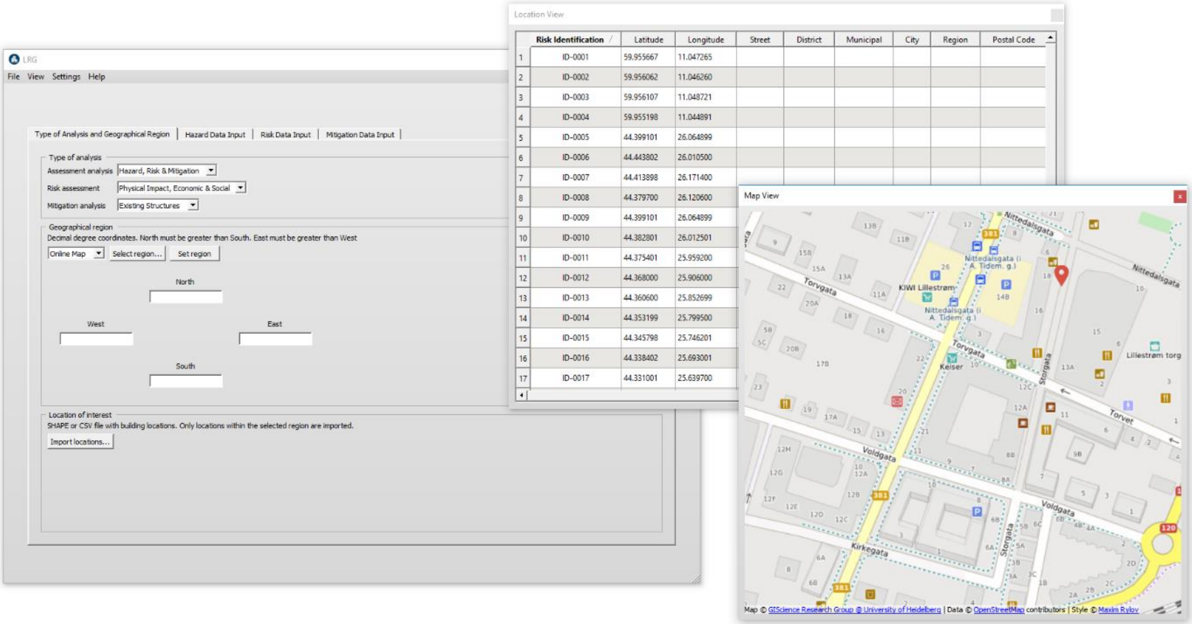

Figure 6. LRG Software - alternative selection of graphical user interface

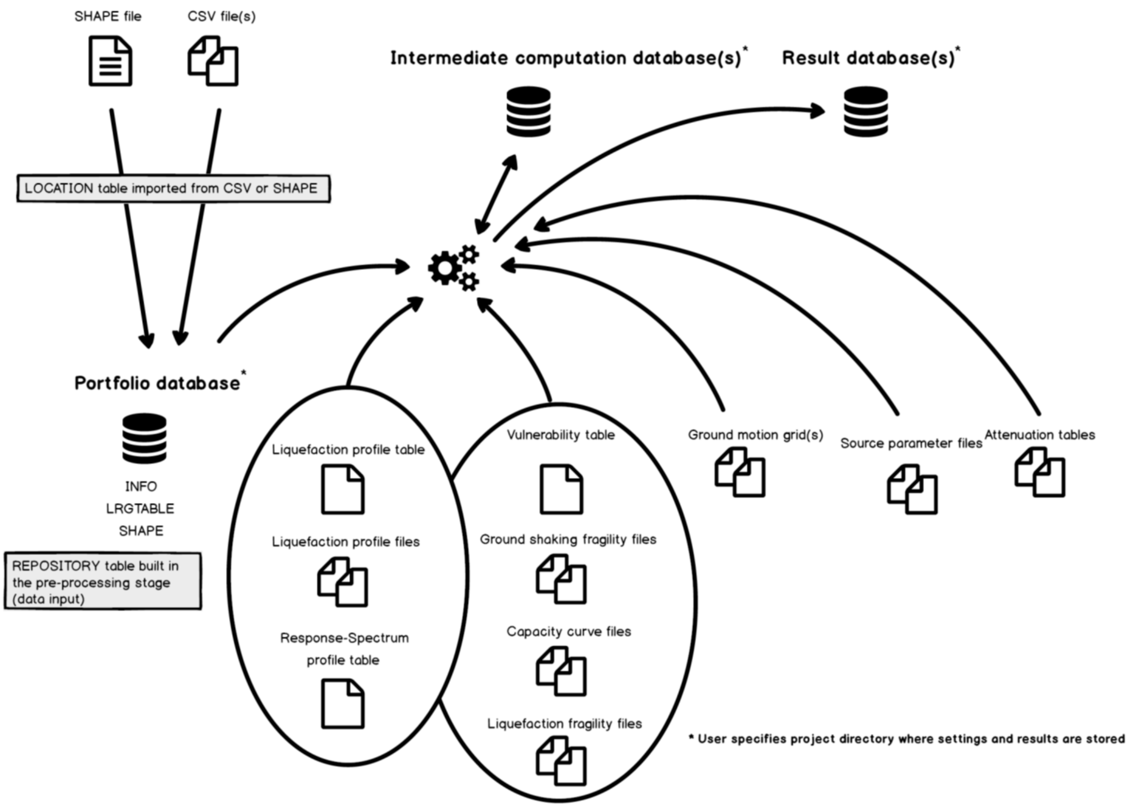

Figure 7. Data flow in the LRG software

Data Flow and Management System

Import of data into the LRG software will be based on tab-separated CSV files or SHAPE files that will be converted to SQLite database files in the project (through a database management system). Results can be exported to CSV or SHAPE, and be generated as 2D grids (e.g. ground shaking or liquefaction hazard maps) through a GIS interactive mapping system. The database files and result files in various formats will be stored in a project directory (see Figure 7).

The development of database management system for handling the database files (input files) and result files (output files) is done through the use of SQLite system (www.sqlite.org) contained in a C-programming library and embedded into the Qt. SQLite stores the entire database (input files such as definitions, tables, indices…etc, and result files in various formats) as a single cross-platform file on a host machine (in a project directory). SQLite databases do not need a server to operate. This simple design is implemented by locking the entire database file during writing. SQLite read operations can be multitasked, though writes can only be performed sequentially.

Control System for Tracking Changes and Coordinating Work

A Control System is one of the modules that the LRG software is based on for tracking changes in computer files and coordinating work on those files. The implementation of this feature has been done based on the use of Git and Bitbucket source code (Loeliger and McCullough 2012). In general, Git allows for source code management in software development, and keeping track of changes in any set of files. It is aimed at speed, data integrity, and support for distributed, nonlinear workflows. The integration of this source provides a full tracking ability independent of network access or a central server. Bitbucket, which is a web-based version control repository, offers both commercial plans and free accounts. It offers free accounts with an unlimited number of private repositories. Bitbucket has 3 deployment models: Cloud, Bitbucket Server and Data Center.

GIS Interactive Mapping System

The LRG software uses Geographic Information Systems (GIS) technology, allowing users to visualize the spatial relationships between various geographic assets or resources for the specific hazard being modeled, a crucial function in the planning process. Open Street Map (Bennet 2010) has been embedded in the Qt for the LRG map module, providing the following features:

- view individual buildings;
- view street names and other labels;
- allowing the overlay of input data (e.g. data on buildings, liquefaction profiles and ground shaking maps) on the LRG map;
- Hide/show overlays of various types;
- obtain a street address from a location (latitude, longitude);
- obtain a location (latitude, longitude) from a street address;
- Click on markers (building, liquefaction profile, …);
- Zoom in and out, and translate the map;
- Specify geographical region; and many more features….

Plotting Curves and Graphs

For technical applications and programs with a technical background, such as plotting curves and graphs, the software development has considered the use of Qwt tool, which is a set of custom Qt widgets. Beside a 2D plot widget the tool provides scales, sliders, dials, compasses, thermometers, wheels and knobs to control or display values, arrays, or ranges of type double.

