Earthquake Loss Estimations: Modelling Losses due to Ground Failure

Juliet Frances Bird

A thesis submitted in fulfilment of the requirements for the degree of Doctor of Philosophy and the Diploma of Imperial College London

January 2005
ABSTRACT

Over the last three decades, significant advances have been made in the development of models to estimate losses caused by ground shaking in future earthquakes. This research focuses on losses caused by liquefaction-induced ground failure. A comprehensive study of the causes of damage and loss in recent earthquakes is used to rank the relative contribution of ground-failure to earthquake losses and to illustrate scenarios where ground-failures can dominate the losses. Subsequently, it is demonstrated that current practice for incorporating liquefaction into earthquake loss estimations is problematic in its application. Simplified approaches are based upon many assumptions and hence carry large and often undefined uncertainties, and more detailed approaches require large volumes of data, which can be prohibitive in terms of time and resources. Even if detailed analysis is possible, it is shown that many aspects of liquefaction, particularly how it impacts the built environment in terms of regional damage and losses are not well covered by our present modelling capabilities.

Having identified shortcomings in current practice, which are supported by case history data from the 1999 Kocaeli, Turkey earthquake, an improved framework is proposed, which seeks to provide a pragmatic solution to the issues of spatial uncertainty and variability in ground conditions and the exposed building stock, while still producing a realistic and meaningful estimation of the expected damage distribution as a result of earthquake hazards.

Within this improved framework, existing building damage scales are shown to be insufficient with respect to describing some modes of liquefaction-related damage and proposed improvements to these scales are presented. Building vulnerability to liquefaction-induced ground deformations is analytically defined, and importantly, the relative uncertainties associated with each level of input data and each stage of the analysis are carefully considered and fully documented.
ACKNOWLEDGEMENTS

The funding for this research has come from an EPSRC Doctoral Training Award, a CASE Award from Arup Geotechnics, and a Marie Curie Fellowship, all of which are gratefully acknowledged.

I would like to acknowledge my supervisor, Dr. Julian Bommer, for his support and enthusiasm throughout the last three years. I am particularly appreciative of the numerous opportunities he has put my way during this period, which have helped to make it such a valuable experience.

The research presented in this thesis is the result of a number of collaborations with colleagues, co-workers and fellow researchers around the world, to which the final product owes everything. Chapter 2 benefitted from my brief but illuminating collaboration with Professors Idriss and Boulanger from the University of California at Davis. Through working with Professor Robin Spence of Cambridge University and Edmund Booth, of Edmund Booth Consulting, my understanding of the structural aspects of loss estimations has been significantly enhanced, and this has in turn improved many of the discussions of these aspects in Chapters 4 and 5. The case study for Adapazari presented in Chapter 5 would not have been possible without the data, discussions, enthusiasm and feedback provided by Dr Rodolfo Sancio, formerly of the University of California at Berkeley, and Professor Jon Bray, also of UC Berkeley. My 3 month industrial secondment to Arup Hong Kong, not only provided the basis for Chapter 6, but also provided enlightening exposure to a commercial loss estimation project, with all its corresponding challenges. My particular thanks to Drs Jack Pappin and Matthew Free for this opportunity, and to Ziggy Lubkowski of Arup for his help and support both before and during my PhD. Finally, the work presented in Chapter 8 is a direct result of my collaboration with researchers at the ROSE School in Pavia, Italy, namely Helen Crowley and Dr Rui Pinho. I am indebted to them both. Particular thanks to Helen for her contributions and enthusiasm, and the difference she has made to my research. Thanks are also due to all of my former and current colleagues at Imperial College.

The support, enthusiasm and, above all, patience of all of my family are appreciated beyond everything. Particular thanks to Dad for the invaluable proof reading. Thanks to Mum and Rehan for always being there for me. Rehan, it is unanimously agreed that I couldn't have done it without you!
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<tr>
<th>Symbol</th>
<th>Description</th>
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<tbody>
<tr>
<td>AAL</td>
<td>Annual average loss</td>
</tr>
<tr>
<td>$a_{\text{max}}$</td>
<td>Alternative terminology for PGA</td>
</tr>
<tr>
<td>$C_B$</td>
<td>Correction for borehole diameters greater than 115mm</td>
</tr>
<tr>
<td>$C_E$</td>
<td>Correction for energy efficiency of SPT hammer</td>
</tr>
<tr>
<td>$C_R$</td>
<td>Correction for rod lengths shorter than 10m</td>
</tr>
<tr>
<td>$C_S$</td>
<td>Correction for non-standard SPT samplers</td>
</tr>
<tr>
<td>CPT</td>
<td>Cone Penetration Test</td>
</tr>
<tr>
<td>CRR</td>
<td>Cyclic resistance ratio</td>
</tr>
<tr>
<td>CSR</td>
<td>Cyclic shear stress ratio</td>
</tr>
<tr>
<td>$\bar{D}$</td>
<td>Average surface fault displacement (m)</td>
</tr>
<tr>
<td>DBELA</td>
<td>Displacement based earthquake loss assessment (see e.g. Crowley et al. 2004a)</td>
</tr>
<tr>
<td>DF</td>
<td>Damage factor</td>
</tr>
<tr>
<td>DR</td>
<td>Damage ratio</td>
</tr>
<tr>
<td>$D_R$</td>
<td>Relative density (of soil)</td>
</tr>
<tr>
<td>DS</td>
<td>Damage state</td>
</tr>
<tr>
<td>DSHA</td>
<td>Deterministic seismic hazard assessment</td>
</tr>
<tr>
<td>EAL</td>
<td>Estimated annualised loss</td>
</tr>
<tr>
<td>EMS</td>
<td>European Macroseismic (intensity) Scale</td>
</tr>
<tr>
<td>F</td>
<td>Factor of safety</td>
</tr>
<tr>
<td>$F_{\text{AL}}$</td>
<td>Soil amplification factor for long period (T=1.0s) acceleration</td>
</tr>
<tr>
<td>$F_{\text{AS}}$</td>
<td>Soil amplification factor for short period (T=0.3s) acceleration</td>
</tr>
<tr>
<td>$f_c$</td>
<td>Corner frequency</td>
</tr>
<tr>
<td>FC</td>
<td>Fines content (% finer than 74μm)</td>
</tr>
<tr>
<td>FEMA</td>
<td>Federal Emergency Management Agency (US)</td>
</tr>
<tr>
<td>$g$</td>
<td>Acceleration due to gravity (9.81m/s²)</td>
</tr>
<tr>
<td>G</td>
<td>Shear modulus</td>
</tr>
<tr>
<td>Notation</td>
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<tr>
<td>G₀</td>
<td>Maximum (small strain) shear modulus</td>
</tr>
<tr>
<td>G𝐃𝐩</td>
<td>Permanent ground deformation (due to ground failure)</td>
</tr>
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<td>G𝐅I</td>
<td>Ground failure index</td>
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<tr>
<td>Hₚ𝐟𝐚𝐜𝐞</td>
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</tr>
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<td>HD𝐆</td>
<td>Highly decomposed granite</td>
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<tr>
<td>HV𝐒𝐑</td>
<td>Horizontal to vertical spectral ratio</td>
</tr>
<tr>
<td>I𝐒𝐄𝐒𝐃</td>
<td>Internet Site for European Strong-Motion Data</td>
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<td>𝐾𝑴</td>
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<td>Ground water level correction factor used in HAZUS liquefaction calculation</td>
</tr>
<tr>
<td>𝐿</td>
<td>Design life (years)</td>
</tr>
<tr>
<td>Lₛ𝐢𝐝𝐞</td>
<td>Maximum length from head to toe of lateral spread</td>
</tr>
<tr>
<td>M𝐚</td>
<td>Million years</td>
</tr>
<tr>
<td>𝑀</td>
<td>Earthquake magnitude</td>
</tr>
<tr>
<td>𝑀</td>
<td>Moment magnitude (alternative notation)</td>
</tr>
<tr>
<td>𝑀𝐰</td>
<td>Moment magnitude</td>
</tr>
<tr>
<td>𝐌𝐋</td>
<td>Local magnitude</td>
</tr>
<tr>
<td>𝐌𝐒</td>
<td>Surface wave magnitude</td>
</tr>
<tr>
<td>𝑀_{𝐦𝐚𝐱}</td>
<td>Maximum magnitude (used in seismic hazard assessment)</td>
</tr>
<tr>
<td>𝑀_{𝐦𝐢𝐧}</td>
<td>Minimum magnitude (used in seismic hazard assessment)</td>
</tr>
<tr>
<td>M𝐃𝐅</td>
<td>Mean damage factor</td>
</tr>
<tr>
<td>M𝐃𝐑</td>
<td>Mean damage ratio</td>
</tr>
<tr>
<td>M𝐌𝐈</td>
<td>Modified Mercalli intensity</td>
</tr>
<tr>
<td>MS𝐅</td>
<td>Magnitude scaling factor</td>
</tr>
<tr>
<td>MS𝐊</td>
<td>Medvedev-Sponheuer-Karnik intensity scale</td>
</tr>
<tr>
<td>𝑁</td>
<td>Annual frequency of occurrence (in context of seismic hazard)</td>
</tr>
<tr>
<td>𝑁</td>
<td>SPT N value</td>
</tr>
</tbody>
</table>
\( N_1 \)  
SPT N corrected for overburden

\( N_a \)  
Adjusted SPT N

\((N_1)_{60}\)  
SPT N corrected for overburden and energy efficiency of equipment

NEHRP  
National Earthquake Hazard Reduction Program

\( P(L) \)  
Probability of liquefaction

\( P_{MI} \)  
Proportion of map unit susceptible to liquefaction

PGA  
Peak ground acceleration

PGD  
Peak ground displacement

PGD  
Permanent ground deformation (HAZUS notation)

PGV  
Peak ground velocity

PI  
Plasticity index

PML  
Probable maximum loss

\( P_{ML} \)  
Proportion of deposit susceptible to liquefaction (in HAZUS, FEMA, 2003)

PSHA  
Probabilistic seismic hazard assessment

\( q \)  
Probability of exceedance

\( R^2 \)  
The proportion of the variance of one variable explained by the variance in another variable. \( R^2=1 \) indicates perfect correlation.

\( R \)  
Source to site distance (in attenuation relationships)

\( R_{jb} \)  
Horizontal distance from site to closest point on surface projection of fault rupture (also known as Joyner-Boore distance)

\( R_{hbp} \)  
Hypocentral distance

\( R_{epi} \)  
Epicentral distance

RC  
Reinforced concrete

\( r_d \)  
stress reduction factor (liquefaction analysis)

\( S_A \)  
Spectral acceleration

\( S_D \)  
Spectral displacement

\( S_{AL} \)  
Long period acceleration, \( = S_A \) at \( T=1.0s \)

\( S_{AS} \)  
Short period acceleration, \( = S_A \) at \( T=0.3s \)

SASW  
Spectral analysis of surface waves
<table>
<thead>
<tr>
<th>Notation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>SBPM</td>
<td>Self-boring pressuremeter</td>
</tr>
<tr>
<td>$S_D$</td>
<td>Spectral displacement</td>
</tr>
<tr>
<td>SPT</td>
<td>Standard penetration test</td>
</tr>
<tr>
<td>SSR</td>
<td>Standard spectral ratio</td>
</tr>
<tr>
<td>$S_{top}$</td>
<td>Average slope across the surface of a lateral spread</td>
</tr>
<tr>
<td>$T$</td>
<td>Response period (s)</td>
</tr>
<tr>
<td>$T_{15}$</td>
<td>Thickness of soil with SPT N &lt; 15</td>
</tr>
<tr>
<td>$T_{AV}$</td>
<td>Period where constant acceleration changes to constant velocity in response spectra</td>
</tr>
<tr>
<td>$T_{VD}$</td>
<td>Period where constant velocity changes to constant displacement in response spectra</td>
</tr>
<tr>
<td>$T_L$</td>
<td>Thickness of liquefied layer</td>
</tr>
<tr>
<td>$T_r$</td>
<td>Recurrence interval of earthquake (years)</td>
</tr>
<tr>
<td>TCIP</td>
<td>Turkish Catastrophe Insurance Pool</td>
</tr>
<tr>
<td>UBC</td>
<td>Uniform building code</td>
</tr>
<tr>
<td>UHRS</td>
<td>Uniform hazard response spectrum</td>
</tr>
<tr>
<td>$V_{s30}$</td>
<td>Average shear wave velocity over top 30m of soil profile</td>
</tr>
<tr>
<td>$V_s$</td>
<td>Shear wave velocity</td>
</tr>
<tr>
<td>$z$</td>
<td>Depth below ground surface (m)</td>
</tr>
<tr>
<td>$Z_{FSmin}$</td>
<td>Depth to minimum factor of safety against liquefaction</td>
</tr>
<tr>
<td>$Z_{liq}$</td>
<td>Depth to top of liquefied layer</td>
</tr>
<tr>
<td>$\Delta_{cls}$</td>
<td>Post-yield limit state displacement capacity of column</td>
</tr>
<tr>
<td>$\Delta_{cls}$</td>
<td>Yield limit state displacement capacity of column</td>
</tr>
<tr>
<td>$\Delta_D$</td>
<td>Horizontal displacement at the top of the column</td>
</tr>
<tr>
<td>$\Delta_{FH}$</td>
<td>Maximum differential horizontal deformation of the foundation</td>
</tr>
<tr>
<td>$\Delta_{FV}$</td>
<td>Maximum differential vertical deformation of the foundation</td>
</tr>
<tr>
<td>$\varepsilon_c$</td>
<td>Limit state strain capacity of concrete</td>
</tr>
<tr>
<td>$\varepsilon_s$</td>
<td>Limit state strain capacity of steel</td>
</tr>
<tr>
<td>$\varepsilon_y$</td>
<td>Yield stress of the reinforcing steel</td>
</tr>
</tbody>
</table>
\( \theta_p \)  Plastic rotational capacity
\( \theta_y \)  Yield rotational capacity
\( \rho \)  Density of soil \((T/m^3)\)
\( \sigma \)  Standard deviation (e.g. of earthquake ground motions)
\( \sigma_v \)  Total vertical stress
\( \sigma_v' \)  Effective vertical stress
\( \tau \)  Shear stress \((kPa)\)
\( \tau/\sigma_v' \)  Ratio of shear stress to initial effective vertical stress
\( \phi_y \)  Yield curvature
\( \Phi \)  Standard cumulative normal distribution
1 INTRODUCTION

The estimation of earthquake losses is becoming increasingly important for governments, businesses, insurers and re-insurers, and private stakeholders. The events of the last decade in Northridge (California) and Kobe (Japan) cost a catastrophic US$20bn and US$150bn respectively, illustrating the scale of financial disasters that earthquakes can cause. On a site-by-site basis, this problem is being addressed through the development of performance-based design methods that allow building owners to specify more than just a life safety level of design, and to define acceptable losses for different levels of earthquake hazard (e.g. SEAOC, 1995). With respect to ground failure hazards, methods such as microzonation, improved land-use planning and mitigation can be used to control these hazards for future developments. On a regional scale, however, it must be recognised that many existing developments remain vulnerable to earthquake damage, and the main objective of loss estimations is to evaluate the distribution and magnitude of these losses.

As early as 1982, it was observed that liquefaction-induced damage had cost society hundreds of millions of dollars (Seed & Idriss, 1982). Earthquake-induced liquefaction is, as noted by Kramer (1996) "one of the most important, interesting, complex, and controversial topics in geotechnical engineering", and many aspects, particularly how liquefaction impacts the built environment in terms of regional damage and losses, are not well represented by our present modelling capabilities. Nonetheless, liquefaction cannot be ignored if studies are to produce realistic and meaningful estimations of damage due to future earthquakes.

Earthquake loss estimation is a technique used to quantify potential losses in an area due to earthquake hazard; the area may be anything from a particular district or zone, to an entire country. Studies have even been carried out to estimate annual economic losses due to earthquakes on a global scale (Chan et al., 1998). Full earthquake loss estimation requires interaction between Earth scientists, engineers, public and private owners of facilities, lifeline operators, planners, and financiers, and as such is a truly multi-disciplinary process. Two particularly significant features from an engineer's perspective are the regional nature of loss estimations, and the fact that they deal with existing buildings and infrastructure, about which very little may be known. Both of these features present an important contrast to site-specific design or even site-specific assessment. Models for the prediction of future earthquake losses are, necessarily, approximations, hence the use of the term estimation. Both the definition of the seismic hazard and the analysis of expected damage levels are greatly simplified in comparison to detailed engineering studies. This thesis focuses on the engineering aspects, i.e. the
estimation of damage, rather than the calculation of economic loss. It is often difficult to describe in engineering terms exactly what ‘damage’ is. In the context of this thesis, damage is defined as “a change in the condition of the structure that adversely affects its future structural performance” (Kehoe, 1998).

The definition of the uncertainty relating to predicted earthquake losses is of fundamental importance. With respect to losses related to ground failure, as this thesis demonstrates, existing methodologies may in many cases produce incorrect results. In these methodologies where ground failure is incorporated there are significant uncertainties related to the methodology and results, which it is the aim of this thesis to recognise, quantify, and, where possible, reduce.

Figure 1-1: Seismic hazards facing the built environment. The hazards are contained within the boxes, and the physical features from which they originate are in the ellipses (Bommer & Boore, 2004)

The built environment is at risk from several hazards directly caused by earthquakes (Figure 1-1). Earthquake losses can be caused by each of these hazards, and in many earthquakes the total losses are a result of more than one, if not all of them. The hazards of strong ground-motion and amplified ground shaking due to topographic or soil effects are together classed as ground shaking. The term ground failure refers to landslides, liquefaction and surface fault
rupture, the physical link between these features being that they are all manifested by permanent ground deformation.

1.1 Earthquake risk and exposure to loss

In developed countries, there is an increasing need for both governments and businesses to protect their assets against significant economic losses in the event of an earthquake. Less than 60 fatalities resulted from the Northridge earthquake in California in 1994, indicating the relative success in countries such as the USA of the implementation of controls to prevent loss of life due to earthquakes.

In developing countries, although economic losses due to earthquakes are generally much less than the figures for Northridge and Kobe quoted at the beginning of this chapter, in relative terms, the financial losses are devastating: the losses incurred by the San Salvador earthquake in 1986 for example were approximately equal to one third of El Salvador's GNP (Coburn & Spence, 2002). An important difference is that developing countries still face catastrophic loss of life, shelter and infrastructure in the event of a large earthquake as was recently observed in Iran, where 30 000 lives were lost due to the Bam earthquake of 26 December 2003. In these countries the prevention of loss of life through effective earthquake-resistant design of buildings remains an urgent priority. Nonetheless, it is still necessary to understand the regional impact of earthquakes, in order to identify areas of high potential losses for both rehabilitation and emergency planning.

Whilst almost nothing can be done to prevent the occurrence of earthquakes, and very little in terms of accurate prediction, there is an enormous amount that can be done in terms of controlling and managing earthquake risk. Although earthquake hazard on a global basis does not change significantly temporally, the risk associated with a given hazard is increasing. The global trend is for populations seeking employment to migrate from rural areas towards cities, causing expansion of urban areas, often in an uncontrolled manner, and a significant increase in the level of exposure.

Risk is quantified in terms of expected losses. The key risks that a populated area faces as a result of an earthquake are:

- Deaths and casualties.
- Collapse and damage of buildings.
- Disruption due to damage and failure of the transport and lifelines.
Catastrophic losses associated with the failure of a critical facility such as a nuclear power plant, or a dam.

Direct and indirect loss of income and long-term effects on the economy of a region or even a country.

1.2 Applications of loss estimations

The estimation of the total expected loss due to a single earthquake or of annual losses due to all expected earthquakes can serve many important purposes, including:

**Raising awareness.** Local populations, authorities or businesses may be ignorant of the dangers they face, reluctant to take steps to reduce their earthquake risk or unaware of the feasibility of effective mitigation measures. In such cases, loss estimations can serve as invaluable illustrations and warning of the effects of an earthquake, both on their own properties and on the region (e.g. Quito Project, Escuela Politécnica Nacional et al., 1994). In developing countries, there can be a tendency to give earthquake risk lower priority than more tangible or pressing concerns, since until the earthquake occurs, the threat is not visible. Similarly, in low hazard areas, there may be a misplaced sense of security against earthquake losses.

**Insurance and reinsurance.** The insurance industry is at increasingly high risk due to natural disasters as portfolios of insured properties in areas of hazard increase, illustrated by Hurricane Andrew in Florida in 1992 and the Northridge earthquake in 1994, which between them cost insurance companies over US$28 billion (Croson & Kunreuther, 1999). Such lessons continue to be reinforced, with the insurance losses in the United States due to the four hurricanes in August/September 2004 estimated to reach US$20 billion. Insurance and reinsurance industries around the world have recognised that they need to understand expected annual losses due to natural hazards, including earthquakes, in order to protect themselves. Earthquake insurance is discussed further in Section 1.3.

**Mitigation policies.** Regional and national seismic design codes and seismic retrofit guidelines require an assessment of regional hazard and risk. Loss estimation results can help to focus retrofit and rehabilitation policies in terms of where funds should be concentrated, through iterative cost-benefit analyses.

**Emergency planning.** Emergency preparation and planning requires an estimation of the number of people needing shelter and medical care, the loss of functionality of critical facilities such as hospitals and the performance of lifelines and their effect on the emergency response.
(disruptions to a transport network may restrict the effectiveness of emergency response vehicles, for example).

**Risk management.** Both public and private companies need to be able to manage their risks. To do so, an estimation of regional effects of an earthquake, as well as an evaluation of site-specific risks and performance, is essential. Many owners may have multiple facilities in a region and need to evaluate the losses affecting each one to develop a comprehensive risk mitigation strategy. Facilities may suffer losses without incurring any direct damage during an earthquake, for example if employees are affected, or delivery routes disrupted.

**Real time estimation of earthquake losses.** Once a regional loss-estimation model exists, it can be used to evaluate any number of alternative scenarios. By predicting the losses due to an earthquake immediately after it has occurred, many parties, including emergency planners and lifeline operators, can gain invaluable information as to where the worst losses are expected, and where recovery efforts should be concentrated. An early estimation of the total economic losses can also be useful where national or international financial assistance is likely to be required (Eguchi *et al.*, 1997). The initial assumptions made, such as the earthquake magnitude, the fault rupture mechanism and the attenuation characteristics will significantly affect the accuracy of the estimated losses (Walker, 2000).

### 1.3 Earthquake insurance

Since earthquake loss estimations are of particular importance to the insurance and reinsurance companies who are increasingly exposed to the devastating consequences of an earthquake in a heavily populated or developed area, it is appropriate at this stage to present a brief explanation of the basics of earthquake insurance.

In California a state run earthquake insurance company has been set up to prevent a repeat of the losses due to Northridge events, where US$12 to 18 billion insured losses were incurred. This fund, known as the Californian Earthquake Authority (CEA) is a state-run insurance fund, providing “mini” earthquake insurance policies for residential properties (Roth, 1996). The New Zealand government has administered a compulsory earthquake insurance scheme since the 1940s (Walker, 2000). Other states and countries such as Turkey (Bommer *et al.*, 2002a), are following suit. In lower-income economies, such as Turkey, it is perceived that the global reinsurance markets are in a far better position to manage and carry the financial risk of an earthquake than the government.
Chapter 1
Introduction

The benefits to an economy of insuring against earthquake losses are that losses are then spread across a much wider market, and also that post-earthquake payouts should be quicker and better managed than they would be in the hands of public offices, which should stimulate regional recovery. To homeowners and businesses, the benefits of earthquake insurance need evaluation on an individual basis (unless they are compulsory), depending upon the level of exposure, the cost of insurance premiums, and the deductibles of the policy.

Insurance companies are generally prepared to underwrite earthquake damage for a fee; the unpredictability of earthquakes means that the policies could be very profitable for many years provided that a catastrophic earthquake does not occur. This unpredictability leads to difficulties in determining the appropriate level of annual premium.

A problem for providers of earthquake insurance is the accumulation of portfolios that are at risk of losses due to a single event in a given geographical area, unlike, for example, fire insurance (Walker, 2000). Insurers can attempt to manage their risks by spreading their policies over a large area and thus limit the proportion of insured properties affected by any one earthquake. The insurance companies’ risk above a given value is frequently underwritten by one or several reinsurance companies, whose portfolios are likely to be spread over an even wider scale. The clearest explanation of the pertinent issues, including why catastrophe risk in general presents more of a challenge to insurers by not following the ‘law of large numbers’, that the author has read is that of Crosen & Kunreuther (1999), an extract of which is reproduced in Appendix A.

An indirect effect of earthquake insurance is the mitigation potential. Faced with high premiums for old or poorly engineered properties, owners may perceive the benefits of retrofit. In reality premiums would probably have to be unfeasibly high before this became a probable scenario for residential buildings (Güldan, 2001), but could influence owners of commercial buildings who could face premiums of over US$1 million/year in the Los Angeles area (Roth, 1996).

1.4 Earthquake losses: direct and indirect

The output of an earthquake loss estimation comprises:

1. Direct damage. The level of seismic hazard, the regional inventory of buildings and infrastructure, and an understanding of the engineering behaviour of each element of the inventory are combined to obtain an estimation of the level of damage to each group of structures in an area. Damage is usually classified into descriptive damage states
ranging from none to complete or damage indices from D0 to D5. Direct damage comprises structural damage, i.e. damage to the load bearing structural elements of a building, and non-structural damage (to non-load bearing components, fittings, building contents etc.).

2. Indirect damage. Indirect damage is a secondary effect of the damage to buildings, facilities or lifelines. Examples of indirect damage include fire following earthquake, leakage of hazardous materials, or inundation due to failure of a dam or levee.

3. Economic loss. This is typically measured by the cost of repair or replacement of damaged buildings, and direct losses due to disruption of businesses. Building damage, lifeline disruptions or less tangible aspects such as slowing down of customer orders, will all lead to loss of income for businesses. Indirect losses due to ripple effects on regional businesses, and socio-economic losses such as the cost of longer journeys to work and the associated loss of leisure time, are much harder to quantify and are therefore frequently left out of a loss estimation study.

Although loss estimations often focus on building damage alone, as Brookshire et al. (1997) state “loss estimation is most useful when it has predictive capability or when it provides policy insights. The best way to attain these capabilities is through an integrated framework that links the geophysical event to the built environment and then transmits the shock through the economy”. A ‘full’ loss estimation requires comprehensive analysis of all three aspects listed above.

This thesis focuses on direct damage, since this is the principle engineering component of loss models. The following sub-sections briefly explain the overall losses that an earthquake-affected community faces.

1.4.1 Direct social loss

The most important social loss is obviously loss of life. Large-scale loss of life is primarily associated with collapse of buildings, or one-off catastrophic events such as the collapse of a long span elevated freeway in California caused by the 1989 Loma Prieta earthquake, or massive flow slides such as the avalanche in Peru in 1970, which wiped out the villages of Yungay and Ranrahirca and caused an estimated 25,000 deaths. On average, over 75% of earthquake related deaths are apparently caused by building collapse (Coburn & Spence, 2002).
Other social impacts include injuries, short- and long-term homelessness, and disruption to schooling, employment, communities and leisure time. Social impacts are extremely difficult to quantify, and are dependent on many variables, such as the time of the day of the earthquake, or the income group of affected householders. Assessment of such impacts, including any attempt to quantify the cost of human lives, falls outside the engineering-based scope of this thesis.

1.4.2 Direct economic loss

The costs associated with direct damage are either capital losses, or income related losses. Capital losses to building stock and lifelines are measured in terms of the repair or replacement costs for both structural and non-structural elements, and contents. Estimated damage states can be related to the ratio of repair costs to total replacement cost where there is sufficient information about the building stock. Income related losses include business disruption, shutdown, loss of function during the repair or rebuilding period, and relocation expenses.

Physical damage to a power supply network for example, would require repair or replacement (capital losses), but these costs may be relatively low compared to the lost income that the power supply company would experience during the repair period. Both the capital losses and the lost income are direct losses. However, if commercial, financial or industrial facilities are affected by the loss of power, the losses suffered by the owners of these facilities are indirect losses (see below). Recent events in the UK illustrate the potential scale of commercial losses due to natural hazards, where a single flash flood in a village in Cornwall on 17 August 2004 was predicted to cost insurers as much as £15 million in terms of lost income from tourism (The Guardian, 18 August 2004).

1.4.3 Indirect economic loss

Indirect losses are more complex both to define and evaluate, since they are not caused directly by earthquake-induced damage to the facility in question, but by damage or disruption outside the facility. Catastrophic earthquakes are known to have significant long-term effects on the economy of a region. Simple examples of indirect losses include the interruption of a supply and demand chain due to transportation disruption between two points in the chain; or loss of revenue to manufacturing facilities which although undamaged by the earthquake lose services such as water or power leading to reduced productivity. Businesses outside a damaged area may also suffer income losses if they are reliant on either demand or supply from businesses within the damaged area, this is known as the ‘ripple’ effect. An indirect economic impact observed after the 1994 Northridge earthquake was the near or total bankruptcy of a number of
small insurance companies who provided earthquake insurance but did not have sufficient funds to carry the extraordinary losses (Roth, 1996).

Very few earthquake loss estimations calculate indirect losses. HAZUS (FEMA, 2003) is one of the few that does, and even then not in any great detail due to the lack of data and the difficulty in quantifying the effects. Economic data in post-earthquake reports is of variable quality, and rarely includes any breakdown of indirect economic losses. Full indirect losses may not be realised until many years after the earthquake. For example, the long-term closure of the Port of Kobe, Japan, after the 1995 earthquake, impacted the city of Kobe far beyond direct capital losses, since many jobs were lost, and hotels, restaurants and petrol stations were all adversely affected by the removal of this thriving industrial facility.

Another indirect impact is on government spending plans and the regional economy. Such effects are more significant for low or moderate income economies. After two major earthquakes in Turkey in 1999, the Turkish government was left liable for an estimated US$20 million of costs (Bommer et al., 2002a). The necessary re-allocation of public funds from other areas to cover these, or to repay debts incurred as a result of the costs, will have a long-term detrimental effect on the Turkish economy.

1.5 Scope of this thesis

The specific objective of this thesis is to examine the ground failure component of loss estimations, in terms of its contribution to total losses, the currently available methods, and potential improvements. In areas susceptible to ground failures such as liquefaction and landslides, these hazards can dominate the nature and distribution of building damage. This has been seen repeatedly in past earthquakes including Kocaeli (Turkey) in 1999, Kobe (Japan) in 1995 and Luzon (Philippines) in 1990.

Ground deformations due to liquefaction include uniform or differential settlements in level ground and uniform or differential displacements towards a slope or free face. The response of buildings to these deformations depends upon their foundation and structural systems. A building that has settled or tilted excessively, due to liquefaction beneath its foundations, such as that shown in Figure 1-2, would be demolished after the earthquake, and therefore the repair costs would be equal to the replacement value in the same way as for a structure that has collapsed due to strong ground shaking (e.g. Figure 1-3). Fatalities, injuries and contents losses for the building in Figure 1-2 may be less than for collapsed buildings, but business interruption losses would be just as great and demolition costs may be higher. For building owners, insurers
or government bodies requiring an estimation of the nature, distribution and extent of future earthquake losses, it is evident that the presence of liquefiable soils warrants careful consideration.

Figure 1-2: Liquefaction induced tilting of a three-storey apartment building in Dagupan, Luzon, Philippines, following the 1990 earthquake. (Courtesy Earthquake Engineering Field Investigation Team, EEFIT, UK)

Figure 1-3: Collapse of a 10 storey aparthotel in Baguio, Luzon, Philippines, following the 1990 earthquake, causing a large number of casualties. (Courtesy Earthquake Engineering Field Investigation Team, EEFIT, UK)

Those involved in earthquake loss modelling are currently presented with three choices with respect to the incorporation of ground failure. They can choose to ignore it, assuming that any estimation of losses caused by shaking would effectively subsume the impact of such secondary hazards; they can include ground failure in a simple manner, using published approaches based upon qualitative data and a large degree of judgement; or they can opt for a detailed assessment by specialists of damage due to ground failure, with the associated time and expense. These options are reviewed in the light of case history data in Chapters 5 and 6, and it is shown that in
fact none of them serve the requirements of a loss estimation well. These requirements can be summarised as: *to provide a pragmatic solution to the issues of spatial uncertainty and variability in ground conditions and the exposed building stock, while still producing a realistic and meaningful estimation of the expected damage distribution as a result of earthquake hazards.*

Selection of the most appropriate methodology for an individual study will be governed by the end user of the results (insurance, emergency planning, government risk assessments); the available resources; the available data; and the regional characteristics of the study area. This thesis aims to provide appropriate guidance and recommendations for the different levels at which losses due to ground failure may be considered, recognising the practical issues relating to the implementation of loss estimation studies.

Chapter 2 presents an overview of seismic hazards, their definition, and their assessment, considering the input requirements of regional damage assessment. The information presented in this chapter provides the context for subsequent discussions of earthquake loss estimation methodologies.

Chapter 3 reviews the principal features of earthquake losses incurred in damaging earthquakes since 1989. The results of the review are analysed in order to provide guidance as to which components of a loss estimation model should merit the greatest time and resource allocation. One of the motivations for the review was to question the almost ubiquitous assumption that ground shaking is the primary cause of earthquake damage and loss. The important issue of the interaction of ground shaking and ground failure for individual buildings is also investigated. The recommendations consider different regional environments and end users, and direct versus indirect losses. The findings of the review are carried forward to the remainder of the thesis and provide a clear context for the recommendations made therein.