Online Help System

A single source text for integrated help manual in the LRG software has been created and stored in XML files using an editor built into the Help & Manual program (www.helpandmanual.com) which is a Windows-based help authoring tool and then the text can to be converted to a number of target formats.

Software Installer System

The system of installation process of the LRG software has been developed using a professional open source system NSIS (Nullsoft Scriptable Install System), which is designed to be as small and flexible as possible and is therefore very suitable for internet distribution. The use of NSIS offers to the LRG Software installer a more stable and reliable installation process, capable of doing everything that is needed to setup the software.

SOFTWARE PROTOCOLS AND MODULES

The LRG software computes and analyses earthquake-induced liquefaction damage, the performance and impact for a specific affected property portfolio. The program can use seismic hazard maps (e.g. PSHA based for given return periods) or scenarios as earthquake load inputs, and liquefaction hazard map. The Software consists of three protocols: Liquefaction Hazard Analysis Protocol, Risk Analysis Protocol, and Mitigation Analysis Protocol. Each protocol consists of an Input and a Results Module.

Protocol for Liquefaction Hazard Analysis

The general procedure for the evaluation of liquefaction hazard and the induced ground deformations comprises three main steps:

- Step 1: evaluation of liquefaction susceptibility, i.e. identify the tendency of the various geomaterials at a given site to undergo a severe loss of shear strength due to the pore water pressure build-up caused by earthquake ground shaking.
- Step 2: evaluation of liquefaction probability, which defines the likelihood of experiencing liquefaction at a specific location. This is primarily influenced by the soil's liquefaction susceptibility, the amplitude and duration of ground shaking, and the ground water depth.
- Step 3: evaluation of liquefaction demand which expresses the various modes of liquefaction-induced ground deformation or displacement beneath an asset (e.g. individual building/CI asset, portfolio of buildings/distributed infrastructure assets, etc.).

The development of the Liquefaction Hazard Analysis Protocol in the LRG software has been conducted by the design and incorporation of methodologies and hazard models from WP2 (Lai et al. 2017). The protocol includes the followings (see Figure 8):

- Sub-Protocol with modules for the incorporated methodologies and their associated inputs for the evaluation of Liquefaction Potential (i.e. susceptibility and probability of liquefaction);
- Sub-Protocol with modules for the incorporated methodologies and their associated inputs for the evaluation of Liquefaction Demand.

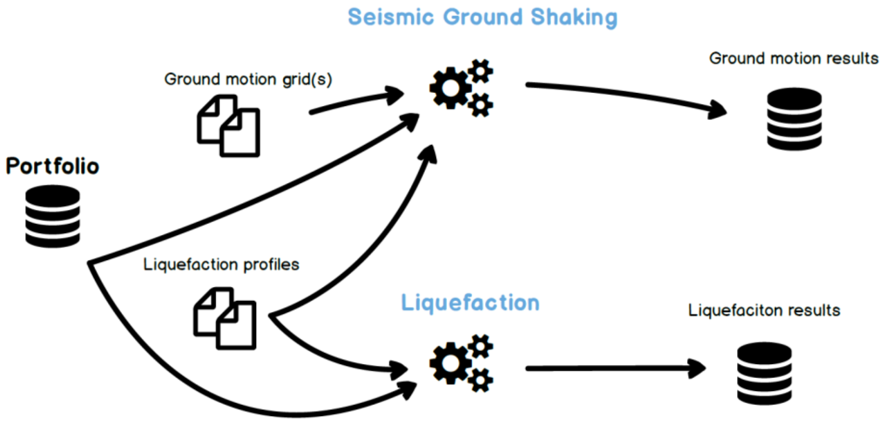

Figure 8. LRG software process for hazard analysis

Sub-Protocol for Liquefaction Potential Analysis

The evaluation of potential liquefaction is a two-step process that involves: (a) the evaluation of liquefaction susceptibility of the soil at a specific location; (b) the evaluation of liquefaction probability for a given level of amplitude and duration of ground shaking.

Module for Liquefaction Susceptibility Estimation

The Module for Liquefaction Susceptibility Estimation is implemented by incorporating the Liquefaction Rating System/Liquefaction Susceptibility Classification Scheme and methodologies and the required parameters for liquefaction susceptibility mapping.

The Module for Liquefaction Susceptibility Estimation is designed such that two options on how to generate liquefaction susceptibility maps are provided in the LRG software (see Figure 9):

- Option-1 (Module 1-1) allows the end-users to provide the required input parameters (e.g. site condition, ground water depth map etc.) for the development of a liquefaction susceptibility map using incorporated methodologies and the Liquefaction Susceptibility Classification Scheme within the LRG software. Then the resulted liquefaction susceptibility map will be combined with the seismic hazard map in order to derive the Probability of Liquefaction map.
- Option-2 (Module 1-2) allows the end users to directly provide a collected generated/existing liquefaction susceptibility map and combining it with the seismic hazard map in order to derive the Probability of Liquefaction map.

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The LRG software will integrate a GIS platform to be used for localised regional assessments of EILD hazards across Europe and a European Liquefaction Hazard Mapping Framework under development in WP2 (Lai et al. 2017).

Module for Liquefaction Potential Estimation

The Module for Liquefaction Potential Estimation is implemented by the incorporation of methodologies, relationships and the required parameters for the computation/evaluation of probability of liquefaction for a given susceptible category of soil at specified level of ground shaking intensity (see Figure 9 and Figure 10).

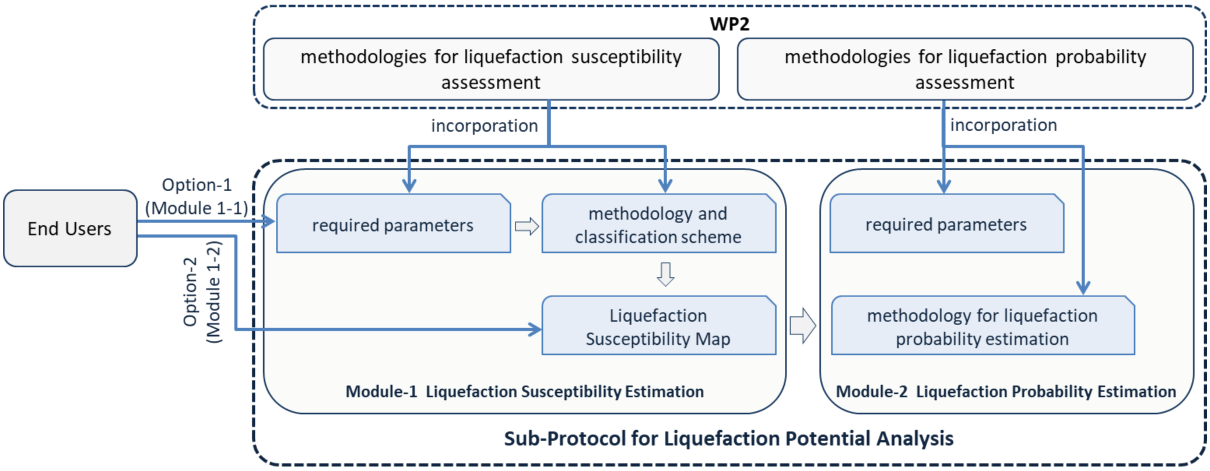

Figure 9. Sub-Protocol for liquefaction potential analysis

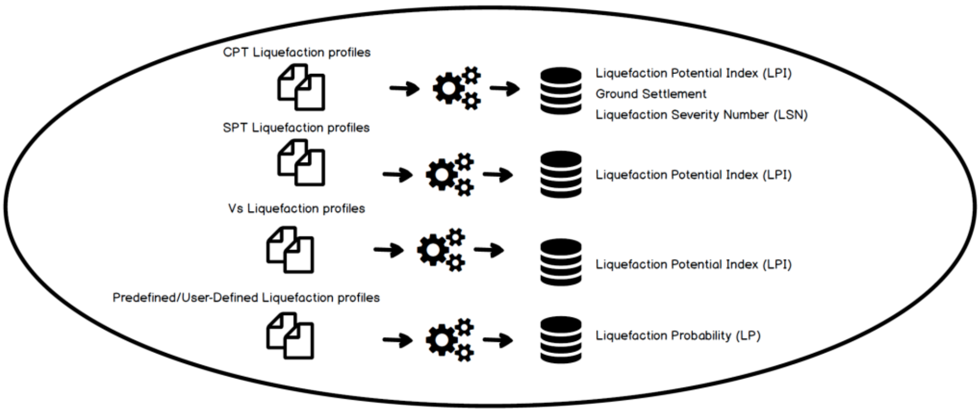

Figure 10. Input and output parameters for liquefaction potential analysis and liquefaction demand analysis

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Sub-Protocol for Liquefaction Demand Analysis

Liquefaction demand expresses the liquefaction-induced ground deformation beneath an asset (individual building/CI asset, portfolio of buildings/distributed infrastructure assets, etc.). In the LRG software, the sub-protocol for liquefaction demand analysis is implemented by incorporating methodologies and the required parameters that allow the computation of ground deformation in terms of intensity measure such as Liquefaction Severity Number (LSN), as well as in terms of modes of permanent ground displacement (see Figure 10 and Figure 11).