In Chapter 4 existing earthquake loss estimation methodologies are reviewed. Since the field is still relatively young, many methodologies are still under development and there is not yet a coherent approach recognised as being standard practice worldwide. The focus of Chapter 4 is on the engineering aspects of loss estimations, which relate to the determination of the seismic hazard, and the assessment of the vulnerability of the exposed infrastructure. Particular attention is given to the HAZUS (FEMA, 2003) methodology, and how it estimates losses due to ground failure. Being the only published methodology to provide a full explanation of how the ground failure component is incorporated; the 'simplified' HAZUS approach is subjected to a detailed and critical review.
Chapter 5 uses field data from the 1999 Kocaeli earthquake to investigate and calibrate current methodologies. Some problems associated with employing such data are illustrated within this chapter. The city of Adapazari suffered significant damage due to liquefaction, and the availability of extensive field data after the earthquake provides a valuable, and rare, opportunity to consider the significance of the observed building performance and to compare it to available prediction methods in a zone of liquefaction.

Chapter 6 presents an application of existing methodologies using simplified in situ geotechnical data, at a level that could realistically be obtained for a regional study. The data, from Hong Kong, are used to evaluate liquefaction hazard in a region of low to moderate seismicity, but with potentially high exposure to losses due to being densely populated and of economic importance to the region. As well as liquefaction, the soil amplification for the area has been estimated and the sensitivity and uncertainties of each hazard are assessed. Many important features of geotechnical elements in regional hazard modelling are illustrated. Potential simplifications to reduce the computational requirements are explored at the end of the chapter.

Chapter 7 discusses the key implications of the preceding chapters, and uses the case study results to illustrate some of the fundamental issues of uncertainties and sensitivity. The HAZUS ground failure component is revisited in the light of the case studies. Based on the findings presented up to and including this chapter, the important criteria for estimating losses due to liquefaction are presented, discussing the uncertainty of each aspect and proposing how to resolve some of the problems currently faced.

Chapter 8 builds on the issues highlighted in Chapter 7 by presenting a new building vulnerability model for ground deformation induced damage. The response of buildings to liquefaction depends upon their foundations and superstructure types, and the mode of ground deformation to which they are subjected. Solutions for different scenarios are described, and two cases are presented in greater detail. These cases are: rigid body deformations of buildings on shallow stiff foundations, for which a primarily empirical solution has been sought; and differential ground movements beneath reinforced concrete frame buildings on flexible shallow foundations, which is solved analytically. Within Chapter 8, a new damage scale is proposed to overcome the shortcomings of existing scales with respect to ground failure-related damage.

Finally, Chapter 9 contains closing discussions, conclusions and recommendations for further work.
2 SEISMIC HAZARDS AND THEIR ASSESSMENT

This chapter describes the principal seismic hazards and their assessment methods. Their impact on the built environment is reviewed in Chapter 3. The objective of this chapter is to contextualise subsequent discussions of loss estimation methodologies and the approaches used to define regional hazards as input to loss models, particularly ground shaking and liquefaction. The input to a loss estimation model can be divided into two parts: the seismic hazard data, and the building and infrastructure inventory data, including performance and occupancy classifications. This chapter considers the former, comprising ground shaking hazard and collateral hazards. Collateral hazards include fault rupture, tsunami, liquefaction, landslide, fire, inundation and release of hazardous materials; all except the first two are dependent on the initial correct assessment of the strength of ground shaking.

The relevant information required to develop a new regional hazard model comprises the input data, the assessment methodology, the form of the results and the definition of uncertainties. The quantity and quality of input data required is of fundamental importance for a regional risk study. In this chapter the required input parameters are presented and in subsequent chapters, options for focussing data collection in the most important areas are considered. Assessment of seismic hazard potential and effects can range from essentially qualitative to complex quantitative approaches. Loss estimations tend towards the simpler approaches due to the scale of the study areas and the necessary simplifications of input data and calculations.

There are uncertainties associated both with the methodologies for estimating hazards and with the required input data. The uncertainties will vary according to the type of methodology, whether theoretical or empirical, and are better defined in some cases than others. Uncertainties in the estimated seismic hazard will propagate through a loss model, and, although they are often unavoidable, it is essential that they are understood and quantified. The seismic hazard can frequently be hidden behind the end results of a loss estimation, and since the users of these end results are rarely experts in seismic hazard themselves, appropriate representation of uncertainties within the model is very important.

The objective of this chapter is to present the necessary information required for the subsequent discussions of loss estimation methodologies. Each hazard included in this Chapter is in itself the subject of very extensive past and present research, and therefore the sections herein are necessarily condensed summaries.
2.1 Input to regional hazard studies

An earthquake is characterised by its source parameters, i.e. its location in time and space, the earthquake fault rupture mechanism and its size in terms of seismic moment or magnitude. Fault ruptures are usually classified in terms of the direction of slip and the relative motion of the two blocks as shown in Figure 2-1.

![Figure 2-1: Mechanisms of fault rupture](image)

By far the best available source data regarding seismicity of a region is instrumental data from seismograph stations. Compilation of regional instrumental data from seismographs and accelerographs allows the spatial distribution of recent earthquakes occurring within a study area to be plotted. The main shortcoming in the use of instrumental earthquake data is the short period of time (just over 100 years) for which these data are available. Large earthquakes may frequently have an average recurrence interval significantly longer than 100 years. For example, the capital city of Portugal, Lisbon, was hit by devastating earthquakes in 1531 and 1755, as well as a smaller earthquake causing damage to the region in 1909 (Teves-Costa & Senos, 1996), but instrumental data are only available for Portugal since 1920, which would not provide any information about these large magnitude and long recurrence interval earthquakes.

Earthquakes that occurred before instrumental recordings were available are known as pre-instrumental or historical earthquakes. Evidence of such earthquakes can be found in contemporary written or even pictorial publications. Records may cover several thousands of years for populated areas in regions such as the Middle East, China and Europe that have long documented histories. Accounts of earthquakes and their effects can be used to estimate the
corresponding macroseismic intensity value (Section 2.4.1) at the location referred to. In this way, when sufficient evidence is available, contours of equal intensity, and therefore the epicentral location and earthquake magnitude can be estimated.

Identification of active faults is also a fundamental component of a seismic hazard analysis. An earthquake catalogue alone, even the most comprehensive one, spans a very short period of geological time compared to the potential time period of fault movement. Furthermore, there may be active faults in a region that have yet to rupture, and thus have no evidence of associated seismicity to date. There is a very clear correlation in this case between investing more money in investigation of fault location and activity and reducing the associated uncertainty.

Available tools for fault identification include geological maps and other published data, geophysical survey data, aerial photography, satellite imagery and intrusive physical investigation such as trenching or boring. Advances in seismology, geomorphology and remote sensing technology, combined with careful inspection of regional earthquake catalogues, have made it possible to locate most shallow superficial faults in continental areas (Jackson, 2001). Evidence of past earthquake activity, even in areas with relatively short documented histories of habitation, for example parts of New Zealand, can be obtained from paleo-seismological information (e.g. Longridge et al. 2003). Trenching across known faults and carbon-14 dating of the uncovered strata can indicate when the fault last ruptured. Forests of uniform age can be evidence of widespread landslides that destroyed the pre-existing forests, although floods could have a similar effect. Even patterns in tree growth rings can show evidence of perturbation of the roots at a particular time in history. Detailed information on earthquakes associated with a fault, such as the alignment of epicentres and earthquake focal mechanism solutions may assist with fault identification; this requires a degree of accuracy in the epicentral location and fault solution.

Further to the identification of fault locations using some or all of the above techniques, the activity of the fault should be evaluated. This involves an assessment of whether differential movement is occurring in the vicinity of a fault boundary, and whether there are instrumentally recorded or historical earthquakes associated with the fault. Ideally, an active fault is characterised by its slip rate, slip per event, rupture length, earthquake size and recurrence interval (Kramer, 1996). For many seismic hazard analyses, such detailed information is not easily available. Expert judgement must be exercised when considering whether a certain fault may be capable of future rupture or not.
2.2 Surface fault rupture

Structures or facilities located on or near to surface fault breaks can experience severe damage and deformation caused by the rupture or distortion of the ground. The area of a fault plane, and thus the amplitude of the slip, is closely related to earthquake magnitude. Large earthquakes can produce fault ruptures that are hundreds of kilometres long. Offsets exceeding 9 m were observed on the Chelungpu Fault during the 1999 Chi-chi earthquake in Taiwan (O'Rourke et al., 2000a). Not all earthquakes will generate fault rupture on the Earth’s surface; those at greater depths or of lower magnitude are unlikely to, this is referred to as blind faulting. Also, where there is a significant thickness of soils overlying rock there may be little evidence of rupture at the surface. Offshore earthquakes, such as those in subduction zones, may generate surface ruptures that can trigger tsunami waves (see Section 2.3), but direct damage is less likely. Surface fault ruptures may be accompanied by significant crustal deformation, which will decrease with distance from the fault, but will affect a significantly wider zone than the fault itself.

The definition of surface fault rupture hazard requires the following:

- The location of the active fault, defined by Jackson (2001) as any fault that is moving or is considered capable of moving at the present time.

- The dimensions that will influence the extent of direct damage. These are the rupture length, the expected offset at the ground surface, which will be greatest closest to the centre of energy release, and the width of the deformation field around the fault.

Wells and Coppersmith (1994) present the empirical correlations between fault rupture dimensions, slip and earthquake magnitude. The average displacement for any slip type, from a database of earthquakes with magnitudes between 5.6 and 8.1 is given as:

$$\log D = -4.80 + 0.69M_w$$  \hspace{1cm} (2-1)

where $D$ is the average surface fault displacement (m) and the standard deviation is 0.36. Different coefficients are presented for particular fault slip types (Figure 2-1), and for the maximum surface displacement.

Fault displacement hazard analyses can be deterministic, i.e. based upon the maximum expected earthquake on a single known fault, with known dimensions, or probabilistic. Youngs et al. (2003) present a method that represents the current state-of-the-art in this field, and allows
owners to specify an acceptable level of damage; their methodology incorporates the uncertainty in variables such as the fault length and the calculation of fault slip to produce a probability density function (PDF) of expected fault displacement. For either approach, the identification, location and classification of active faults, described in the preceding section, are fundamentally important.

Where the location of active faults is known, planning regulations may specify a 'no-build' zone in order to protect against damage. For regionally distributed facilities, such as pipelines and transportation networks, it may not be possible to avoid crossing faults. Furthermore, the presence of existing buildings on or near to active faults must be considered in loss modelling.

2.3 Tsunamis

Tsunamis are sea waves triggered by large-magnitude, shallow-depth oceanic earthquakes (amongst other causes). These waves can travel for thousands of kilometres, at speeds up to 1000 km/hr, without significant energy losses. This creates an added complication when incorporating tsunamis into loss models, as tsunamis may be triggered by earthquakes very distant from other earthquake sources considered to affect a study area. The waves, by the time they hit coastlines, can be tens of metres high, and can cause widespread destruction to coastal settlements and ports. The likelihood of tsunami hazard is related to the location and faulting characteristics of the earthquake and the topography and near-shore bathymetry of the coastal shelf.

2.4 Parameters for the definition of ground shaking hazard

Ground shaking is the result of seismic waves propagating through the Earth’s crust due to fault rupture. The shaking places horizontal and vertical loading onto a structure and it is the ability (or lack of) of the structure to support these loads that dictates the level of damage it sustains. The amplitude, frequency content and duration of the ground shaking together describe the demand of an earthquake on a structure. Ground motions can be characterised using either macroseismic intensity scales or instrumental parameters, both of which are explained in this section.

2.4.1 Intensity scales

Macroseismic intensities are qualitative measures based upon human perceptions of shaking, and the observed response or damage level in man-made structures, described using index scales. The European Macroseismic Intensity scale, EMS 98 (Grünthal, 1998) is reproduced in
Appendix B. The descriptions of damage grades shown in Appendix B are useful not only for determining the intensity, but also for classifying damage levels in field surveys, and describing predicted damage levels from loss estimation. Although intensity scales are implicitly related to the amplitude, frequency and duration of an earthquake ground motion, there is no means of directly extracting these parameters from intensity values.

Since intensity scales correlate directly to observed damage in structures, they are, in this respect, sometimes considered a natural choice for use in loss assessments. One problem related to the use of intensity for assessment of seismic hazard is the differences in building types, both regionally and temporally. The Modified Mercalli index (MMI), for example, was modified to apply to Californian conditions in 1931 (Richter, 1958), and is to a large extent obsolete when considering the assessment of modern engineered structures (Musson, 2000).

Further shortcomings related to the use of intensity scales for any kind of engineering assessment are that they are non-linear and non-continuous. The form of the relationships used to predict ground motion at a site (see Section 2.5) assumes a continuous variable, and it is incorrect to make this assumption with respect to intensity. A further disadvantage is that intensity values “are not directly applicable to engineering analysis” (ATC, 1985). This means that any motion-damage relationships using intensity are based on empirical data and judgement only. The use of intensity scales as a ground-motion parameter in earthquake loss estimations increases the uncertainty of the results because of this subjectivity. Furthermore the incorporation of local soil effects into an intensity-based hazard assessment is, at best, arbitrary (see Section 2.7). Despite these shortcomings, intensity scales remain widely used by practitioners of seismic risk assessment (e.g. Musson, 2000), although a need to update many of the existing vulnerability-damage models, which are currently only available in terms of intensity, has been recognised (Faccioli et al., 1999).

2.4.2 Instrumental parameters

Physical parameters that can be obtained directly from strong-motion records are considerably less subjective than intensity scales and therefore have lower associated uncertainties. They also have the significant advantage that they can “appropriately and easily account for the often dominating influence of site effects, and of near-field conditions” (Faccioli et al., 1999). Furthermore, being both continuous and measurable, instrumental parameters are appropriate for use in ground-motion prediction equations.
Peak values of acceleration, velocity or displacement are the single maximum values from a record, or from a pair of orthogonal horizontal records. The most commonly used parameter to define the amplitude of ground shaking at a site is the peak ground acceleration (PGA), which can be obtained directly from an acceleration time history. Although a significant improvement over the use of intensity scales, PGA does not give any indication of either the frequency content or the duration of ground-motions, which are strongly related to their damaging potential. There are many recorded incidents of very high PGA values where little associated damage to engineered structures was observed (e.g. 1.1g was recorded during the El Salvador earthquake of 13 January, 2001). PGA does not take into account the dynamic response of the building itself and therefore has almost no correlation with the damage potential of the ground motion (Bommer et al., 2002a). Since the occurrence of liquefaction and landslide are caused by the inertial forces within the ground, peak accelerations, or parameters derived thereof, tend to be reasonably well correlated to the occurrence of permanent ground deformations (GD_p or, PGD').

Peak ground velocity (PGV) is less sensitive to the high frequency component of a record than PGA, and as such may be a preferable indicator of ground-motion amplitude for the design of buildings sensitive to intermediate frequencies. For buried pipelines, the ground-shaking hazard is best defined in terms of PGV (Tromans, 2004). This has a reasonable theoretical basis, as the peak ground strain induced by a simple plane wave can be shown to be proportional to the peak particle velocity. The correlation of PGV with pipeline damage due to transient ground movements is also supported by field evidence: GIS-based analysis following both the 1994 Northridge and 1995 Kobe earthquakes showed pipeline damage to correlate better with PGV than with PGA (Isoyama et al., 2000; O'Rourke et al. 2001). Peak ground displacements (GD_t') are related to the low-frequency part of the ground-motion and are therefore relevant to long-period structures such as bridges or very tall buildings.

Response spectra are preferable to peak values as measures of ground shaking for design and assessment, since they represent both the amplitude and the frequency content. The use of response spectra, whether elastic or inelastic, has become state-of-the-practice in earthquake engineering, and most code-based designs use elastic response spectra as a starting point and make allowances for ductility and damping levels.

---

1 Notation. Permanent ground deformations, resulting from ground failure, are often referred to as PGD. Peak ground displacements, consistent with the terminology of PGA and PGV are also commonly referred to as PGD. In this thesis, the less common, but less confusing notations of GD_p (permanent) and GD_t (transient) are used respectively,
'Pseudo' spectral accelerations and velocities, for low levels of damping (less than 20%) can be obtained from displacement spectra as follows:

\[
\begin{align*}
PSA &= SD \cdot \frac{2\pi}{T} \\
PSV &= PSA \cdot \frac{2\pi}{T} = SD \cdot \left(\frac{2\pi}{T}\right)^2
\end{align*}
\]  

(2-2)  

(2-3)

Although the use of response spectra is accepted as being theoretically the most appropriate ground-motion characterisation for loss estimations (ATC, 1985), some of the perceived difficulties in its application are described below:

- Given the complexity and uncertainty of regional ground-motion variation and soil response, attempts to carry out detailed predictions on a large scale are sometimes perceived as "inappropriate and impractical" (Musson, 2000). Using multiple parameter ground-motion characterisations rather than single-valued indices requires significantly greater computational effort (ATC, 1985).

- From past earthquakes, there are large amounts of data correlating damage to intensity, a smaller amount correlating damage to measured or estimated PGA, and even less correlating damage to measured spectral accelerations. The shortage of empirical data is counteracted to some extent by improved capabilities in terms of engineering analysis based upon response spectra. The difficulties in verification of loss models through calibration to real data still present a major problem.

- There are fewer published attenuation relationships (see Section 2.5) for spectral ordinates than for PGA, and these are based upon more limited data (Whitman et al., 1997). This shortcoming is also becoming less critical with time as more relationships are developed.

Notwithstanding the above, recent developments in loss estimation methodologies, described in Chapter 4 recognise the importance of reproducing more accurately the current procedures used in engineering design and evaluation of specific buildings. Some of these developments make use of displacement-based spectra (Figure 2-2), recognising that structural damage is more closely related to displacement than to forces. This allows the response of structures to be calculated using engineering analysis as well as representing the ground-motion completely and therefore has a number of advantages over the use of either intensity or peak amplitudes.
However, the change towards this methodology is slow, and many practitioners retain the use of intensity scales or PGA for the reasons described above.

Figure 2-2: Displacement response spectra for different damping levels from the 1999 Hector Mine earthquake, California. At very long periods, the spectra all converge to the peak ground displacement. (Bommer & Boore, 2004)

2.5 Ground-motion prediction relationships

Ground-motion prediction relationships, also referred to as attenuation relationships, are used to describe the change in earthquake ground-motion with magnitude and distance.

Ground-motion prediction relationships have been derived for many parameters, including PGA, PGV and GD₁, as well as spectral values of acceleration, velocity and displacement, macroseismic intensity and Arias intensity. The largest body of relationships is for PGA. Douglas (2004) presents a comprehensive summary of published relationships (over 150 for PGA, and 97 for spectral ordinates), although due to the large body of work in this field such a summary is likely to be out of date very soon after publication. Relationships are generally developed either on a regional basis, or from similar seismic environments, such as subduction zones, or intraplate regions (i.e. regions located away from any active plate boundaries).

The earthquake magnitude and the source-to-site distance are always included in ground-motion prediction relationships; other factors that may also be included are local soil conditions, and the style-of-faulting. The incorporation of site effects using coefficients in predictive relationships is discussed in more detail in Section 2.7.2. Style-of-faulting, when included, has been found to have a negligible effect on the standard deviation of the relationships (Bommer et al., 2003).
Chapter 2  Seismic Hazards and their Assessment

The uncertainty associated with attenuation relationships is an extremely important feature for seismic hazard assessments. This comes from the large scatter within any database of strong-motion records used to derive the relationships, and the use of a relatively simple model to represent a very complex phenomenon. The scatter is represented by the standard deviation, $\sigma$, of the residuals about the predicted parameter. The median $+ 1\sigma$ value, or the 84-percentile, can commonly be of the order of 1.8 times greater than the median (50-percentile) value. The uncertainty in estimated ground-motion amplitude comes from the natural variability in ground motions, i.e. for any given earthquake magnitude, distance, local geology, fault rupture mechanism etc. no two records will be identical. This randomness is classified as the aleatory uncertainty. The second element component of uncertainty, relating to incomplete knowledge and referred to as epistemic uncertainty, is discussed further in Section 2.6.3.

2.6 Seismic hazard assessment

Seismic hazard assessment involves the quantitative estimation of expected ground-shaking hazard, either at a particular site or over a particular region. The main components are the identification of seismic sources and the selection of an appropriate procedure to estimate hazard.

2.6.1 Seismic source zones

Seismic source zones are essentially bounded regions that are assumed to have uniform seismicity within the boundaries. It is assumed that each source zone can be characterised by one earthquake generating process, and that earthquakes have an equal probability of occurrence within a source zone. Seismic source zones may be linear sources, representing known active faults, or area zones, where evidence shows diffused seismic activity or where there is insufficient information to locate faults. The minimum ($M_{\text{min}}$) and maximum magnitude ($M_{\text{max}}$) need to be defined; where $M_{\text{min}}$ is the minimum value below which it is considered that events will be of no engineering significance (for simplicity) and $M_{\text{max}}$ is the maximum value that can physically be expected to occur within each zone, based upon either the length and slip rate of the fault, or on past seismicity in the zone.

2.6.2 Deterministic hazard assessment methodology

The term deterministic signifies a cause and effect procedure, whereby the same set of input parameters will always produce the same outcome. At its simplest level, a deterministic hazard assessment will identify a single earthquake, referred to as a ‘scenario earthquake’, based upon a
seismic source considered most likely to affect the site or region in consideration. Multiple scenarios may also be considered, depending upon the seismic characteristics of the study area and the design or assessment requirements of the facility. The scenario earthquake will be defined by its magnitude, distance to site, fault characteristics and possibly rupture direction (Abrahamson, 2000). From these parameters, the ground motion can be estimated in a straightforward manner using carefully selected ground-motion prediction relationships (see Section 2.5).

A deterministic earthquake scenario has no recurrence interval associated with it, although by implication, its selection as a design criterion assumes that the earthquake will occur during the design life of the facility under consideration (Bommer, 2002). Potentially, structures within the same site but with different importance levels could therefore be designed to withstand the same deterministic event. Any number of deterministic scenarios may be selected, usually one in each source zone will be considered, although such decisions are made subjectively. Where there is sufficient data to characterise the activity of the region, the recurrence interval of deterministic events can be estimated.

The design ground-motion assessed in this way is generally described as the maximum credible earthquake (MCE). In simple terms, the MCE is the biggest earthquake that can reasonably be assumed to occur at any time in the future. Having defined the maximum credible earthquake and the closest distance to the site, a decision must also be made as to what level of confidence in the predicted ground-motion to adopt, i.e. how many standard deviations above the median predicted value from the selected ground-motion prediction relationship. An element of judgement is usually required for this decision, bearing in mind that building owners should be ultimately responsible for making such risk-based decisions; not engineers or seismologists.

### 2.6.3 Probabilistic hazard assessment methodology

The methodology used for probabilistic seismic hazard assessments (PSHA), originally proposed by Cornell in 1968, is considerably more complex than that of a deterministic assessment, in that it involves more steps and requires a number of decisions to be made. A PSHA involves "computing how often a suite of specified levels of ground motion will be exceeded at the site" (Abrahamson, 2000). Where DSHA considers selected combinations of magnitude, distance and standard deviation, a PSHA considers all possible combinations.

Seismic source zones, as described in Section 2.6.1, are identified the same way irrespective of the chosen forecasting model. The subsequent steps of a probabilistic hazard assessment are
explained briefly below. A comprehensive, up-to-date description of the methodology is given by McGuire (2004). A very important aspect of any probabilistic seismic hazard assessment is the selection of an appropriate probability of exceedance during a facility’s design life; in the use of code-specified design levels such as 10% in 50 years it is important to understand the rationale and risk criteria behind this level (Bommer, 2004).

From the defined seismic sources and the earthquake catalogue for the region, an analysis is carried out to define the number of earthquakes (N) of magnitude greater than or equal to M each year. The relationship between the magnitude and the frequency of occurrence can be represented by the following logarithmic relationship (Gutenberg & Richter, 1956):

$$\log(N) = A - b \cdot M$$

where A is the parameter defining the activity, and b defines the slope of the recurrence relationship and describes the relative proportions of small and large earthquakes (Figure 2-3).

![Figure 2-3: Idealised earthquake recurrence relationship (Dowrick, 1987)](image)

Ground-motion prediction relationships are then selected that are appropriate to the tectonics and seismicity of the region. The frequency of a specified ground-motion level being exceeded is then calculated, firstly by determining the likelihood that it will be exceeded for a given magnitude-distance-standard deviation combination, and then multiplying this by the annual frequency of such an event occurring within each source zone. This is repeated for each possible combination, and the overall annual frequency of exceedance is obtained by summing these results.

A logic tree is often used in PSHAs in order to incorporate the epistemic uncertainties arising from incomplete knowledge, for example regarding the delineation of source zones, the
selection of ground motion prediction relationships or the specification of a maximum magnitude for a zone. Logic trees require discrete alternatives to be specified for uncertain variables, with the relative likelihood that each alternative is the correct value represented by a weighting factor of the input parameter (Figure 2-4). The overall confidence level in the results is obtained by summing along each branch using the weightings given, the sum of all branches being equal to unity thus sampling all possible outcomes. The weighting factors are typically based on subjective judgement, (referred to as ‘degree of belief probability’ by Vick, 2002) rather than frequency-based probabilities, mainly because there are insufficient data and knowledge to allow an objective statistical analysis. Incorporation of epistemic uncertainty is for this reason a more complex procedure than modelling aleatory uncertainty, which can be done from frequency-based statistical analysis. The difference between the two is that more information and better understanding could not reduce the aleatory uncertainty to zero; it could only make its determination more accurate, whereas with sufficient investigation and knowledge, the epistemic uncertainty could be removed. The logic tree is an important, but also a highly contentious component of probabilistic seismic hazard assessments; it is common to observe that no two ‘experts’ are likely to achieve complete agreement regarding these subjective judgements (Bommer, 2002).

![Figure 2-4: Example logic tree for a PSHA for Hong Kong by Free et al., (2004). Weightings are shown in parentheses.](attachment:logic_tree.png)

The principle feature of PSHAs as opposed to deterministic studies is that the results, presented in the form of uniform hazard response spectra (UHRS), are representative of all earthquake occurrences that are judged to contribute to a particular level of ground-motion hazard (Figure 2-5). This can be perceived as an advantage, in that all possible scenarios are accounted for, but
also presents a disadvantage, in that there is no ‘design earthquake’, i.e. a single event, defined in terms of magnitude and distance, that is representative of the design spectrum. For many applications, a scenario earthquake is essential; for example, the selection of suitable time histories for design requires the magnitude and distance to be known, as do other applications such as liquefaction assessment.

![Uniform Hazard Spectrum](image)

**Figure 2-5:** Illustration of the concept of a uniform hazard spectrum being an envelope of many possible earthquake scenarios affecting a site (Reiter, 1990)

This disadvantage can be overcome through disaggregation of a uniform hazard spectrum in order to obtain the magnitude-distance-standard deviation combinations that contribute the most to the ground-motion hazard. Methodologies for disaggregation are presented by McGuire (1995) and Bazzurro & Cornell (1999).

### 2.6.4 Selection of deterministic or probabilistic approach

Table 2-1 gives examples of different end requirements of a seismic hazard assessment, and the predominant methodologies adopted for such requirements. This table is indicative only, and McGuire (2001) also states that “the most insightful assessment of seismic hazard and risk will be made through recursive analysis” wherein a combination of probabilistic and deterministic methodologies is adopted to provide a sufficiently detailed model. Furthermore, as McGuire points out, “a proper probabilistic analysis must include all credible deterministic scenarios, to itself be credible”. For loss assessments, the end requirement may be one or several of the options in Table 2-1 and so either approach may be adopted. Another factor in the selection of the most appropriate model is the seismicity of the study area. Where seismicity is low, and the determination of seismic source zones has a high degree of uncertainty associated with it, the use of a deterministic approach may be less appropriate.

**Table 2-1:** Examples of selection of hazard assessment methodology (McGuire, 2001)
<table>
<thead>
<tr>
<th>Decision</th>
<th>Quantitative Aspects of Decision</th>
<th>Predominant Approach</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic design levels</td>
<td>Highly quantitative</td>
<td>Probabilistic</td>
</tr>
<tr>
<td>Retrofit design</td>
<td>Highly quantitative</td>
<td>Probabilistic</td>
</tr>
<tr>
<td>Insurance/reinsurance</td>
<td>Highly quantitative</td>
<td>Probabilistic</td>
</tr>
<tr>
<td>Design of redundant industrial systems</td>
<td>Quantitative or qualitative</td>
<td>Both</td>
</tr>
<tr>
<td>Training and plans for emergency response</td>
<td>Mostly qualitative</td>
<td>Deterministic</td>
</tr>
<tr>
<td>Plans for post-earthquake recovery</td>
<td>Mostly qualitative</td>
<td>Deterministic</td>
</tr>
<tr>
<td>Plans for long-term recovery, local</td>
<td>Mostly qualitative</td>
<td>Deterministic</td>
</tr>
<tr>
<td>Plans for long-term recovery, regional</td>
<td>Mostly quantitative</td>
<td>Probabilistic</td>
</tr>
</tbody>
</table>

Earthquake losses estimated on the basis of PSHA, whilst allowing the average annual losses to be estimated for the purposes of insurance premiums, for example, do not provide any useful information on how the exposed stock is likely to be affected for one anticipated event and do not allow magnitude and distance dependent features such as duration of shaking and damping to be rigorously incorporated into building vulnerability models. Deterministic results on the other hand, whilst being useful in providing this information, and therefore important for the preparation of emergency response plans for example, do not allow financial planning, because they provide only one or several sets of results, which have no associated frequency of occurrence. The use of multiple scenario earthquakes to represent the hazard for a loss estimation model has been proposed (e.g. Bommer et al., 2002a), which solves some of these problems, whereby the effect of each individual earthquake, with an estimated frequency of exceedance, is considered in turn, and then the resulting damage data is analysed in order to obtain annual loss estimates. Although requiring a much greater number of calculations than either a single deterministic scenario or PSHA, this approach has the following advantages (Bommer et al., 2002a):

- A known magnitude and distance combination, rather than being hidden behind the uniform hazard spectra from PSHA, is the preferred input to assessment of liquefaction and landslide hazard and analytical site response assessments.
- Damage calculations can similarly be based upon transparent magnitude, duration and spectral shape, unlike in the case of PSHAs.
Scenario losses for planning purposes may be directly extracted from the results, with no additional calculations required.

Multiple scenarios for hazard and risk calculations may be derived in a deterministic manner, based upon judgement regarding likely earthquakes associated with different faults and seismic source zones. Alternatively stochastic modelling can be carried out, using the seismic source model, which defines the source zones and their magnitude-recurrence relationships, and a Monte Carlo process to generate random synthetic earthquake catalogues that satisfy the assumption of uniform seismicity within each source zone (Musson, 1999). The latter approach combines the advantages of probabilistic modelling, in considering all contributions including uncertainty, to the hazard affecting a site with the ability to model the effect of individual earthquakes on the built environment.

2.7 Site effects

The effect of local soil conditions upon the bedrock ground-motion is well known. As illustrated in Figure 2-6, soft or loose near surface soils will generally amplify the long-period response, whereas high-frequency components of strong shaking will be de-amplified. The presence of soft soils is therefore potentially most damaging for tall or flexible structures. This is well illustrated by the 1985 Michoacan earthquake, where, despite the fault rupture being over 400 km from Mexico City, the presence of deep, soft, lacustrine deposits beneath the city served to amplify the ground shaking significantly. The result was a concentration of damage to buildings of between seven and eighteen storeys, whose natural period coincided with the dominant period of the ground motion.

![Figure 2-6: The effect of different soil profiles on spectral shapes (Seed & Idriss, 1982)](image)

Soil is a highly non-linear material, in which the stiffness progressively decreases with increasing shear stress and strain until, at sufficiently high strains, it will undergo large plastic
Chapter 2  Seismic Hazards and their Assessment

deformations representative of failure. Earthquake loading can encompass a much greater range of strains than considered for static design. Except at very small strains, soil behaves inelastically, and therefore exhibits hysteretic damping when loaded cyclically, which increases with increasing shear strain. The response of loose saturated cohesionless soils to cyclic loading may lead to liquefaction, described in Section 2.9. These complexities of soil behaviour must be taken into account in any assessment of seismic hazard, and the various means of doing so are presented below.

2.7.1 Incorporation of soil effects into intensity scales

Macroseismic intensity scales (Section 2.4.1) do not explicitly account for soil effects (although high observed intensities in past earthquakes are often found to be implicitly associated with poor ground conditions). Therefore, combining prediction of ground-motion intensity with knowledge of local soil conditions in order to more accurately represent actual levels of shaking at a site can only be done through some arbitrary approach, such as that proposed by Evernden and Thomson (1985) whereby increments of Modified Mercalli intensity are assigned to different geological units. Such an approach has two major shortcomings: firstly the non-linearity of intensity scales means that adding a value of, say, 0.5 to EMS VI is not the same as adding the same value to EMS IX, and secondly intensity scales are discrete indices and therefore the use of fractions of whole numbers has no meaning. However, this approach is the only way in which soil effects can be systematically introduced into intensity-based loss estimations (Spence et al., 2003). The only other option is for users to consider soil conditions on an individual site basis and apply judgement to determine the increased risk due to the presence of soft or loose ground.