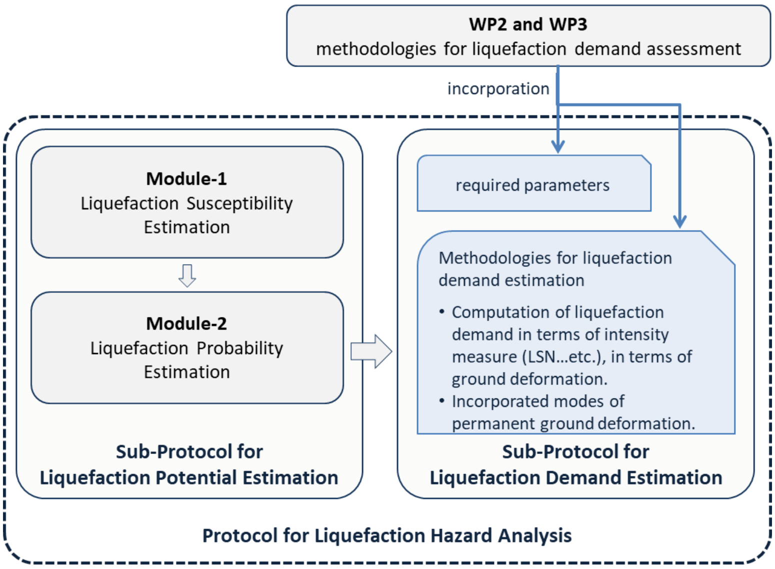

Figure 11. Sub-Protocol for liquefaction demand estimation

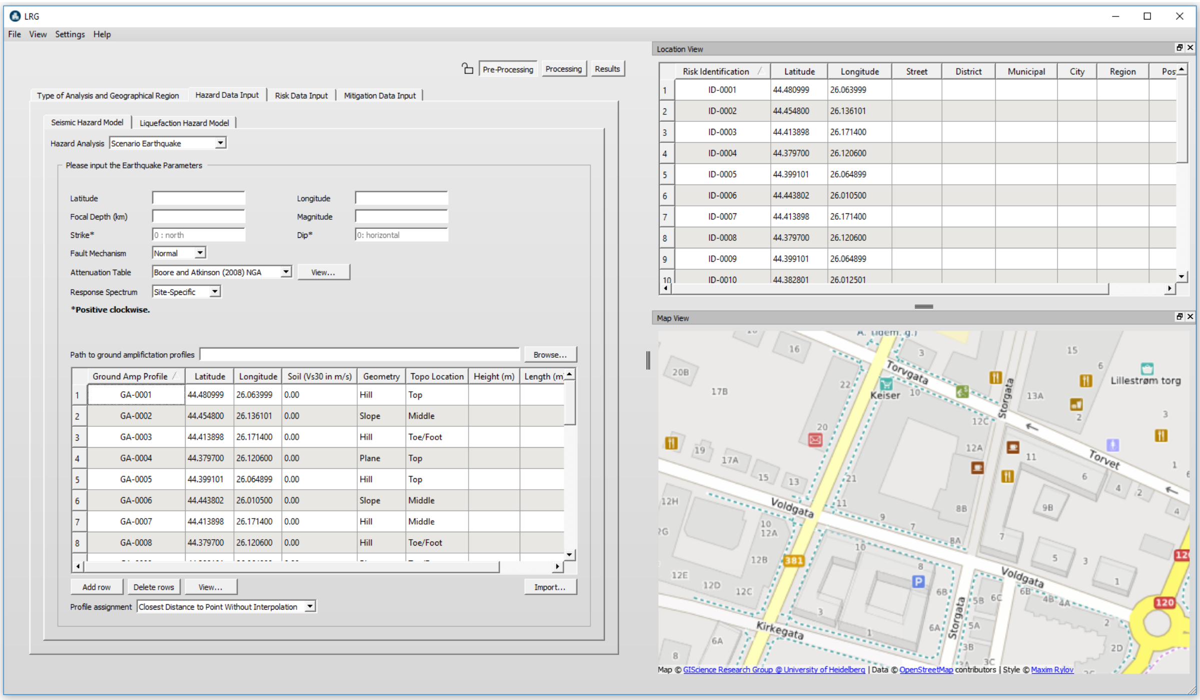

Figure 12. LRG software screenshot – types and forms of input data for seismic hazard analysis

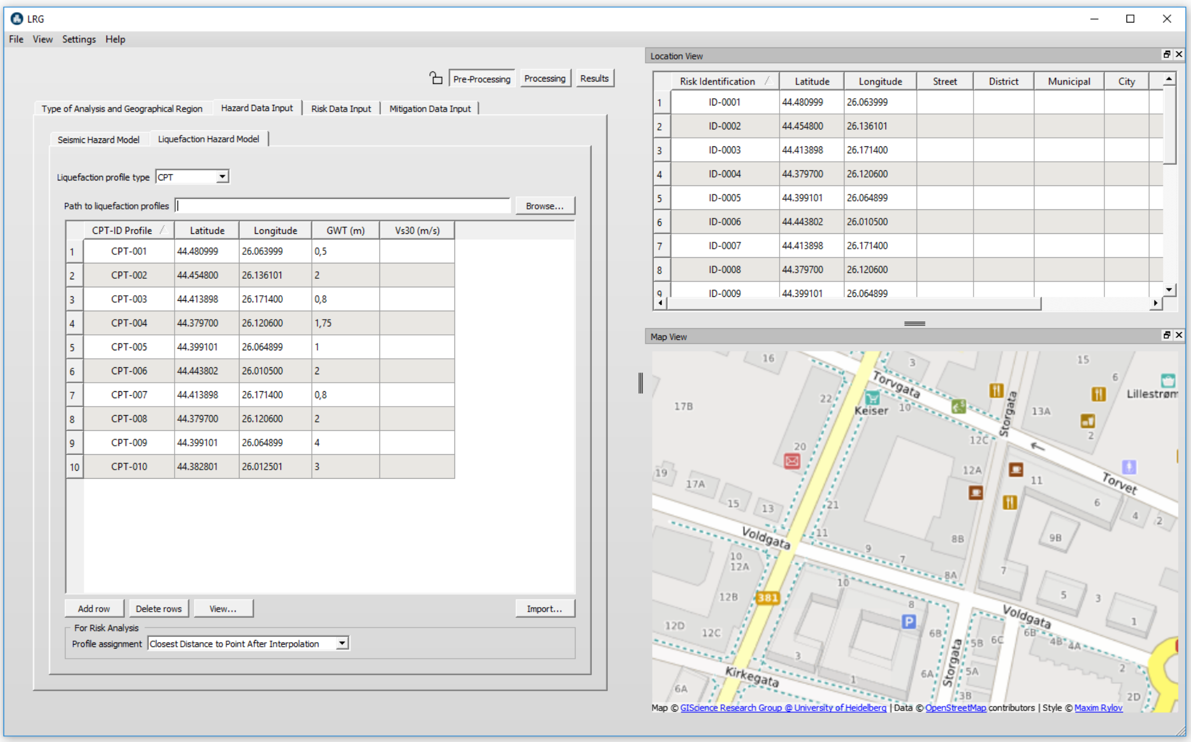

Figure 13. LRG software screenshot – types and forms of input data for liquefaction hazard analysis

Protocol for Risk Analysis

The development of the Liquefaction Risk Analysis Protocol in the LRG software has been conducted by the design and incorporation methodologies and risk models from WP3 (Viana da Fonseca et al. 2017). The protocol includes the followings (see Figure 14):

- Sub-Protocol with modules for the incorporated methodologies and their associated inputs for building response analysis to a defined liquefaction demand (e.g. LSN, mode of liquefaction ground deformation, etc.), and evaluation of the various mechanisms of structural deformation;
- Sub-Protocol with modules for the incorporated methodologies and their associated inputs for the ground shaking and liquefaction vulnerability models, and evaluation of structure performance to a defined liquefaction demand.