2.7.2 Modelling soil effects in ground-motion prediction relationships

One of the simplest approaches to incorporate soil conditions into design response spectra is through the selection of predictive relationships that represent different site classes. Most relationships include site classification as a third variable after magnitude and distance (Douglas, 2004). The classifications can range in complexity from a simple binary classification of soil/rock, to including the shear wave velocity of the site as an independent variable within the relationship. A major drawback in the derivation of these ‘attenuation relationships’ is the lack of good quality data on the local soil conditions at the strong-motion stations, and this is the biggest factor affecting the simplicity or complexity of the derived relationships. For example, studies by Boore et al. (1993) and Ambraseys et al. (1996) define soil classification boundaries in terms of shear wave velocity, but since there are no shear wave
velocity measurements at many of the stations they use, these definitions can only be approximate (Douglas, 2001). Even so, the use of shear wave velocity is preferable to other, subjective, classifications as it is a measurable quantity and not open to interpretation by users of the relationships.

In order to include soil effects in an attenuation relationship, a simplification must be made that the degree of amplification due to soils is independent of the amplitude of the motion, because many records of different amplitude are combined for the purposes of deriving attenuation laws. Since a single soil term is generally used in such relationships, regardless of the magnitude or distance values, the product is an average of small strain (high stiffness) and large strain (low stiffness) soil behaviour, thus ignoring non-linear effects. Figure 2-7 illustrates how for softer soils at longer periods this may produce fairly significant differences compared to amplitude-dependent soil factors, particularly for strong shaking. However, the approach is straightforward to apply, and has the (debatable) advantage of only having one set of uncertainties, represented by the standard deviation of the relationships, as opposed to the other methods described below.

![Figure 2-7](image)

**Figure 2-7**: Amplification factors from the attenuation relationships of Boore et al. (1997) from the ratio of soil to rock motions (curved lines), compared to the range of amplification factors for long period motions given in the NEHRP Provisions (FEMA, 1997) described below. The lower bounds of the NEHRP factors are for the largest amplitude of ground shaking, representing non-linearity.

### 2.7.3 Code amplification factors

Most seismic design codes include site classification schemes, with factors to apply to the design rock spectra. The site classifications and soil amplification factors proposed in the NEHRP Provisions for the seismic rehabilitation of buildings (FEMA, 1997) are widely used, both in the United States and elsewhere. The soil classification scheme is presented in Table
2-2, and the amplification factors in Tables 2-3 and 2-4. The final surface response spectra based upon the NEHRP approach is thus obtained by multiplying the rock spectra by the $F_{AS}$ values from Table 2-3 over the short period range and by the $F_{AL}$ values from Table 2-4 over the constant velocity portion of the spectra (see Section 5.2.2). The use of two factors for short- and long-period components of the design spectra provides a much improved representation of the dependency of soil amplification on period and amplitude of shaking. Other codes adopt only a single factor, which fails to capture either period dependency or non-linearity.

Dobry et al. (2000) describe the derivation of the NEHRP factors, which originate from a combination of empirical data (response spectral ratios of soil site records to rock site records from the Loma Prieta earthquake) and numerical site response analyses at higher accelerations. The numerical analyses used the program SHAKE, described in Section 2.7.4, and therefore have the associated uncertainties and shortcomings related to using one-dimensional and equivalent linear analyses.

Table 2-2: Soil classification from NEHRP Provisions (FEMA 1997)

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Description</th>
<th>Average Properties in top 30m</th>
<th>Site Class</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Shear-wave velocity, $V_{S30}$ (m/s)</td>
<td>SPT N (Blows/300mm)</td>
<td>Undrained shear strength, $S_u$ (kPa)</td>
</tr>
<tr>
<td>A</td>
<td>Hard Rock</td>
<td>&gt; 1500</td>
<td>Not applicable</td>
<td>Not applicable</td>
</tr>
<tr>
<td>B</td>
<td>Rock</td>
<td>760 - 1500</td>
<td>Not applicable</td>
<td>Not applicable</td>
</tr>
<tr>
<td>C</td>
<td>Very dense soil and soft rock</td>
<td>360 - 760</td>
<td>&gt; 50</td>
<td>&gt; 100</td>
</tr>
<tr>
<td>D</td>
<td>Stiff soil</td>
<td>180 - 360</td>
<td>15 - 50</td>
<td>50 - 100</td>
</tr>
<tr>
<td>E</td>
<td>Soft soil</td>
<td>&lt; 180</td>
<td>&lt; 15</td>
<td>&lt; 50</td>
</tr>
<tr>
<td>F*</td>
<td>-</td>
<td>Profiles containing soils with one or more of the following:</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Soil vulnerable to potential collapse under seismic loading, e.g. liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils.</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>H&gt;8m of peat and/or highly organic clay</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Very high plasticity clays (H&gt;8m with PI&gt;75%)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>Very thick soft/medium stiff clays (H&gt;36m)</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

* Site class F requires site specific investigation. For the purposes of regional hazard mapping, site class F is ignored (e.g. see HAZUS, FEMA, 2003).

Where: $S_u =$ undrained shear strength  
SPT N = standard penetration test blow count
PI = Plasticity Index

w = water content.

Table 2-3: Values of $F_{AS}$ as a function of site class and short-period maximum considered earthquake spectral acceleration, $S_{AS}$ (after FEMA, 1997)

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Mapped Maximum Considered Earthquake Spectral Response Acceleration at Short Periods</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$S_{AS} \leq 0.25$</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.2</td>
</tr>
<tr>
<td>D</td>
<td>1.6</td>
</tr>
<tr>
<td>E</td>
<td>2.5</td>
</tr>
<tr>
<td>F</td>
<td>*</td>
</tr>
</tbody>
</table>

Table 2-4: Values of $F_{AL}$ as a function of site class and 1.0 second period maximum considered earthquake spectral acceleration, $S_{AL}$ (after FEMA, 1997)

<table>
<thead>
<tr>
<th>Site Class</th>
<th>Mapped Maximum Considered Earthquake Spectral Response Acceleration at 1 Second Periods</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$S_{AL} \leq 0.1$</td>
</tr>
<tr>
<td>A</td>
<td>0.8</td>
</tr>
<tr>
<td>B</td>
<td>1.0</td>
</tr>
<tr>
<td>C</td>
<td>1.7</td>
</tr>
<tr>
<td>D</td>
<td>2.4</td>
</tr>
<tr>
<td>E</td>
<td>3.5</td>
</tr>
<tr>
<td>F</td>
<td>*</td>
</tr>
</tbody>
</table>

The use of site classifications and amplification factors is well suited to regional hazard studies, since zoning can be undertaken using a reasonable level of input data, and the amplification factors may be applied to the bedrock response spectra using simple algorithms in database or GIS environments. However, given the potential importance of site effects, there are some inconsistencies in this approach. The schemes are generally based upon the properties of the top 30m of the soil only and may therefore overlook particular features of the deep geology. Furthermore, these amplification factors have a very significant scatter associated with them, due to the complex behaviour of soils under cyclic loading, and the need to group many sites within a single classification (Figure 2-8). Although it is customary to include the standard deviation of the selected ground-motion prediction relationships (Section 2.5), only the median
values of the amplification factors are generally considered, and further review may be required to ensure that the potential variability of the site amplification is dealt with appropriately.

![Figure 2-8: Relationships between maximum accelerations on rock and soil sites (Idriss, 1991). This figure demonstrates the non-linear behaviour of the soft soil sites, whereby at very high accelerations the rock motions are de-amplified. The uncertainty associated with the use of a median factor only is also illustrated.](image)

### 2.7.4 Analytical modelling of site response effects

The final and most rigorous approach to incorporating soil effects into a seismic hazard assessment is to carry out site response analyses, although even these can vary in complexity. Site response analyses will compute the response of a characterised soil profile above bedrock to a given bedrock ground-motion. They require more detailed input data than the two previously described methods. Sufficient borehole data are required to develop a model of the average stratigraphy at the site, and the small strain and dynamic soil properties require a more sophisticated testing and investigation procedure than may be carried out in a standard site investigation. The depth to bedrock or to the 'equivalent bedrock', a stiff layer at which the input motion is estimated, is required which may not be encountered using standard drilling equipment.

The simplest site response analyses are one-dimensional, modelling the vertical propagation of horizontal seismic shear waves through infinitely wide uniformly horizontal soil layers, subject to an earthquake motion at the base of the column. The soil profile is characterised by its density, strength, stiffness and the non-linear stiffness degradation with strain (e.g. Figure 2-9).

SHAKE (Schnabel et al., 1976, Idriss & Sun, 1992), a widely used and accepted 1-d site response analysis program is an equivalent-linear model, using an iterative procedure to obtain the average secant stiffness (i.e. an equivalent linear stiffness) for a given strain level over each
cycle of loading (Figure 2-10). The equivalent linear method has been found to give reasonable results at strains of less than 2% and accelerations less than 0.3 to 0.4g (Kramer & Paulsen, 2004).

![Figure 2-9: Typical stiffness degradation curves for clays of different Plasticity Indices (%)](Vucetic & Dobry, 1991)

At higher accelerations the non-linear effects become more important, and the equivalent linear assumption may not provide sufficient accuracy. One-dimensional non-linear soil models exist (e.g. DESRA, Lee & Finn, 1978) that solve the equations of motion through direct integration in the time domain. Such methods are often necessary where large strains or displacements are expected. The computer program SIREN, developed by J.W. Pappin of Ove Arup & Partners has been used in this research. This is a non-linear program, which solves the problem in the time domain using finite differences (Heidebrecht et al., 1990; Henderson et al., 1990). This program has been compared with SHAKE by its developers, and the two were found to agree.
well at lower strain levels, with SIREN predicting surface accelerations about 10% lower at larger strain values, due to the non-linear model. SIREN does not, unlike some non-linear programs, include the build up of pore-water pressure.

One-dimensional analyses are computationally efficient, and the input data is relatively straightforward to define. For level ground or gently sloping sites and homogeneous soil layers they are a reasonable simplification. FLUSH (Lysmer et al., 1975) follows a similar methodology to SHAKE, but uses two-dimensional models, in order to include the effects of non-homogeneous soil layers either in properties or stratigraphy, and basin effects.

It is unusual, but not impossible, for sufficient site investigation data to be available on a regional scale to use analytical procedures to assess site response for loss assessments. The study of regional site amplification patterns for Hong Kong, described in Chapter 6, shows how this can be done. Uncertainties in this type of analysis relate to the selection of the model, the definition of dynamic soil properties, and the selection of appropriate strong-motion records for their input. There is no uniform agreement within the earthquake engineering profession as to how many records should be selected or which parameters are the most important for use as selection criteria (Bommer & Acevedo, 2004). Nonetheless, the uncertainties associated with site response analyses are significantly less than those related to the use either of site classifications in attenuation relationships, or code-based amplification factors, and they can be incorporated in a rigorous manner through the use of either probabilistic methods or sensitivity studies.

### 2.8 Topographic amplification

Topographic surface features such as ridges, canyons or slopes can affect the amplitude of the surface ground-motion. At ridgelines or cliff tops (convex features) the effect is amplification of the motion due to focussing of the seismic waves; in valleys or canyons, de-amplification can occur due to de-focussing. In addition to amplifying the ground-motion at the base of the buildings located on slopes, the topographic site effect during an earthquake may contribute to the reactivation of landslides and rockslides (Paolucci, 2002). Amplification has been observed to be greater where the wavelength is of similar magnitude to the length of the feature (e.g. the ridge length), and on steeper or higher slopes. There is no precedent for rigorously including such effects in loss models, or even in hazard studies (Stewart et al. 2001), particularly where topographic features are irregular. Simplified approaches, using amplification factors, are presented in seismic codes including Eurocode 8 and the French seismic design code (PS-92).
2.9 Liquefaction

Liquefaction hazard and the associated risk to buildings and infrastructure is the main subject of this thesis. In this section, the principles of liquefaction and its assessment are presented. Particular issues related to earthquake loss estimations are discussed in subsequent chapters.

Liquefaction can lead to flow failure (the horizontal movement of the liquefied soil mass), lateral spreading caused by incremental lateral movements during the earthquake shaking, and vertical settlements. The significance of liquefaction during an earthquake can be very variable and a detailed review of damage and losses related to liquefaction is presented in Chapter 3.

Figure 2-11: Flow chart for the assessment of the potential of earthquake-induced liquefaction to damage structures, lifelines and other facilities

The potential for liquefaction to cause damage is assessed in three stages: the susceptibility to liquefaction; the likelihood of liquefaction being triggered by the design earthquake scenario; and the consequences related to liquefaction (Figure 2-11). The onset of liquefaction is influenced by:

- Environmental factors, such as the location of the water table.
- Site stratigraphy and depositional and seismic loading history of the soil.
- Soil characteristics such as relative density, grain size distribution, mineralogy, and the presence of any cementing agents.
• The characteristics of the earthquake under consideration, mainly the amplitude of ground shaking and its duration.

2.9.1 The principles of liquefaction

When a saturated cohesionless soil is cyclically loaded, its particle structure can tend to collapse to a denser arrangement. If the soil's permeability and the site stratigraphy are such that drainage cannot occur during rapid earthquake loading, then as the collapse occurs, stresses will be transferred from the soil grain contacts to the pore water, leading to an increase in pore water pressure. In simple terms, when the pore water pressure increases, the effective stress on the particle structure will reduce, and as it approaches zero, the shear resistance of the soil will also approach zero.

![Diagram of Stress-strain behaviour of contractive and dilative soils under monotonic and cyclic loading](image)

**Figure 2-12:** Stress-strain behaviour of contractive and dilative soils under monotonic and cyclic loading. $\tau_0$ = initial (static) shear stress. In all cases the steady state undrained shear strength is ultimately reached. (NRC, 1985)

The stress-strain behaviour of liquefying sand depends strongly on its relative density, as illustrated in Figure 2-12. When loose sand liquefies, the gravitational static shear stresses may exceed the shear resistance of the soil and rapid deformation with very large shear strains can commence; this is referred to as *flow deformation*. The soil behaviour is termed *contractive*, and the shear resistance exhibited by the liquefied soil during flow deformation is termed the *residual strength*. When moderately dense granular soils are cyclically sheared, pore pressures may similarly rise and liquefaction can be triggered. Rather than undergoing flow deformation however, the soil particle structure may try to expand, or *dilate* as it reaches a certain level of shear strain, termed *dilative* behaviour. For undrained conditions, this leads to a reduction in
the pore water pressures and a corresponding increase in effective stress and shear resistance. A shear stress reversal, however, such as will occur many times during an earthquake, may cause the soil particle structure to be incrementally contractive and the state of zero effective stress may be temporarily reached once more. This continued cycle of zero effective stress and strength regain, and the ground deformation that will accompany it, is termed cyclic mobility. The cumulative deformations can be significant, particularly if the duration of shaking is long, but dilative soils do not exhibit very large flow deformations in the way that contractive soils do.

Sand boils are caused by the tendency of the excess pore water pressures to dissipate upwards, towards the free surface, carrying soil particles up from the liquefied layer through cracks or channels in the overlying material and ejecting them at the ground surface (Figure 2-13). Sand boils in themselves are not necessarily damaging to structures.

**Figure 2-13:** Sand boil in liquefied soil following an earthquake, Japan, 1983 (Photograph courtesy I.M. Idriss)

**Figure 2-14:** Schematic illustration of void redistribution resulting from upward seepage of pore pressure impeded by a lower permeability overlying layer (after NRC, 1985)
The upward seepage of pore water, driven by the earthquake-induced excess pore pressures, can be impeded by less permeable overlying soil layers, which can result in water accumulating near the interface between the liquefied soil and the overlying lower-permeability soil. The accumulation of water can loosen the soil and possibly even result in the formation of water films, either of which greatly reduces the available shear resistance along the interface (Figure 2-14). This phenomenon is referred to as void redistribution. A key consequence of void redistribution is that the residual shear strength of liquefied soil does not just depend on the pre-earthquake properties of the soil, but rather also depends on those factors affecting the dissipation and movement of pore water following earthquake shaking (Boulanger & Idriss, 2004). Such features are likely to be implicitly included in procedures based upon empirical observations.

Liquefaction can also affect the characteristics of the strong ground-motion. The accelerogram in Figure 2-15 clearly shows the occurrence of liquefaction beneath the instrument; it also shows that there are some strong peaks in the ground shaking before the onset of liquefaction, and some longer period motions subsequently.

![Figure 2-15: Horizontal accelerograms recorded in Niigata, Japan in the 1964 earthquake. The area surrounding the recording station was heavily damaged by liquefaction. The onset of liquefaction appears to have occurred at approximately 10s after strong ground shaking commenced.](image)

### 2.9.2 Liquefaction susceptibility

The most obvious evidence that a particular deposit is susceptible to liquefaction is historical precedence, either at the same site or in similar conditions elsewhere; it is not unusual for liquefaction to re-occur in the same location. In regions of potential hazard where there is no
historical evidence, other information must be used to make an initial assessment of the liquefaction hazard.