Sub-Protocol for Structure Response Analysis

The Sub-Protocol for structure response to a defined liquefaction ground deformation is a two-step process that involves: (a) definition of structure-foundation system; (b) prediction of structural deformation mechanisms/response (Figure 14).

Module for Structure-Foundation System Classification

This Module defines a foundation system classification scheme to be used in predicting the structure response to liquefaction demand. It is developed through the incorporation of the classification scheme and required parameters.

Module for Structure Response Analysis

Developed through the incorporation of methodologies and required parameters for the prediction of structural deformation mechanisms that buildings may experience when subject to a defined liquefaction-induced ground deformation mode. The type of the structure response (mechanism of structural deformation) will depend primarily on the type of the foundation system.

Sub-Protocol for Damage Analysis and Performance Evaluation

The Sub-Protocol for asset damage and performance evaluation is a two-step process that involves: (a) incorporation of fragility model for the defined asset class; and (b) methodologies for the evaluation of damage and the overall performance of the asset (Figure 14).

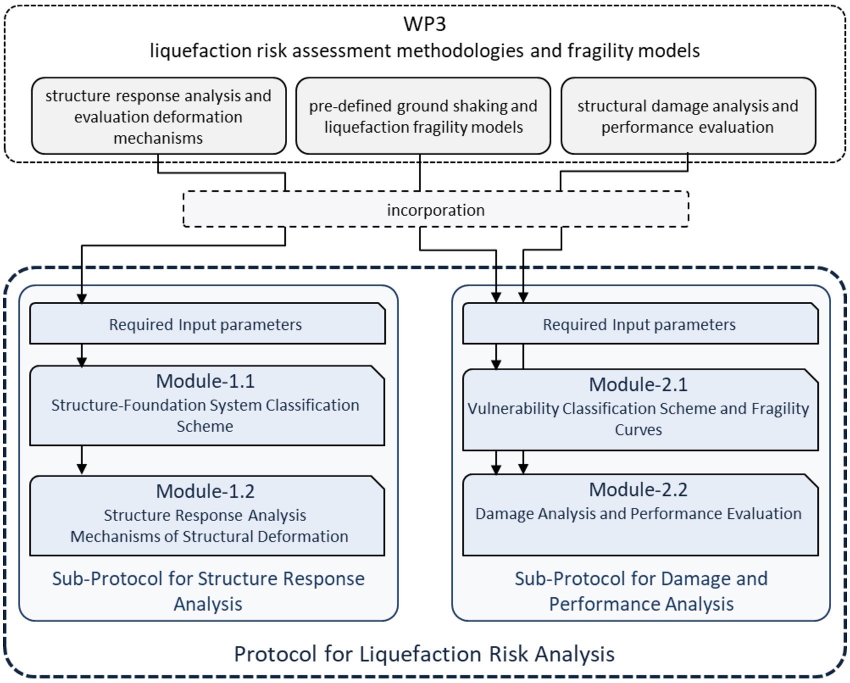

Figure 14. Protocol for liquefaction risk analysis

Module for Vulnerability Classification and Fragility Modelling

This Module allows end-users to input the required parameters for the incorporated procedure of liquefaction vulnerability classification scheme and fragility modelling, to be used for prediction of damage states under a given level of liquefaction demand. Vulnerability is defined in terms of the foundation system, as well as structural system and the member properties.

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Module for Damage Analysis and Performance Evaluation

This Module incorporates methodologies and required parameters for the analysis of the physical damage ratio and the overall performance (by selecting the appropriate vulnerability class and fragility models) to a defined structure response/ground shaking and liquefaction demand.

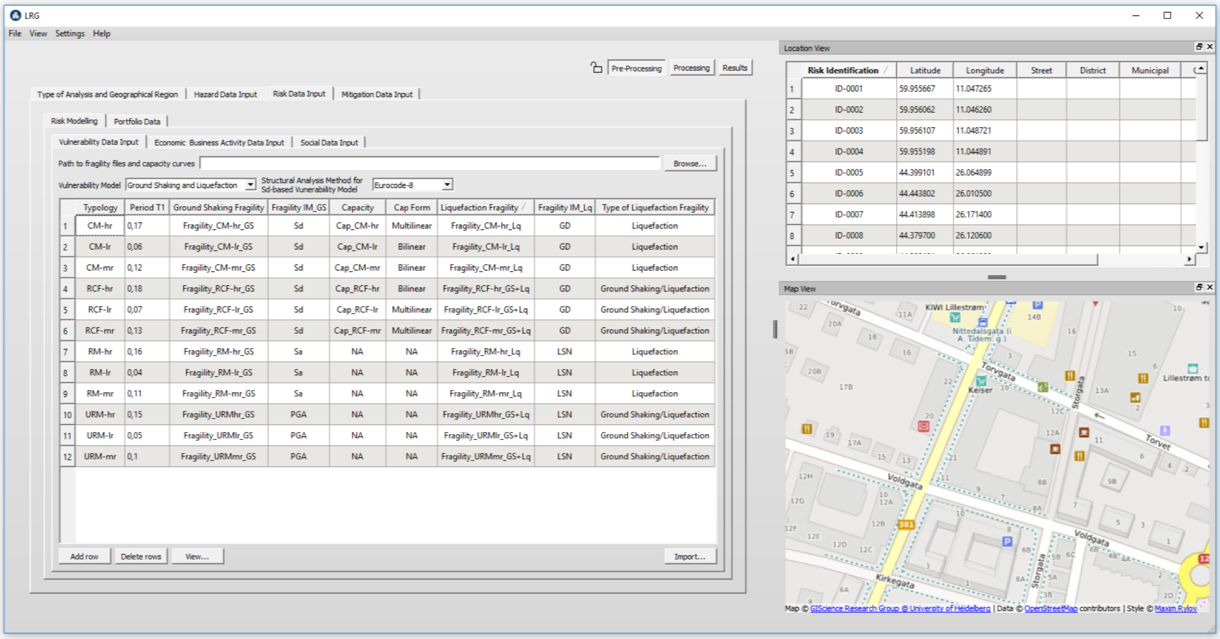

Figure 15. LRG software screen shot – types and forms of input data for risk analysis

Protocol for Mitigation Analysis

The existing approaches for liquefaction mitigation can be techniques for reducing the site susceptibility to liquefaction, or techniques for enhancing the capacity of structures to prevent their collapse if the ground should liquefy. In the LRG software, the protocol for mitigation analysis is designed to include: a process for selecting an appropriate mitigation measure considering the actual in-site condition, and a process for cost-benefit analysis and socio-economic impact (provided from WP4).

Sub-Protocol for Mitigation Measure Selection

In the LRG software, liquefaction mitigation measures are categorized into two main groups (see Figure 16): (a) measures and techniques applicable in a situation of an existing structure/infrastructures; and (b) measures and techniques applicable in a situation of a free-field condition site. The Protocol for Mitigation Analysis will include a module where end users can input site-specific information on the respective area under investigation to determine appropriate mitigation measures and present them to the user in a concise and helpful way.

Sub-Protocol for Cost-Benefit Analysis

The protocol for mitigation analysis also incorporates a cost-benefit analysis that is used to approximately estimate the cost of each mitigation technique.

The protocol for mitigation analysis is a work in progress which will be developed in collaboration with the project partners. Deliverable 4.5 "*The liquefaction mitigation techniques guidelines*" from WP4 will be relied upon for the content of the mitigation techniques recommended.