**Table 2-5: Susceptibility of sedimentary deposits to liquefaction during strong seismic shaking**  
(after Youd & Perkins, 1978)

<table>
<thead>
<tr>
<th>Type of deposit</th>
<th>Distribution of cohesionless sediments in deposit</th>
<th>Likelihood that cohesionless sediments, when saturated would be susceptible to liquefaction</th>
<th>&lt;500 year</th>
<th>Holocene</th>
<th>Pleistocene</th>
<th>Pre-Pleistocene</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>a) Continental Deposits</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>River Channel</td>
<td>Locally variable</td>
<td></td>
<td>Very High</td>
<td>High</td>
<td>Low</td>
<td>Very Low</td>
</tr>
<tr>
<td>Flood Plain</td>
<td>Locally variable</td>
<td></td>
<td>High</td>
<td>Moderate</td>
<td>Low</td>
<td>Very Low</td>
</tr>
<tr>
<td>Alluvial fan and plains</td>
<td>Widespread</td>
<td>Moderate</td>
<td>Low</td>
<td>Low</td>
<td>Very low</td>
<td></td>
</tr>
<tr>
<td>Marine terraces and plains</td>
<td>Widespread</td>
<td>-</td>
<td>Low</td>
<td>Very low</td>
<td>Very low</td>
<td></td>
</tr>
<tr>
<td>Delta and fan-delta</td>
<td>Widespread</td>
<td>High</td>
<td>Moderate</td>
<td>Low</td>
<td>Very low</td>
<td></td>
</tr>
<tr>
<td>Lacustrine and playa</td>
<td>Variable</td>
<td>High</td>
<td>Moderate</td>
<td>Low</td>
<td>Very low</td>
<td></td>
</tr>
<tr>
<td>Colluvium</td>
<td>Variable</td>
<td></td>
<td>High</td>
<td>Moderate</td>
<td>Low</td>
<td>Very low</td>
</tr>
<tr>
<td>Talus</td>
<td>Widespread</td>
<td></td>
<td>Low</td>
<td>Low</td>
<td>Very low</td>
<td>Very low</td>
</tr>
<tr>
<td>Dunes</td>
<td>Widespread</td>
<td></td>
<td>High</td>
<td>Moderate</td>
<td>Low</td>
<td>Very low</td>
</tr>
<tr>
<td>Loess</td>
<td>Variable</td>
<td></td>
<td>High</td>
<td>High</td>
<td>High</td>
<td>Unknown</td>
</tr>
<tr>
<td>Glacial till</td>
<td>Variable</td>
<td></td>
<td>Low</td>
<td>Low</td>
<td>Very low</td>
<td>Very low</td>
</tr>
<tr>
<td>Tuff</td>
<td>Rare</td>
<td></td>
<td>Low</td>
<td>Low</td>
<td>Very low</td>
<td>Very low</td>
</tr>
<tr>
<td>Tephra</td>
<td>Widespread</td>
<td></td>
<td>High</td>
<td>High</td>
<td>?</td>
<td>?</td>
</tr>
<tr>
<td>Residual soils</td>
<td>Rare</td>
<td></td>
<td>Low</td>
<td>Low</td>
<td>Very low</td>
<td>Very low</td>
</tr>
<tr>
<td>Sebka</td>
<td>Locally variable</td>
<td></td>
<td>High</td>
<td>Moderate</td>
<td>Low</td>
<td>Very low</td>
</tr>
<tr>
<td><strong>b) Coastal Zone</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Delta</td>
<td>Widespread</td>
<td></td>
<td>Very high</td>
<td>High</td>
<td>Low</td>
<td>Very low</td>
</tr>
<tr>
<td>Estuarine beach/</td>
<td>Locally variable</td>
<td></td>
<td>High</td>
<td>Moderate</td>
<td>Low</td>
<td>Very low</td>
</tr>
<tr>
<td>High wave energy</td>
<td>Widespread</td>
<td></td>
<td>Moderate</td>
<td>Low</td>
<td>Very low</td>
<td>Very low</td>
</tr>
<tr>
<td>Low wave energy</td>
<td>Widespread</td>
<td></td>
<td>High</td>
<td>Moderate</td>
<td>Low</td>
<td>Very low</td>
</tr>
<tr>
<td>Lagoonal</td>
<td>Locally variable</td>
<td></td>
<td>High</td>
<td>Moderate</td>
<td>Low</td>
<td>Very low</td>
</tr>
<tr>
<td>Fore shore</td>
<td>Locally variable</td>
<td></td>
<td>High</td>
<td>Moderate</td>
<td>Low</td>
<td>Very low</td>
</tr>
<tr>
<td><strong>c) Artificial Fill</strong></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Uncompacted fill</td>
<td>Variable</td>
<td></td>
<td>Very high</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Compacted fill</td>
<td>Variable</td>
<td></td>
<td>Low</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
Chapter 2 Seismic Hazards and their Assessment

Table 2-6: Susceptibility of geomorphological units to liquefaction (Iwasaki et al. 1982)

<table>
<thead>
<tr>
<th>Rank</th>
<th>Geomorphological Unit</th>
<th>Liquefaction Potential</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Present river bed, old river bed, swamp, reclaimed land, interdune lowland</td>
<td>Liquefaction likely</td>
</tr>
<tr>
<td>B</td>
<td>Fan, natural levee, sand dune, flood plain, beach, other plains</td>
<td>Liquefaction possible</td>
</tr>
<tr>
<td>C</td>
<td>Terrace, hill, mountain</td>
<td>Liquefaction not likely</td>
</tr>
</tbody>
</table>

The criteria reproduced in Table 2-5 and 2-6 classify different levels of liquefaction susceptibility for various types and ages of soil deposits. These are widely used by engineers and engineering geologists to produce regional maps of liquefaction hazard for planning purposes, or for preliminary assessment of potential hazards for site selection purposes.

Saturated loose cohesionless soils are the most vulnerable to liquefaction. Older deposits generally have higher densities and thus greater resistance to liquefaction. The mode of deposition is also an important factor, as can be seen in Table 2-5 and 2-6. Alluvial, fluvial, marine, deltaic or wind-blown deposits generally have a higher susceptibility than either residual soils or glacial tills, materials which are more highly consolidated.

There are few case histories of liquefaction occurring at depths greater than about 10-15 m. This is partly due to the fact that soils at greater depths tend to be older and denser, and therefore have a higher cyclic resistance to liquefaction. In addition, liquefaction at greater depths beneath level ground would not necessarily be evident at the ground surface, and thus may go undetected.

Table 2-7: Review of compositional criteria for soils that are vulnerable to liquefaction (Clare et al., 2002)

<table>
<thead>
<tr>
<th>Reference</th>
<th>Maximum Content</th>
<th>Clay Content</th>
<th>Plasticity Index</th>
<th>Notes/ Additional Criteria to be Satisfied</th>
</tr>
</thead>
<tbody>
<tr>
<td>El Hosri et al., 1984</td>
<td>0.002mm&lt; 20%</td>
<td>&lt; 10%</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Tokimatsu and Yoshimi, 1984</td>
<td>0.005mm&lt; 20%</td>
<td>&lt; 10%</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>Cao and Law, 1991</td>
<td>0.002mm&lt; 20%</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>AIJ, 1998</td>
<td>Clay fraction &lt; 10%</td>
<td>&lt; 15%</td>
<td>Liquefies if either criterion is met</td>
<td></td>
</tr>
<tr>
<td>Wang, 1981</td>
<td>0.005mm&lt; 15%</td>
<td>&lt; 15%</td>
<td>LL &lt;35%, $W_c &gt; 0.9LL</td>
<td></td>
</tr>
<tr>
<td>Finn et al., 1994</td>
<td>0.005mm&lt; 10%</td>
<td>&lt; 15%</td>
<td>LL &lt;36%, $W_c &gt; 0.9LL + 2%</td>
<td></td>
</tr>
<tr>
<td>Youd et al., 2001</td>
<td>Cyclic resistance increased by 10% for soils with fines and PI &gt; 15%</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Where some site specific information is available, compositional criteria can be used to make preliminary assessments of liquefaction susceptibility. The fines content and clay content,
Atterberg limits and particle size distribution are all relevant. While clays are generally resistant to liquefaction, it is also understood that plastic silts and clays may develop significant shear strains under sufficiently strong and sustained earthquake loading, leading to strength loss and rapid deformation or even instability. Compositional characteristics are used in various criteria, a summary of which is presented in Table 2-7, which illustrates the variability of such criteria. The so-called “Chinese Criteria” (after Wang, 1981) are frequently cited, but these have recently been shown to be potentially unconservative (e.g. Bray et al., 2004) - a classification of “non-liquefiable” using these criteria cannot be taken as evidence of the absence of a problem.

2.9.3 Evaluation of liquefaction potential

The state-of-the-practice approach for evaluating liquefaction potential at a specific site is often referred to as the simplified procedure (Seed & Idriss, 1971). This procedure compares the shear stresses induced by the earthquake with those required to cause liquefaction in the soil profile, where the liquefaction resistance is based upon empirical data. The procedure is based upon the following relationship:

\[
CSR = \frac{\tau_{av}}{\sigma_v'} = 0.65 \cdot \frac{a_{max}}{g} \cdot \frac{\sigma_v}{\sigma_v'} \cdot \frac{r_d}{MSF} \tag{2-5}
\]

where: CSR is the cyclic shear stress ratio, \(\tau_{av}/\sigma_v'\) at the depth of the soil layer under consideration

\(a_{max}\) is the peak ground acceleration at the surface, \(g\) is acceleration due to gravity

\(\sigma_v/\sigma_v'\) is the ratio of total to effective vertical stress in the soil layer

\(r_d\) is a stress reduction coefficient (Figure 2-16)

MSF is a magnitude scaling factor, equal to unity for \(M=7.5\) earthquakes, and greater than unity for \(M<7.5\), to correct for the duration of shaking.

Updated recommended procedures for the evaluation of \(r_d\) and MSF for use in conjunction with Equation 2-5 are presented by Idriss and Boulanger (2004). The state-of-the-practice for these procedures is presented in the summary of the NCEER 1997 workshop on liquefaction by Youd et al. (2001).
The factor of safety (F) against liquefaction is then determined as:

\[ F = \frac{CRR}{CSR} \]  

(2-6)

where CRR is the cyclic resistance ratio of the soil layer. The evaluation of CRR can be determined directly from laboratory tests on high quality undisturbed samples, or reconstituted samples for new fills, or indirectly by in situ testing. The collection of high quality undisturbed soil samples, using techniques such as ground freezing, and undertaking cyclic testing in the laboratory (either cyclic simple shear or triaxial tests), is an effective but expensive method of evaluating the cyclic resistance. In practice, by far the most common approach used by engineers to determine CRR is based on in situ testing, in particular the standard penetration test (SPT) and the cone penetration test (CPT). Shear wave velocity \( (V_s) \) measurements and the Becker penetration test (BPT) are also used in some specific applications.

There are now a number of publications providing guidance on the estimation of CRR using in situ measurements (e.g. Youd et al., 2001; Moss, 2003; Idriss & Boulanger, 2004; Andrus & Stokoe, 2000). Such correlations are obtained through back analysis of case history data, and as such are related to the variables used to define the CSR (Equation 2-5) for the case histories, which can vary considerably amongst different authors. For this reason, it is important to be consistent in the selected methodology, since the combination of different recommendations could lead to incorrect results. Figure 2-17 shows a range of correlations relating CRR to \( (N_1)_{60} \) for clean sands (i.e., fines content \( \leq 5\% \)). \( (N_1)_{60} \) represents the SPT blow count corrected to a hammer energy level of 60% (to compensate for variations in testing equipment and procedures).
and normalised to an effective vertical stress of one atmosphere (to remove the dependency on overburden). The significant variation between published results is apparent in this figure, particularly at high shear stresses, where there are fewer data points.

Figure 2-17: Correlation between cyclic shear stress ratio (for M = 7.5) causing liquefaction and standard penetration resistance for clean sands (Idriss & Boulanger, 2004)

The simplified procedure described above is essentially deterministic and only provides engineers with the information that liquefaction will (F<1.0) or will not (F≥1.0) occur. In many cases, particularly for damage assessments, this information alone is insufficient to make an informed decision regarding project risk. The significant scatter in the empirical data from which charts such as Figure 2-17 have been derived means that where the median values yield a factor of safety comfortably greater than 1.0, there may be an associated probability, albeit very low, of a factor of safety less than 1.0 for the same data. Probabilistic approaches towards liquefaction assessment have been developed, by, amongst others, Seed et al. (2003) and Rodríguez-Marek et al. (2001) although these are not so widely used as the deterministic methods. The selection of the most appropriate methodology must carefully consider the input data required compared to what is available. The probabilistic relationships presented by Seed et al. (2003) also use site response analyses to determine the shear stress profile, thus reducing some of the uncertainties associated with the simplified calculation of the CSR.

2.9.4 Permanent ground deformation

The extent of the permanent ground deformation as a result of liquefaction will dictate the engineering solutions in terms of mitigation or re-design. For loss estimations, it is the extent of permanent ground deformation that is the most important parameter, since the assessment of
liquefaction potential on its own is of little relevance if the consequent ground deformations are not expected to cause damage to buildings and infrastructure. Even where a probability of failure (i.e. \( P(F<1.0) \)) has been calculated, the expected movements associated with this failure are not necessarily damaging. The main modes of permanent ground deformations due to liquefaction are lateral, either flow failure or lateral spreading, or vertical, due to either bearing capacity failure or volumetric strain-induced settlements.

Liquefaction flow failure occurs when the static shear stresses exceed the residual shear strength of a liquefied soil. Normally such failures occur on sloping ground, which was stable in its static condition but became unstable due to the reduced shear resistance of the liquefied soil. Once movement is mobilised, displacements can be very large (up to 10s of metres) and very rapid. The potential for flow failure can be assessed using standard slope stability analyses, substituting the residual undrained shear strength (Section 2.9.5) of the liquefied layer for its static properties, where a factor of safety below one indicates a flow failure hazard.

Lateral spreading is also a down-slope failure mechanism, related to cyclic mobility. The disturbing forces are a combination of the gravitational static down slope forces and the inertial loads generated by the earthquake itself. Lateral spreads can occur on gentle slopes or where there is a free face (Figure 2-18). The factor of safety against slope failure may remain greater than one, and the ground deformation is a result of the progressive movement of surface layers as a result of the oscillation of the ground.

![Figure 2-18: Lateral spreading adjacent to a river channel (a) before and (b) after earthquake (Kramer, 1996)](image)

Liquefaction is a result of the tendency for saturated granular soils to densify under earthquake shaking and the eventual manifestation of this behaviour is settlement at the ground surface as
excess pore pressures dissipate after the earthquake. Settlements are usually estimated using free-field, one-dimensional relationships for the volumetric strain induced in the soil as a function of both the relative density of the soil and the maximum shear strain induced by the earthquake (e.g. Figure 2-19). In practice, the degree of settlement is complicated by the heterogeneity of the soil, the interaction between vertical and lateral movements, and the presence of the structure, and most simplified methods for predicting volumetric strains have quite high associated uncertainties.

![Figure 2-19](chart LINK)

**Figure 2-19:** Chart for the estimation of volumetric strain as a function of the factor of safety against liquefaction (F), Equation 2-6, and the relative density of the liquefied soil layer (D_p).

*(Figure courtesy R.W. Boulanger, U.C. Davis)*

If the residual shear strength of the liquefied soil beneath is sufficiently low, then bearing capacity failure will occur causing the structure to uniformly settle or tilt, or to 'punch' through to stiffer layers below (Figure 2-20). These types of failures are generally accompanied by heaving of the ground around the foundation. The potential for a post-earthquake bearing capacity failure can be estimated using simple static bearing capacity formulae, substituting the residual undrained shear strength of the liquefied layers. Bearing capacity failure during the earthquake should consider the effects of eccentricity of loading due to the horizontal forces and the inertia in the foundation. Failure will occur if the factor of safety is less than 1.0. However, structures that are safe against bearing failure can still develop excessive settlements (total and
differential) depending on the strains (shear and volumetric) that develop in the underlying soils.

The evaluation of potential bearing failure requires site specific information of a kind that is highly unlikely to be available for earthquake loss estimations, relating to the bearing pressures beneath each building and its pre-earthquake factor of safety. Other modes of vertical movement include loss of ground due to sand boiling or the vertical component of lateral deformation of a volume of soil. All of these hazards are potentially damaging to foundations; unfortunately there are no reliable procedures for their estimation, particularly on a regional basis. Liquefaction-induced settlements are generally approximated by the use of free-field volumetric strain approaches.