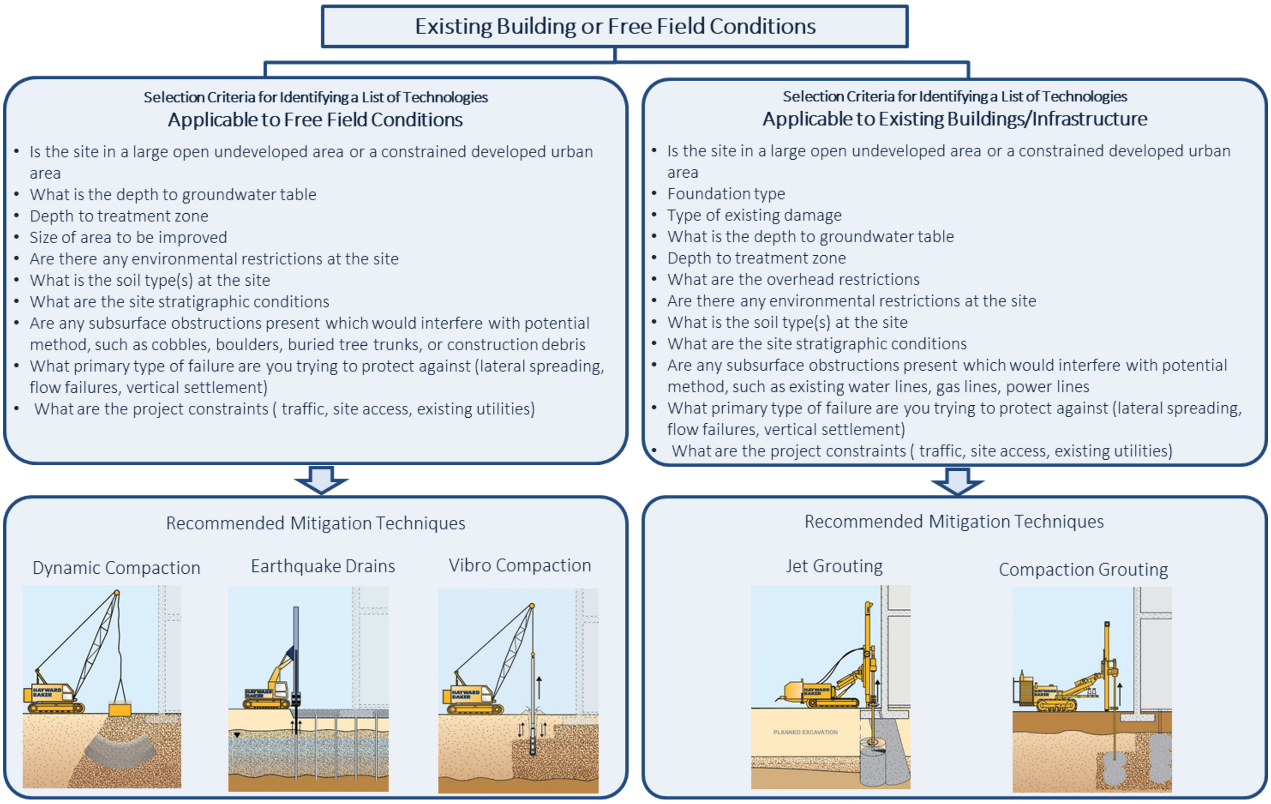

Figure 16. A framework methodology to determine suggested mitigation measures

SUMMARY AND NEXT STEPS

This report provides a description of the LRG software development process that has been undergoing through various work tasks within Work Package 6. The aim is to develop an easy-to use software application toolbox, wherein civil engineers and other relevant stakeholders involved in the design and implementation of a structure/infrastructure is guided to make informed assessments on the feasibility and cost-benefit of applying certain liquefaction mitigation techniques within specific European regions. The LRG software is designed to be applied for an individual level (individual structure/infrastructure) and for region/city level (i.e. in an urban area, GIS-based outputs) with procedures for calculating socio-economic impacts and proposing risk reduction and resilience improvement strategies.

The various parts of the LRG software development presented in this report are still work in progress which will be updated and amended throughout the duration of the LIQUEFACT project including more inputs from the different WPs conducted by project partners. Specifically: (a) integration of updated and more elaborated procedures for performing localised liquefaction analysis and liquefaction hazard map to be provided by WP2; (b) integration of built-in vulnerability models and procedures of performance assessment to be provided by WP3; and (c) the integration of liquefaction mitigation measures and cost-benefit database to be provided by WP4.

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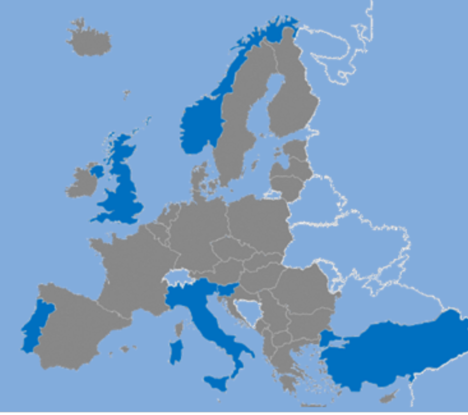
Appendix C

PowerPoint slides from the RAIF/LRG Sprint Test

European Commission

Horizon 2020
European Union funding
for Research & Innovation

liquefACT



Rome Workshop - November 17th 2017

ASSESSMENT AND MITIGATION OF LIQUEFACTION POTENTIAL ACROSS EUROPE

A holistic approach to protect structures / infrastructures for improved resilience to earthquake-induced liquefaction disasters

RAIF AND SELENA-LRG SPRINT TEST WORKSHOP

Keith Jones and Abdelghani Meslem

Anglia Ruskin University

Programme for the Day

- Aims and objectives of the workshop
- Overview of the RAIF and SELENA
- Presentation of D1.4: Detailed user requirements and research output protocols for the LIQUEFACT Reference Guide
  - 6 stage RAIF model
  - CI Healthcare Scenario
  - Data requirements
    - Indicators and metrics
  - Protocols for data collection
    - Existing data sources?
      - What, where, who?
    - New data sources?
      - Specification, WPs?
- Review each stage of the RAIF in 6 workshop sessions
- Identify the role of SELENA-LRG (if any) at each stage of the RAIF

- To review the ability of the RAIF and SELENA-LRG to support a facility manager/operational engineer assess:
  - The antecedent vulnerability and resilience of their infrastructure assets to an EILD event and
  - Assess the relative improvement in vulnerability and resilience that could be achieved through the use of a range of mitigation interventions.
- The workshop will use a hypothetical (simulation) healthcare system
  - Allows for scale – from assets dispersed across a region to a localised asset located on a site to test the RAIF and SELENA-LRG
  - Modern assets should have been designed to resist ground shaking and as such it will allow us to isolate the liquefaction impacts.
- The workshop will work through each step of the 6 stage assessment process (D1.4) to test the logic and data needs of the RAIF and SELENA-LRG
  - Identify what data (performance indicators, metrics and variables) are needed by the RAIF and SELENA-LRG at each stage of the assessment process.

- The workshop will identify which Liquefact work package is responsible for:
  - Identifying or developing each indicator, metric and/or variable.
- At the end of the workshop we will have produced a specification that:
  - Describes the aim of the indicator, metric or variable;
    - Existing or new
  - Details the data type/format (qualitative, quantitative or mixed; empirical or theoretical; measured or derived from expert opinion, etc.) of each indicator, metric or variable; and
  - Identifies the limitations of the indicator, metric or variable.
- The output from the workshop we will be a report that details how the RAIF and SELENA-LRG will use the indicators, metrics and/or variables to assess CI and community resilience to an EILD event.
- This will then be the basis for an updated version of D1.4 and the validation tests to be performed in our case studies.