![Figure 2-20: Tilted buildings with shallow foundations in Niigata, Japan, 1964 as a result of liquefaction induced bearing capacity failure.](image)

The evaluation of liquefaction-induced ground deformations, both vertical and horizontal, is far from straight-forward. There are multiple complex mechanisms which can cause damaging ground deformations. The amount of movement can be estimated using relationships developed from empirical data, soil mechanics theory or numerical modelling. Simplified relationships developed using empirical data are the easiest to employ, but users should consider the uncertainty associated with any simplification of this very complex phenomenon, which, necessarily, neglect certain features such as three-dimensional effects, localisations and redistribution of pore water pressures.

Table 2-8 shows some commonly used methodologies, most of which rely heavily on empirical data. One reason for this reliance is the difficulty in obtaining measured in situ soil properties for input to soil mechanics theory and the variability associated with such properties (Glaser, 1994). Another is that geotechnical analysis techniques do not generally allow for large strain
deformations of soils (i.e. failure). Even where full constitutive soil models in a finite element framework are used, these cannot, to the best of the author’s knowledge, go beyond the initiation of liquefaction, at which point the procedure will break down and fail to converge (e.g. Koutsourelakis et al., 2002). Youd et al. (2002) state that 'no physical theory' exists to confirm their relationships. Furthermore, numerical models have been found to be particularly sensitive to small variations in input parameters (Seed et al., 2003), an undesirable feature in such an uncertain field as loss estimation. For earthquake loss estimations, empirical processes, based upon easily obtainable parameters, or at least parameters that can be estimated using a reasonable level of judgement, are preferable to complex numerical approaches. The uncertainty associated with any method should always be considered. The empirical methods shown in Table 2-8 are generally accepted to only be accurate to within a factor of 2 or 3 (e.g. Glaser, 1994; Youd et al. 2002) and their predictive capacity tends to be worse for small-to-moderate deformations (0.3 to 0.75 m, Seed et al., 2003). All of the methods shown use different approaches and uncertain variables; none of them have overwhelming support in their favour compared to the others.

For the prediction of lateral movements, the EPOLLS (Empirical Prediction of Liquefaction Lateral Spreading) methodology (Rauch & Martin, 2000), has some advantageous features, in that it provides three different levels of empirical relationship, depending on the amount of data available. In the absence of geotechnical data, the highest-level relationship requires only the definition of magnitude, distance, peak acceleration and duration of strong shaking as input to the lateral spreading calculation. When more data are available to describe the topography and geology of the site an increased number of variables are introduced to the relationships, with two more levels presented. An additional advantage is that the formulae estimate the average rather than the maximum displacement, which is more appropriate to the requirements of a loss estimation study. There is also a study of the statistical variation of horizontal movements within a lateral spread (Rauch & Martin, 2001).

The uncertainty associated with determining the expected permanent ground deformation arises from the following areas:

- The level of earthquake hazard in terms of the variability associated with the ground-motion estimation and the uncertainties regarding earthquake occurrence.
- The likelihood of liquefaction triggering, based upon empirical data with associated scatter.
### Table 2-8: Selected methodologies for determining liquefaction-induced permanent ground deformations

<table>
<thead>
<tr>
<th>Reference</th>
<th>Description</th>
<th>Variable input parameters*</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>I. Lateral ground deformation</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Youd et al. (2002)</td>
<td>Equations obtained through multi-linear regression of empirical data</td>
<td>M, R, thickness of layer with ((N_1)<em>{50} &lt; 15) ((T</em>{15})), ((N_1)<em>{50}), FC, (D</em>{50})</td>
<td>Shown to be accurate within a factor of 2</td>
</tr>
<tr>
<td>Bardet et al. (2002)</td>
<td>Multi-linear regression of empirical data</td>
<td>M, R, (T_{15}), slope ((S)) and free face ratio ((W))</td>
<td>Same dataset as Youd et al. (2002) but in a probabilistic framework</td>
</tr>
<tr>
<td>Youd and Perkins (1987)</td>
<td>Equations for Liquefaction Severity Index (LSI), defined as maximum lateral deformation in inches, with (0 &lt; LSI &lt; 100)</td>
<td>M, R</td>
<td>Maxium value. Single topographic/geologic environment only.</td>
</tr>
<tr>
<td>Rauch and Martin (2000)</td>
<td>Empirical prediction of liquefaction lateral spread</td>
<td>i. M, R, PGA, Duration</td>
<td>Average displacements 3 equations, for different levels of input data: site, regional and geotechnical</td>
</tr>
<tr>
<td></td>
<td></td>
<td>ii. M, R, PGA, duration, (L_{slide}), (S_{top}), (H_{face}) (if free face present)</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>iii. M, R, PGA, duration, (L_{slide}), (S_{top}), (H_{face}), (Z_{FSmin}), (Z_{iq})</td>
<td></td>
</tr>
<tr>
<td><strong>II. Settlement</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Tokimatsu and Seed</td>
<td>Charts relating in situ density measurements to earthquake induced shear strain. Laboratory and empirical data</td>
<td>((N_1)_{50}), CSR, FC</td>
<td>Shown to yield correct results within a factor of 2 – 3 (Glaser, 1994)</td>
</tr>
<tr>
<td>1987</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Ishihara and Yoshimine</td>
<td>Charts relating (F_l) to in situ density and volumetric strain</td>
<td>(F_l), (N_l), FC</td>
<td></td>
</tr>
<tr>
<td>(1992)</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>III. Combined lateral and vertical movement</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shamoto et al. (1998)</td>
<td>Charts for volumetric and shear strains</td>
<td>(\tau/\sigma), (N_1), FC</td>
<td>Uses soil mechanics theory as well as empirical data</td>
</tr>
</tbody>
</table>

* See ‘Abbreviations’ at beginning of thesis for explanation of notation
The calculation methodology used to estimate permanent ground deformation, which will have uncertainties due to scatter, and, more fundamentally, due to limitations and limited accuracy.

The in situ soil parameters, such as penetration resistances, which have an uncertainty due to insufficiency of geotechnical data and thus the difficulty in accurately defining the thickness and extent of different stratigraphic layers (epistemic), due to natural heterogeneity of soils (aleatory), and due to measurement or equipment biases. Standard penetration test (SPT) measurements have been cited as having coefficients of variation from 25 to 50% (Koutsourelakis et al., 2002; Baecher & Christian, 2003).

The necessary simplification of soil properties and stratigraphy, since even in the unlikely event that comprehensive geotechnical data were available, it would not be feasible on a large scale to incorporate them without some simplification.

Of particular concern in terms of assessing building damage are the expected differential settlements or differential lateral movements. Ishihara and Yoshimine (1992) state that "because of nonhomogeneous conditions in the soil deposits, the settlements seldom occur uniformly even in small localised areas and differential settlements become the major cause of the damage to lifelines or other facilities". As noted by Fenton and Vanmarcke (1998) and Vanmarcke (1977), it is the maxima and minima within a soil profile that can control geotechnical performance, so consideration of average properties, and the average (uniform) deformations that these yield, is insufficient for damage estimation requirements. Differential ground settlements will occur due to heterogeneity in soil stiffness and stratigraphy both laterally and with depth. Differential lateral movements will also be caused by the variability of in situ ground conditions, but they will also have a less random component related to geometric variability in terms of increasing distance from a free face or from the toe of a slope. Rauch and Martin (2001) found that the variability of lateral displacements across a slope to be reasonably represented by a gamma distribution.

2.9.5 Residual undrained shear strength

The residual shear strength of a liquefied soil can be determined in the laboratory, or estimated using correlations between in situ test data and liquefied shear strength obtained through back analysis of field case histories. Field data tend to indicate much lower shear strengths than laboratory tests. As well as the difficulty in obtaining truly undisturbed samples, there are some phenomena which cannot be replicated in laboratory tests. The layered nature and contrasting
permeability of in situ soil deposits can impede the flow of water as earthquake-induced excess pore water pressures dissipate; this can lead to localised weakening of the soil at permeability interfaces (see Figure 2-14). A further in situ phenomenon that cannot be replicated by testing small samples is the potential intermixing of soil layers with different characteristics during the shear deformation, which can reduce the shear strength. Field data, however, are likely to implicitly include such phenomena, and therefore use of empirical correlations to estimate post-liquefaction shear strengths may be advantageous (e.g. Seed, 1987).

2.9.6 Mitigation

For design purposes, when the potential liquefaction risk is considered to be unacceptable for the performance requirements of an engineering project, there are three principal options available:

Relocation. Relocating a structure or facility to avoid susceptible zones may be the most straightforward option in terms of mitigating the effect of liquefaction. For regionally distributed facilities such as lifeline or transportation networks, this option is unlikely to be practical.

Prevention. The use of ground improvement to limit or prevent the occurrence of liquefaction has been shown to be effective in many past earthquakes. Ground improvement may involve increasing the density of the liquefiable soil through compaction, vibration or replacement; reinforcing and densifying the soil through jet grouting or deep soil mixing, or providing additional drainage to allow excess pore water pressures to dissipate more rapidly. Stone
columns for example (Figure 2-21), densify the in situ soil and have the secondary benefits of reinforcing the natural soil and providing drainage paths. Again, this is unlikely to be an appropriate solution for regionally distributed facilities.

*Accommodation through design.* Foundations can be designed to withstand expected ground deformations and to ensure that movements are not translated to the superstructure. Piled foundations can be designed to accommodate additional lateral loads imparted by soil movement, or to have sufficient vertical capacity even in the case of negative skin friction due to settlement. Shallow foundations can be designed to be sufficiently strong so as to behave as a rigid body when subjected to ground deformations. In the case of lifeline networks, the accommodation of the expected movements, and the implementation of appropriate response and repair measures, is frequently the only available solution.

For assessment, as opposed to design, mitigation is not directly relevant. However, if it is found that the consequences of liquefaction-induced damage are unacceptable in financial or social terms then mitigation may be considered and the cost-benefit implications considered using a loss assessment model. Consideration of mitigation is also important in regions with good seismic design practice, since it may be that appropriate mitigation will have been implemented, in which case to evaluate the full potential losses due to liquefaction would be over-conservative.

### 2.10 Landslides

Man-made or naturally occurring slopes with marginal or even moderate factors of safety against failure in static conditions may become unstable due to additional earthquake loading. Earthquake-induced landslides that cause significant losses are rare events, but can be catastrophic, as discussed in Chapter 3. Since liquefaction, rather than landslides, is the focus of this thesis, only a brief overview of landslide hazards and their assessments can be presented in this section; however, this is not intended to discount their importance.

The stability of slopes is quantified by their factor of safety, where in simple terms, the factor of safety is the ratio of the resistance to movement to the disturbing forces (gravitational and seismic). Where this ratio is less than unity, slopes are liable to movement. As with liquefaction, probabilistic approaches may also be used to determine the probability of slope failures (e.g. Murphy & Mankelow, 2004). The additional disturbing forces experienced by a slope during an earthquake are frequently sufficient to cause failure. The finite level of seismic action required to bring a slope to failure during an earthquake is described as the ‘critical’
acceleration, a parameter related to the slope properties and not to the earthquake. The principal parameters describing the static stability of a slope are its geometry, soil strength, groundwater conditions and the existence of pre-existing shear surfaces. Groundwater conditions in particular add an extra complexity to the analyses, since these will fluctuate seasonally and so the distribution of values needs to be considered. Most slope stability assessment methodologies use detailed site specific information to describe these parameters, and their application is suited to engineering analysis and design rather than to regional zonation for loss assessments. The ability to make regional assessments is very dependent upon the quality and quantity of the available data.

The term ‘landslides’ is used collectively to describe many different modes of failure including rock falls and slides, earth slides, earth slumps and debris slides (Rodriguez et al., 1999). According to the classification scheme used by Keefer (1984), landslides can be either disrupted slides, whereby there is a very high level of internal disruption of the rock or soil mass, (e.g. rock falls or avalanches), or coherent slides, whereby the soil mass moves as an intact block (e.g. deep-seated translational or rotational slides). Secondary attributes in Keefer’s classification scheme include water content, velocity of movement and depth of slide mass. Lateral spreads and flow failures, although a type of landslide, are categorised as liquefaction-related movements in this thesis. Landslides can be triggered during earthquake shaking and also hours or even days after an earthquake. Even where slopes do not fail as a result of an earthquake, destabilising changes may take place that have long-term consequences on the stability of slopes (Wasowski et al., 2000).

In the same way as for liquefaction hazard, an assessment of landslide hazard needs to consider the potential for instability to occur, its likelihood under a specified level of earthquake shaking, the consequences in terms of permanent ground deformation, and how this affects buildings and infrastructure that the slide may impinge upon.

The stability of individual slopes is evaluated using detailed geotechnical investigation data and stability calculations. These may use simplified limit equilibrium approaches or more complex numerical modelling. In situ monitoring may also provide useful data for the determination of individual slope stability. Pseudo-static analyses are those that employ static methods of slope stability analysis, with additional destabilising forces due to the earthquake. The method of Sarma (1979) uses such analyses to obtain the threshold or critical acceleration, the minimum peak ground acceleration that will cause a slope to have a factor safety of unity on its most critical slip surface. The deformation of the slope is then obtained from the ratio of the critical acceleration to the design peak ground acceleration.
More detailed analyses of slope stability during earthquakes may be carried out using finite element analyses, as described for example by Kramer (1996). Such methods have little relevance to the requirements of regional loss estimations and are not discussed in this thesis. Regional assessments of potential seismic slope instabilities are generally carried out for simplified geological, topographic and climatic conditions and slope geometry representative of the study region (Rodriguez et al., 1999). Quantitative assessment of regional landslide hazard can be very complex because of the variation in slope characteristics over an affected region. Furthermore, even where the potential for earthquake-induced landslides to occur has been identified and mapped, such maps do not provide the essential information for the purposes of loss estimations, i.e. the probability and amplitude of expected slope displacements.

The likelihood of landslides occurring is related to the strength of earthquake shaking, which in itself is an uncertain parameter. The preferred approach for the assessment of landslide hazard on a regional basis is to use a probabilistic approach, which gives a conditional probability of failure for a specified level of earthquake shaking. A further complication, noted by Wasowski et al. (2000), is that landslides can be triggered by relatively small magnitude earthquakes, and therefore a complete estimation of hazard needs to consider these smaller earthquakes with shorter recurrence intervals, which would not necessarily cause any direct damage to the built environment.

2.11 Induced hazards

The incorporation of fire following earthquake, inundation, or hazardous materials into a loss estimation, briefly described below, requires a level of detail that for all but the most sophisticated models, is too great, and the uncertainty associated with any results is very large. For the purposes of most loss estimations, the identification of possible high risk areas or high risk facilities, with no subsequent quantification of expected losses, is all that can reasonably be incorporated.

2.11.1 Fire following earthquake

Fire following earthquake can be triggered for a number of reasons, ranging from household cooking facilities to chemical explosions. Some of the many variables that affect the control of spread of fire are:

- Pre-earthquake building controls: building separation, use of fire retardant materials.
• The weather conditions at the time of the earthquake: wind speed and direction and rainfall.

• The structural performance of fire stations in the earthquake; serious damage will impair the fire fighters ability to respond to fires.

• The levels of damage and disruption incurred by lifelines: principally the water distribution networks and the transportation networks. A failure of either of these seriously restricts fire-fighting capabilities.

• The scale of damage: if many buildings are collapsed and many lives are at risk, emergency response will be dedicated to search and rescue in the worst affected areas, and to medical treatment, and there may be a shortage of manpower to fight the spread of fire.

Fires following earthquakes can cause significant losses, the most publicised examples of this being the 1906 San Francisco earthquake and the 1923 Great Kanto earthquake in Japan. The accurate modelling of probable spread of fire following an earthquake, and the quantification of damage and loss resulting from it is extremely complex; models are dependent for example on evaluating the expected speed and direction of the wind at the time of the fire.

2.11.2 Inundation

Inundation, other than that caused by tsunamis (Section 2.3), is a hazard predominantly associated with the catastrophic failure of large dams located within range of inhabited areas. Maps showing zones of different intensities of flooding in the event of a dam failure is a site-specific design issue associated with the risk mitigation for the dam itself, and again, is complex to incorporate into a regional loss estimation. Where such studies have been carried out for existing dams, it may be possible to incorporate the results into a loss model.

2.11.3 Release of hazardous materials

The final collateral hazard considered in this section is release of hazardous materials. As with fire, there are many variables associated with the identification of this hazard and its potential impacts upon the environment or society. The seriousness of an event could range from localised chemical spills to the release of large amounts of toxic materials into a drinking water supply. There appear to be relatively few case histories of such events occurring after earthquakes; none were identified within the review of 50 earthquakes presented in Chapter 3.
As with inundation, any detailed studies of the risk of release of hazardous materials are most likely to be carried out as part of a site-specific risk assessment.
3 EARTHQUAKE LOSSES DUE TO GROUND FAILURE

This chapter presents a review of the principal features of earthquake losses incurred in damaging earthquakes over the last 15 years. Survey data are impartially analysed, considering both ground failure and ground shaking as sources of damage, and their relative contribution to the overall damage in each section of the regional infrastructure is presented. There are many other variables influencing these contributions, including the size of the earthquake, the economic status of the affected region, local geology and terrain and the building stock, which have been considered.