- Each breakout session will:
  - ➢ Review the requirements of the RAIF and SELENA-LRG
  - ➢ Identify what indicators, metrics and variables are needed to support that stage of the RAIF?
    - Existing indicators, metrics, variables
    - New indicators, metrics, variables
- For existing indicators, metrics and variables:
  - ➢ How robust are they:
  - ➢ Who owns them and how do we access them?
- For new indicators, metrics and variables:
  - ➢ Who is responsible for developing them?
  - ➢ How will they be tested/validated?
- Write a detailed specification for indicator, metric or variable;
  - ➢ Aim
  - ➢ Data format (specifically for inclusion in SELENA-LRG)
  - ➢ Limitations

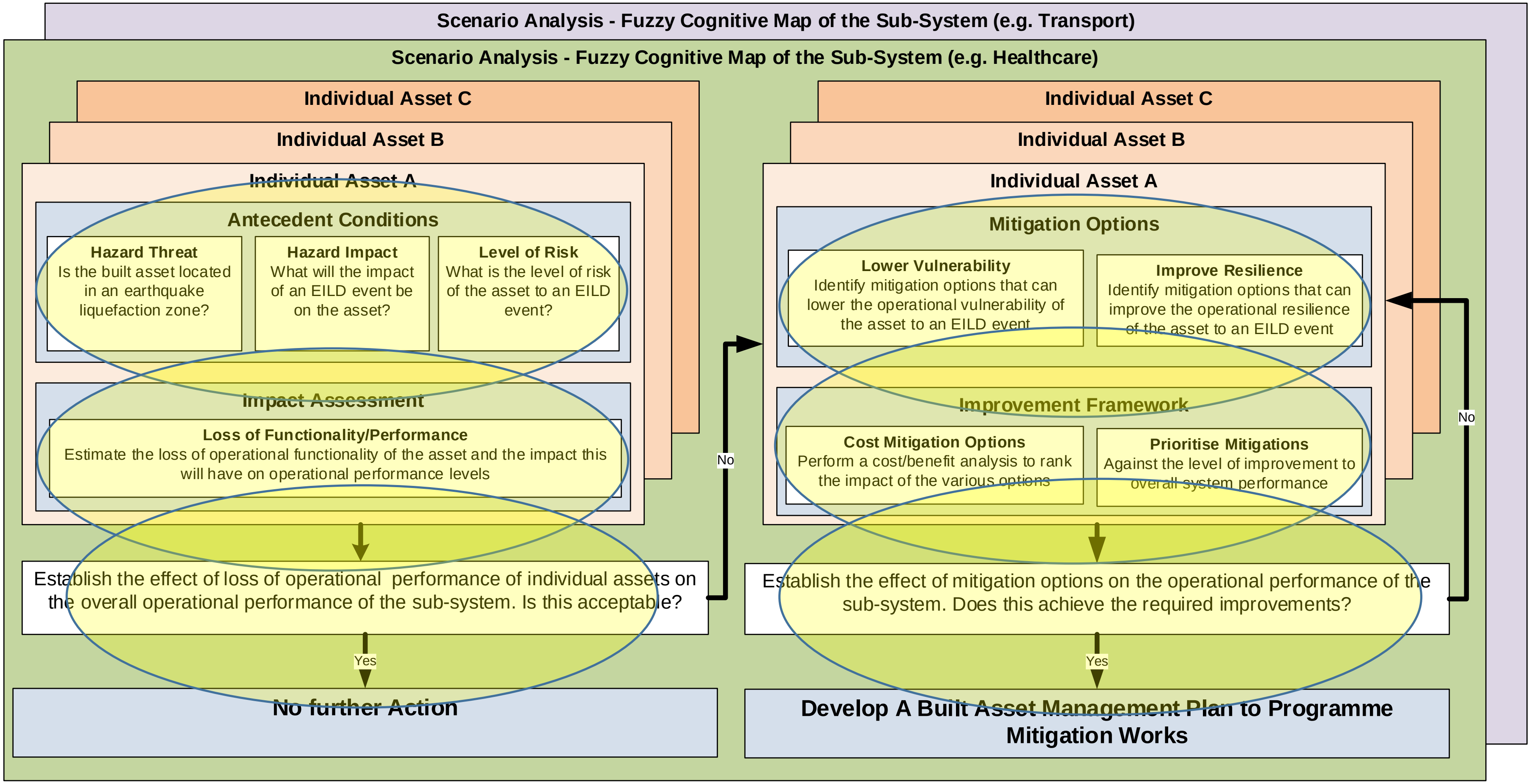
Workshop Programme

Time	Activity
9.30 – 9.45	Aims and objectives of the workshop (KJ)
9.45-10.15	Overview of the RAIF (KJ) and SELENA-LRG (AM)
10.15-10.45	Presentation and discussion of D1.4 (KJ)
10.45-11.00	Coffee
11.00-11.30	Breakout session: Stage 1 – Antecedent Condition Analysis
11.30-12.00	Breakout session: Stage 2 – Impact Assessment
12.00-12.30	Breakout session: Stage 3 – Community Impact Scenarios
12.30-13.30	Lunch
13.00-14.00	Breakout session: Stage 4 – Mitigation Options
14.00-14.30	Breakout session: Stage 5 – Improvement Framework
14.30-15.00	Breakout session: Stage 6 – Built Asset management Planning
15.00-15.30	Coffee
15.30-16.30	Review of the day and next steps

REVIEW OF THE RESILIENCE ASSESSMENT AND IMPROVEMENT FRAMEWORK (Project Objective 1)

- The RAIF provides the theoretical basis for the development of a range of decision support tools that will be integrated into the SELENA-LRG Software and associated LIQUEFACT decision making toolbox.
- The framework consists of 6 stages that help stakeholders to:
  - Assess their hazard liquefaction susceptibility and define the physical vulnerabilities of their assets;
  - Assess the impact of an EILD event on the loss of function of their assets and performance of their service delivery;
  - Identify possible mitigation measures to reduce failure and the consequences of loss of function and performance on community resilience;
  - Evaluate the adoption of mitigation measures in terms of improvements to community resilience;
  - Cost and prioritize the mitigation measures to optimize adaptive capacity;
  - Develop built asset management plans.

RESILIENCE ASSESSMENT AND IMPROVEMENT FRAMEWORK (RAIF)



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OVERVIEW OD DELIVERABLE D1.4

- In order for the RAIF to function effectively the format of the outputs from the different WPs must be consistent with each other and the data generated in each work package must be directly transferable to other WPs.
  - ➢ The aim of D1.4 was to identify the data interactions between WPs and the RAIF and SELENA-LRG.
- Each stage of the RAIF was explored in detail and its potential application to a region/built asset was examined using a simplified primary health care scenario (similar to the SYNERGY project FP7).
  - ➢ The scenario was evaluated using our understanding of the Liquefact bid document; the kick-off meeting; the Bologna and Ferrara Workshops
- The breakout sessions today will provide an opportunity to question the understanding and suggest alternative approaches.
  - ➢ Requirements of the RAIF
  - ➢ Data needed to support the RAIF
  - ➢ Role of SELENA-LRG

Case study box 1: Primary Health Care Scenario

The facilities manager for a regional hospital has been asked to assess the potential impact of an EILD event on the functioning of the hospital. The hospital is located on 4 sites across a small city. Each site contains a number of buildings that provide primary care, administrative and support services to the city community. Whilst each hospital unit concentrates on a primary specialism (e.g. maternity, oncology etc.) they all have a small emergency unit and orthopaedic capabilities. The hospital's buildings range from 100 year old masonry structures; through 50 year old steel and concrete frame structures to modern pre-cast modular units. All buildings are in a good state of repair.

BREAKOUT SESSION 1

ANTECEDENT CONDITION ANALYSIS

- The first stage of the RAIF requires an assessment of vulnerability of
  - individual building/infrastructure assetS, portfolio of buildings/distributed infrastructure assets, town/city wide buildings/infrastructure, regional wide buildings/infrastructure, state wide buildings/infrastructure assets etc.) to an EILD event.
- Vulnerability is:
  - The **characteristics and circumstances** (physical, social, economic and environmental) of a community, system or asset that make it susceptible to the damaging effects of a hazard.
- Are the hospital's assets located in a geographical area that is susceptible to an EILD event?
  - What are the earthquake hazard characteristics?
  - What are the ground condition characteristics?
- Taken together the earthquake hazard characteristic and the ground condition characteristics form a **liquefaction susceptibility** matrix.

		Earthquake Hazard Characteristic					
		TBD	TBD	TBD	TBD	TBD	TBD
Ground Characterization	TBD	Medium	Medium	High	High	Very High	Very High
	TBD	Low	Medium	Medium	High	High	Very High
	TBD	Low	Low	Medium	Medium	High	High
	TBD	Very Low	Very low	Low	Low	Medium	Medium
	TBD	Very Low	Very Low	Very Low	Low	Low	Medium

- Assumption - data provided by WP2 and WP4 will be used to develop the susceptibility matrix.
  - ➢ If the assets are not susceptible to and EILD event then the assessment is complete.
  - ➢ If an asset is susceptible then further investigation is required.

- Role of SELINA-LRG
- A GIS map that will allow built asset managers to geo-locate their built assets onto the map and identify those assets that are potentially exposed to an EILD event.
    - ➢ Hazard level from Very-Low to Very High

Case study box 2: Hazard level of a hypothetical health care structure

On applying the above methodology the facilities managers has identified that two of the hospital's sites are located in an earthquake zone where the generic ground conditions are prone to liquefaction. The hazard level for each of these sites ranges from medium to high depending upon the earthquake characteristic scenario considered. Each of these sites therefore warrants further investigation.

- In order to assess how an individual building/infrastructure asset is likely to be affected by an EILD hazard an assessment needs to be made of the potential impact of liquefaction on the integrity of the building/infrastructure assets on the site.
- For buildings, for example, the vulnerability/resilience is likely to be a combination of construction topology and foundation type.

		Building/infrastructure typology					
		TBD	TBD	TBD	TBD	TBD	TBD
Foundation typology	TBD	Medium	Medium	High	High	Very High	Very High
	TBD	Low	Medium	Medium	High	High	Very High
	TBD	Low	Low	Medium	Medium	High	High
	TBD	Very Low	Very low	Low	Low	Medium	Medium
	TBD	Very Low	Very Low	Very Low	Low	Low	Medium

Assumption – data provided by WP3 and WP4 will be used to develop the vulnerability/resilience matrix.

- The building vulnerability/resilience matrix provides a rapid screening tool with which to identify the relative levels of vulnerability/resilience of each building on a site.
- The level of vulnerability/resilience will be classified using qualitative labels ranging from "Very Low" to "Very High".
- Although the vulnerability/resilience matrix is shown as 2 dimensional it is more likely to be 3 dimensional to take account the different hazard levels.
- Role of SELENA-LRG
  - The rapid screening tool of generic vulnerability and resilience of different combinations of building topology and foundation type.

Case study box 3: Vulnerability of a hypothetical health structure

The facilities manager undertakes further investigation of the two hospital sites located in an earthquake zone where the generic ground conditions are prone to liquefaction.

Hospital A contains a single multi-story hospital building with a footprint of about 1000m^2. The building is of steel frame construction with infill panel walling designed and built to national design and construction codes applicable in the 1990's. The buildings foundations are typical for this type of building. The vulnerability/resilience of this building topology for a medium level of hazard-exposure is likely to be low whilst for a high level hazard-exposure it is likely to be medium.

Case study box 3: Vulnerability of a hypothetical health structure

Hospital B contains 4 low rise hospital buildings located separately on a large site. Each building has a separate primary function (acute medical services, out-patient services, administration, and support services). The buildings are of different construction types and date from the 1920's to the 1970's. All the buildings have been regularly maintained and refurbished so that they are currently in good condition. The buildings foundations are typical for the different types of building. The vulnerability/resilience level of these building topologies under the medium level hazard-exposure scenario has been assessed as:

Building A – Low
Building B – Low
Building C – Medium
Building D – High

Case study box 3: Vulnerability of a hypothetical health structure

The vulnerability/resilience of these building topologies under the high level hazard- exposure scenario has been assessed as:

Building A – Medium
Building B – Medium
Building C – High
Building D – Very High

BREAKOUT SESSION 2
IMPACT ASSESSMENT

Stage 2: Impact Assessment

- The loss of functionality (performance) will be made on a case by case basis using the expert knowledge of the facilities manager and building users to interpret the impact that any given level of risk will have on service functionality and performance.
- It is currently assumed that the loss of functionality will be categorised using qualitative labels ranging from “minor cosmetic damage” to “major structural damage” with the loss of performance being a further qualitative statement contextualising the impact of the loss of functionality.
- Assumption - WP5 will provide the framework for assessing loss of performance; WP7 will provide the data to test the framework
- Role of SELENA-LRG
  - Host the performance assessment framework

- The two scores from the hazard-exposure and vulnerability/resilience matrices will be used to assess the level of risk to building / infrastructure asset(s) which in turn will be used as the basis to assess the loss of functionality of the building/infrastructure asset(s) and performance of service delivery immediately following an EILD event.

		Hazard Level					
		TBD	TBD	TBD	TBD	TBD	TBD
Building Level	TBD	Medium	Medium	High	High	Very High	Very High
	TBD	Low	Medium	Medium	High	High	Very High
	TBD	Low	Low	Medium	Medium	High	High
	TBD	Very Low	Very low	Low	Low	Medium	Medium
	TBD	Very Low	Very Low	Very Low	Low	Low	Medium

- Assumption - data from WP2, WP3 and WP4 will be detailed enough to allow typical impacts to a building to be assessed.
- WP5 will provide the model to link physical impacts to service performance level.

Case study box 4: Risk/Impact Assessment of a hypothetical health structure

Hospital A has a low vulnerability/resilience when exposed to a medium level hazard event; and a medium vulnerability/resilience when exposed to a high level hazard event. Thus the potential impact on functionality for the medium level hazard exposure scenario is likely to be Low whilst the for the high level hazard exposure scenario it is likely to be High.

For the Low Risk scenario discussions between the facilities manager, building users and the health authorities technical consultants identified the likelihood of “minor cosmetic damage” to the building resulting in minimal impact on the performance of the hospital immediately following an EILD event. The hospital could be back to full performance levels once emergency clean-up operations were complete.

Case study box 4: Risk/Impact Assessment of a hypothetical health structure

For the High Risk scenario discussions between the facilities manager, building users and the health authorities technical consultants identified the likelihood of "major structural damage" to the building resulting in complete loss of performance of the hospital immediately following an EILD event. The hospital would be back to full performance levels once rebuilding work had been completed.

A similar exercise for Hospital B identified 4 risk scenarios for each hazard-exposure level. For the medium level hazard-exposure scenario the level of risk, impact on functionality and loss of performance were:

Building A – Low Risk; minor cosmetic damage; minimal impact on performance
Building B – Low Risk; minor cosmetic damage; minimal impact on performance
Building C – Medium Risk; cosmetic damage and minor building services disruption; major impact on performance until post event safety checks on building services are complete then depending on the outcome of the checks full performance levels will be achieved once repairs are complete.
Building D – High Risk; major structural damage; complete loss of performance until repairs are complete.

Case study box 4: Risk/Impact Assessment of a hypothetical health structure

The vulnerability/resilience of these building topologies under the high level hazard-exposure scenario has been assessed as:

Building A – Medium Risk; cosmetic damage and minor building services disruption; major impact on performance until post event safety checks on building services are complete then depending on the outcome of the checks full performance levels will be achieved once repairs are complete.

Building B – Medium Risk; cosmetic damage and minor structural damage; major impact on performance until post event safety checks on building integrity are complete then depending on the outcome of the checks parts of the hospital may be out of action until structural repairs are complete. Full performance levels will be only achieved once repairs are complete.

Building C – High Risk; major structural damage; complete loss of performance until repairs are complete.

Building D – Very High Risk: partial or full failure of the building; complete loss of performance until rebuilding is complete.

BREAKOUT SESSION 3
COMMUNITY IMPACT SCENARIOS

- The impact of the loss of performance of individual building/infrastructure assets on the resilience of a community following an EILD event will be assessed by integrating the performance outcomes identified in stage 1/2 of the RAIF into a FCM (stage 3 of the RAIF) that describes the complex relationships (physical, social, organizational, economic etc.) that constitute a communities resilience to disaster events.
- The resilience modelling component of the RAIF seek to identify and investigate all the factors that influence the vulnerability, resilience and adaptive capacity of an urban community to an EILD event.
- The FCM has been replaced by an AHP model using:
  - UNSIDR Disaster Resilience Scorecard for Cities (D5.1/D5.2)
  - Critical Infrastructure Resilience Model (D5.1/D5.2)

Case study box 6: Impact assessment of the hypothetical health system

The health care authority responsible for mitigation investment decisions wants to better understand the impact that the loss of performance of the hospital assets identified in its risk assessment will have on the overall resilience of the primary healthcare system. A FCM has been developed by the city authority that identifies the factors that affect the cities resilience to an EILD event; a part of which is a primary health care sub-system FCM. The facilities manager can enter the performance levels identified from the risk/impact assessments and the FCM models the impact that these scenarios will have on the resilience of the primary health care sector and on the community as a whole. This information can then be used as a baseline to estimate the improvement in resilience that could be expected from the different mitigation options that will be modelled in stage 4 of the RAIF. In essence the FCM resilience modelling can be used to set improvement performance standards that any mitigation options have to meet.

Case study box 6: Impact assessment of the hypothetical health system

In the hypothetical scenario being considered here, of the 4 hospitals that constitute the primary care system only 2 are susceptible to liquefaction and of these one is classed at Low-High risk and the other as Medium-High risk. Under the Low-Medium risk scenarios it is unlikely that all performance would be lost with both hospitals able to continue to function after the EILD event. When this data is entered into the FCM it classifies the resilience of the primary health care system as Medium-High. Under the High risk scenarios then it is likely that all performance could be lost from both hospitals and when this data is entered into the FCM it classifies the resilience of the primary health care system as Low. These assessments now provide the basis by which improvements in resilience can be assessed for each mitigation option evaluated in stage 4.

Similar analyses can be done at the community level when all the sub-system FCMs are developed.

- Assumption – data from WP5 can be used to model the community resilience at a level of detail that will allow changes in resilience to be observed for different mitigation interventions. The case study region (area) in WP7 will provide a suitable platform to test the community resilience model.
- Role of SELENA-LRG
  - ➢ Host the community and CI resilience assessment tools (e.g. modified version of the UNISDR Scorecard and the AHP CI Scorecard).
  - ➢ Provide a guidance document on how to build a community resilience model

BREAKOUT SESSION 4
MITIGATION OPTIONS

Stage 4: Mitigation Options

- Once the baseline assessment of the resilience of the sub-systems and community to an EILD event has been established and the required improvements in resilience have been defined the ability of a range of mitigation actions to achieve the required improvements can be evaluated.
- This analysis requires a range of mitigation actions to be identified (both physical and operational) and the effect of each on the level of performance of individual buildings/infrastructure assets to be evaluated using the impact assessment matrix outlined in Stage 2.
- Two types of mitigation actions need to be considered;
  - Those that seek to reduce a building/infrastructure assets vulnerability/increase its resilience; and
  - Those that seek to reduce the hazard level.
- The former are likely to be building level interventions; the latter are likely to be ground level interventions.

Stage 4: Mitigation Options

- Assumptions - The technical building level interventions developed in WP3; the ground interventions developed in WP4; and the operational interventions developed in WP5 will provide sufficient detail on reduced physical impact to allow post mitigation service level performance to be assessed.
- Mitigation options will be ranked according to their impact on the sub-system level and on their contribution to improving overall community resilience.
- Role of SELENA-LRG
    - Provide details of typical/possible mitigation options with indicative costing for typical building/foundation topologies

Case study box 7: Mitigation Options for the hypothetical health system

The facilities manager has been tasked with evaluating the potential improvements that can be made to the resilience of both hospitals that are susceptible to EILD events. The facilities manager has commissioned technical consultants to prepare a feasibility report on a range of technical mitigation actions that can be applied to the hospital buildings to reduce their vulnerability or improve their resilience to an EILD event. A range of structural and foundation mitigation actions are identified and the impact that each of these would have on the building vulnerability and hazard impact are assessed.

For Hospital A the building level mitigation actions could lower the risk assessment from Low–High to Low-Medium. This would have the effect of reducing the impact on performance from potential long term closure of the hospital to possible short term loss of performance across part of the hospital. For Hospital B the risk assessment for all buildings could be lowed to Low-Medium meaning that no buildings would close as a result of an EILD event. When these scenarios were run through the FCM primary health care sub-system the level of resilience was predicted to rise from Low to Medium-High. In addition, for Hospital B it would be possible to improve the performance of the hospital by making changes to its operational characteristics by moving critical services from buildings that are highly vulnerable to those that are less vulnerable.

Case study box 7: Mitigation Options for the hypothetical health system

A similar set of technical feasibility reports were commissioned on ground improvement mitigation to reduce the hazard impact (reduce the likelihood of liquefaction). A range of ground improvement mitigation actions are identified and the impact that each of these would have on the buildings hazard level were assessed.

For Hospital A the ground improvement mitigation actions could lower the risk assessment from Low–High to Low. This would have the effect of reducing the impact on performance from potential long term closure of the hospital to possible short term loss of performance due to minor cosmetic damage. For Hospital B the risk assessment for all buildings could be lowed to Low meaning that no buildings would close as a result of an EILD event. When these scenarios were run through the FCM primary health care sub-system the level of resilience was predicted to rise from Low to High.

Each mitigation option was ranked on its potential improvement capability.

Similar analyses can be done at the community level when all the sub-system FCMs are developed.

BREAKOUT SESSION 5

IMPROVEMENT FRAMEWORK

- Once the mitigation options have been identified a cost/benefit analysis will be calculated for each specific sub-system component.
- The cost/benefit analysis will need to consider both direct and indirect costs (e.g. physical, loss of revenue during refurbishment period, etc.) and benefits (e.g. to the organisation, community, etc.) and extend the analysis across geographical and temporal scales (e.g. consider the inter-relationships between multiple similar assets, consider the implications of delaying refurbishment until later in a building/infrastructure life cycle).
- Assumptions – WP5 will provide a generic liquefaction cost benefit tool; WP7 (through the case study) will provide indicative cost/benefit data from the Emilia Romagna earthquakes.
- Role of SELENA-LRG
  - Local and regional level loss assessments

Case study box 8: Improvement Framework for the hypothetical health system

Following detailed cost/benefit analyses of the mitigation options for Hospitals A and B the health care authority have decided to instigate the ground work mitigation actions to Hospital A but not to instigate any mitigation actions to Hospital B.

Hospital A is a fairly new building, designed and built to a high standard and still retaining significant residual value. The investment in the ground mitigation actions is justified because of the residual value and other performance considerations.

Hospital B is a mixture of buildings from the 1920' to 1970's and although they are in a good state of repair they weren't designed to modern standards and they have low residual value and is due a major renovation in about 10 years' time when it will be demolished and replaced with a new hospital facility. In the meantime the resilience of Hospital B will be improved by re-organising its health care delivery model to ensure that high value activities (in terms of community resilience to a disaster event) are located in the least vulnerable/most resilient buildings.

BREAKOUT SESSION 6

BUILT ASSET MANAGEMENT PLAN

- Once priorities have been set, detailed built asset management plans can be developed. These plans require detailed design solutions to be developed for each mitigation intervention and all financial and legal conditions to be addressed before contracts are let. Once implemented, the performance of mitigation intervention against the performance specification detailed in stage 4 is monitored through detailed simulation or in response to an EILD event.

Case study box 9: BAMP for the hypothetical health system

The facilities manager commissions the design and construction of the mitigation actions and monitors their performance through the use of simulations of an EILD event.

- Assumptions – WP2, WP3 and WP4 in conjunction with WP8 can develop detailed design and testing guidance.

- Role of SELENA-LRG
  - Repository of guidance or signpost to guidance

DATA REQUIREMENTS

DATA DEFINITION	DATA SPECIFICATION	CURRENT STATUS
Liquefaction Hazard Map	Geo-referenced map showing the soil susceptibility to liquefaction event	Under development in WP2
Resilience & Vulnerability assessment of structures	Simulation of liquefaction-induced damage and fragility analysis of structures	Under development in WP3
Liquefaction mitigation measures	Analysis and test of the mitigation measures for protection/resilience of assets	Under development in WP4
Community Resilience Assessment	Analysis of the community resilience before and after mitigation	To be developed in WP5
Test and case study validation	Test of the framework and of all the models of analysis created	To be developed in WP7

REVIEW OF THE DAY
NEXT STEPS