The objective of this review is straightforward: to aid decision making in terms of which components of a loss estimation model should merit the most time and resource allocation. The general assumption that ground shaking is the principal cause of damage and loss, although intuitively reasonable, was considered worthy of further investigation. The findings of this chapter will help to contextualise the remaining issues covered in this thesis and to identify situations where they are of the greatest importance.

Field reconnaissance of the damage caused by earthquakes has been standard practice for many years, and detailed observations and records of the damage that can be caused by ground failure have been made as far back as the two major damaging earthquakes in 1964, in Alaska and Niigata, Japan, which first highlighted the damaging potential of ground failure. During the 40 years that have passed since these earthquakes, understanding of both the phenomenon of liquefaction and the triggering of landslides has progressed significantly and is still developing.

Hays (1998) comments on some of the principal lessons learned from the “scientific laboratories” of past damaging earthquakes, one of which is “failing to identify soils and slopes susceptible to permanent ground displacement”. Time and again it has been seen that the occurrence of liquefaction in loose cohesionless soils, and the occurrence of instabilities in slopes with marginal static safety factors, were easily predictable from analyses of case history data. The identification of vulnerable sites through microzonation, and improved land use planning around these zones, would significantly reduce the damage and disruption associated with the phenomenon of ground failure. However, as shown in this chapter, such lessons have not yet been put into practice. For earthquake loss estimations it is the current state of practice that is relevant, which, with relation to earthquake-induced ground failure is that in certain environments, ground failure continues to be an important – and predictable – hazard.
The built environment is at risk from several hazards directly caused by earthquakes (see Figure 1-1). Earthquake losses can result from each of these, and in many earthquakes the total losses are a result of most, if not all, of these hazards. In this study strong ground-motion and amplified ground shaking due to topographic or soil effects are classed as ground shaking. The term ground failure is used to refer to landslides, liquefaction and surface fault rupture, the physical link between these features being that they are all manifested by a permanent ground deformation. Although the focus of this thesis is liquefaction-induced ground failure, in this chapter all forms of ground failure are considered, as are tsunamis, in order to provide a comparison of the relative impact of all the hazards shown in Figure 1-1. Induced hazards such as floods, fire or the release of hazardous materials, which are potentially catastrophic but rare, are not covered.

There are a number of published databases of ground failure hazards, notable examples being the work of Keefer (1984) and Rodriguez et al. (1999) who compiled databases of earthquake-induced landslides. Similarly valuable inventories of liquefaction field case histories have been compiled (e.g. Youd & Hoose, 1978).

For the purposes of this thesis, it is the risk rather than the hazard that is relevant; a landslide in an uninhabited mountainous region, whilst being an important earthquake related hazard, is unlikely to lead to any significant damage to the built environment and therefore the associated losses for such an event will be negligible or even non-existent. Therefore an inventory of risk posed by each of the direct hazards is presented in this chapter, where risk is defined by the equation:

\[
\text{Seismic Risk} = \text{Seismic Hazard} \times \text{Vulnerability} \times \text{Exposure} \times \text{Specific Cost} \quad (3-1)
\]

The definition of direct and indirect losses was briefly presented in Chapter 1. Economic loss data in post-earthquake reports is of variable quality, but it rarely includes any breakdown of indirect economic losses; apart from anything else, it can take many years for the full indirect losses to be realised. Quantitative illustrations of the scale of indirect earthquake losses include the Port of Kobe, whose closure following the January 1995 earthquake impacted the city for many years. Even 4 years after the earthquake, the Port of Kobe was only at 80% of its previous productivity, reflecting the permanent loss of business caused by competition from other ports in the region (Audigier et al., 2000).
3.1 Selection of earthquakes

Only earthquakes occurring since 1989 have been included in this review in order to maintain some consistency in the data. Information of particular interest includes statistics of earthquake damage, losses and disruption, regional factors and spatial trends. The objectives of earthquake field reconnaissance have changed somewhat over the years, and it was found that since 1989 there has been a reasonably consistent focus on the same features. In earlier years investigators concentrated on the fundamental lessons to be learnt regarding the performance of the built environment under strong shaking, and information such as the economic losses and the building damage distribution were of less importance. Since there are so many other variables influencing earthquake damage in a region, reducing the uncertainties related to differences in reporting seems prudent. A further advantage of using a limited time period is that the variability of the affected building stock is reduced, although there are still obvious global variations in building practice. There is also a trend for investigating smaller magnitude, less damaging earthquakes: by extending the database much further back, there would potentially be a bias towards the larger catastrophic earthquakes in terms of the published reports. Starting the inventory in 1989 also allowed the inclusion of the Loma Prieta earthquake, for which extensive reports are available on both ground shaking and ground failure effects. The selection does not mean that there are not many valuable lessons to be learned from earthquakes before 1989. In particular, the 1964 earthquakes in Niigata and Alaska provided much of the empirical data on the occurrence of landslides and liquefaction that is still used in assessment methodologies today, including some of those presented in Table 2-8.

The earthquakes reviewed are listed in Table 3-1. Other than the restriction on the time-frame considered, all damaging earthquakes for which field reconnaissance has been reported and published are included, irrespective of their size, location or damage levels, since all of these variables can be informative and relevant. Inevitably, some damaging earthquakes during this period are not included, generally because no engineering damage reports for these events were obtained. For example, according to contemporary newspaper reports, two earthquakes in Afghanistan on March 3 and March 25 2002 left over 1000 dead and many more injured and homeless. Due to the political situation in the region at the time of the events, reconnaissance of the earthquake effects from a scientific perspective would neither have been feasible nor appropriate. Notwithstanding such omissions from the database, it is considered that the number and variety of the earthquakes included is sufficient to provide some very useful illustrations.
<table>
<thead>
<tr>
<th>Name</th>
<th>Date</th>
<th>Mw</th>
<th>Focal Depth (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loma Prieta, USA</td>
<td>17/10/1989</td>
<td>6.9</td>
<td>20</td>
</tr>
<tr>
<td>Newcastle, Australia</td>
<td>20/12/1989</td>
<td>6.0</td>
<td>10</td>
</tr>
<tr>
<td>Manjil, Iran</td>
<td>20/06/1990</td>
<td>7.7</td>
<td>20-30</td>
</tr>
<tr>
<td>Luzon, Philippines</td>
<td>16/07/1990</td>
<td>7.1</td>
<td>30</td>
</tr>
<tr>
<td>Augusta, Sicily</td>
<td>13/12/1990</td>
<td>5.8</td>
<td>10</td>
</tr>
<tr>
<td>Limón, Costa Rica</td>
<td>22/04/1991</td>
<td>7.4</td>
<td>20</td>
</tr>
<tr>
<td>Georgia</td>
<td>29/04/1991</td>
<td>7.1</td>
<td>20</td>
</tr>
<tr>
<td>Erzincan, Turkey</td>
<td>13/03/1992</td>
<td>6.7</td>
<td>30</td>
</tr>
<tr>
<td>Landers, USA</td>
<td>28/06/1992</td>
<td>7.3</td>
<td>10</td>
</tr>
<tr>
<td>Big Bear, USA</td>
<td>28/06/1992</td>
<td>6.4</td>
<td>10</td>
</tr>
<tr>
<td>Cairo, Egypt</td>
<td>12/10/1992</td>
<td>5.7</td>
<td>30</td>
</tr>
<tr>
<td>Flores Island, Indonesia</td>
<td>12/12/1992</td>
<td>7.9</td>
<td>40</td>
</tr>
<tr>
<td>Hokkaido-Nansei-Oki</td>
<td>12/07/1993</td>
<td>7.7</td>
<td>15-35</td>
</tr>
<tr>
<td>Guam</td>
<td>08/08/1993</td>
<td>7.5</td>
<td>60</td>
</tr>
<tr>
<td>Khatari, India</td>
<td>30/09/1993</td>
<td>6.2</td>
<td>10</td>
</tr>
<tr>
<td>Northridge, US</td>
<td>17/01/1994</td>
<td>6.7</td>
<td>20</td>
</tr>
<tr>
<td>Paez, Colombia</td>
<td>00/06/1994</td>
<td>6.7</td>
<td>10</td>
</tr>
<tr>
<td>Kobe, Japan</td>
<td>17/01/1995</td>
<td>6.9</td>
<td>20</td>
</tr>
<tr>
<td>Kozani-Grevena, Greece</td>
<td>13/05/1995</td>
<td>6.4</td>
<td>10</td>
</tr>
<tr>
<td>Neftegorsk, Sakhalin</td>
<td>28/05/1995</td>
<td>7.1</td>
<td>10</td>
</tr>
<tr>
<td>Aeglon, Greece</td>
<td>15/06/1995</td>
<td>6.4</td>
<td>10</td>
</tr>
</tbody>
</table>

**Table 3-1: Summary of earthquake database**

<table>
<thead>
<tr>
<th>Source Type</th>
<th>Affected Region</th>
<th>Vulnerability</th>
<th>Impact</th>
<th>Total losses USD</th>
<th>Fatalities</th>
<th>Reporting</th>
</tr>
</thead>
<tbody>
<tr>
<td>SCR^a</td>
<td>C, M, U</td>
<td>Low</td>
<td>High</td>
<td>10 bn</td>
<td>63</td>
<td>4</td>
</tr>
</tbody>
</table>

<p>| SCR^a       | P               | Moderate      | Low    | 1.1 bn           | 12         | 1         |
| Di          | M, U            | High          | High   | 7.2 bn           | 50 000     | 1         |
| C, M, DI, P | High            | High          | 1.5 bn | 1700             | 3          |
| C, M, Vo    | Moderate        | Low           | -      | 19               | 2          |
| C, M        | Moderate        | Moderate      | 500 mn | 53               | 3          |
| M, Ru       | Moderate        | Moderate      | 1.7 bn | 114              | 1          |
| Va          | Moderate        | High          | 400 mn | 541              | 3          |
| M, Ds, R    | Low             | Low           | 100 mn | 1                | 2          |
| M, Ds, R    | Low             | Low           | 100 mn | 2                | 2          |
| Dl, Ds, U   | High            | High          | 2 bn   | &gt; 500            | 2          |
| C, Vo       | High            | High          | 300 mn | 2200             | 1          |
| C, M, P, Va | Low             | Moderate      | 1 bn   | 231              | 3          |
| C, Pt       | Moderate        | Moderate      | 200 mn | 0                | 2          |
| P?, Ru      | High            | High          | 333 mn | &gt; 11 000         | 2          |
| C, M, Va, U | Low             | Low           | 20 bn  | 57               | 4          |
| J, Ru       | High            | Moderate      | -      | 1100             | 1          |
| C, U        | Low             | High          | 147 bn | 5500             | 4          |
| M           | Moderate        | Low           | 450 mn | 0                | 1          |
| C           | Moderate        | High          | 400 mn | 2000             | 1          |
| C           | Moderate        | Moderate      | 600 mn | 26               | 2          |</p>
<table>
<thead>
<tr>
<th>Name</th>
<th>Date</th>
<th>Mw</th>
<th>Focal Depth (km)</th>
<th>Source Type</th>
<th>Affected region(^h)</th>
<th>Vulnerability(^b)</th>
<th>Impact(^d)</th>
<th>Total losses USD(^a)</th>
<th>Fatalities</th>
<th>Reporting(^f)</th>
</tr>
</thead>
</table>
### Table 3-1: continued

<table>
<thead>
<tr>
<th>Name</th>
<th>Date</th>
<th>Mw</th>
<th>Focal Depth (km)</th>
<th>Source Type</th>
<th>Affected region</th>
<th>Vulnerability</th>
<th>Impact</th>
<th>Total losses USD</th>
<th>Fatalities</th>
<th>Reporting</th>
</tr>
</thead>
<tbody>
<tr>
<td>El Salvador</td>
<td>13/02/2001</td>
<td>6.6</td>
<td>15</td>
<td>Active crustal</td>
<td>C, M, Vo</td>
<td>High</td>
<td>Moderate</td>
<td></td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>Nisqually, USA</td>
<td>28/02/2001</td>
<td>6.8</td>
<td>50</td>
<td>Subduction, IS</td>
<td>C</td>
<td>Low</td>
<td>Low</td>
<td>1.5 bn</td>
<td>1</td>
<td>2</td>
</tr>
<tr>
<td>Atico, Peru</td>
<td>23/06/2001</td>
<td>8.4</td>
<td>30</td>
<td>Subduction, IP</td>
<td>C, M, Va, Pt</td>
<td>High</td>
<td>Moderate</td>
<td>300 mn</td>
<td>77</td>
<td>2</td>
</tr>
<tr>
<td>Molise, Italy</td>
<td>31/10/2002</td>
<td>5.9</td>
<td>20</td>
<td>Active crustal</td>
<td>M</td>
<td>Moderate</td>
<td>Low</td>
<td>300 mn</td>
<td>30</td>
<td>1</td>
</tr>
<tr>
<td>Denali, Alaska</td>
<td>03/11/2002</td>
<td>7.9</td>
<td>5</td>
<td>Active crustal</td>
<td>M, P, Va, G, R</td>
<td>Low</td>
<td>Low</td>
<td>200 mn</td>
<td>0</td>
<td>2</td>
</tr>
<tr>
<td>Colima, Mexico</td>
<td>21/01/2003</td>
<td>7.5</td>
<td>10</td>
<td>Subduction, IP</td>
<td>C, M</td>
<td>Moderate</td>
<td>Moderate</td>
<td>21</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Bingöl, Turkey</td>
<td>01/05/2003</td>
<td>6.4</td>
<td>10</td>
<td>Active crustal</td>
<td>P</td>
<td>Moderate</td>
<td>Moderate</td>
<td>178</td>
<td>1</td>
<td></td>
</tr>
<tr>
<td>Boumerdes, Algeria</td>
<td>21/05/2003</td>
<td>6.8</td>
<td>10</td>
<td>Active crustal</td>
<td>C, P, D, U</td>
<td>High</td>
<td>High</td>
<td>5 bn</td>
<td>2300</td>
<td>2</td>
</tr>
</tbody>
</table>

a  Focal depths rounded to nearest 10km.
b  C=coastal, M=mountainous, Vo=volcanic, D=deltaic, P=plains, Va=Valley, Ds=Desert, B=Basin, Je=Jungle, G=Glacier, Pt=Plateau, H=Hills, U=Urban, Ru=Rural, R=Remote

c  Qualitative rating, determined by degree of preparedness and ability to recover at the time of the earthquake.
d  high impact signifies extensive damage and disruption and that the region took some time to recover from the earthquake, according to reports.
e  In current US Dollar values, i.e. the cost at the time of the earthquake.
f  Qualitative rating, based upon the extent of published damage reports available, where the greater the extent of reporting, the lower the uncertainty related to the earthquake damage and statistics: 1 = lowest, only 1 or 2 reports obtained; 2 = moderate, at least one very detailed report and others exist for verification; 3 = well reported, at least one full volume published; 4 = exceptional reporting.
g  Stable Continental Region, as defined by Johnston et al., 1994

h  IP = Interplate, IS = Intraslab

i  losses and fatalities are reported for the January and February 2001 El Salvador earthquakes combined.
Several earthquakes were excluded from the review, since although there was a published field reconnaissance report, there was no notable damage observed. An example of this is the $M_w$ 5.0 Napa Valley, California earthquake in September 2000, where damage was mainly non-structural and disruption was minimal. The reported losses were still $15$ million, but, as already noted, due to the lack of detailed economic data in many cases, the focus of this study is on physical damage and therefore this earthquake is not included.

3.1.1 Recent events

The most recent earthquake to be included in the database was the Boumerdes, Algeria event in May 2003. The main body of this study was completed in October 2003. The damaging earthquakes in Morocco in January 2004, Bam (Iran) in December 2003, California in December 2003 and Japan in September 2003 have not been included in the main database, although it is worth noting that there were no obvious features of these earthquakes to contradict the findings of this chapter.

3.1.2 Earthquake variability

Variability among the earthquakes studied, apart from the obvious distributions of locations and magnitudes, includes the quality and quantity of reporting and damage data, the level of damage caused and the vulnerability of the affected region, where, as would be expected, the impact of earthquakes is typically much greater in highly vulnerable regions (see Figure 3-1).

Table 3-2: Comparison of economic and social losses as indicators of earthquake impact

<table>
<thead>
<tr>
<th>Top 10 earthquakes by total losses</th>
<th>Top 10 earthquakes by fatalities</th>
</tr>
</thead>
<tbody>
<tr>
<td>Earthquake</td>
<td>Total losses, $US bn 2002$</td>
</tr>
<tr>
<td>Kobe, Japan, 1995</td>
<td>120</td>
</tr>
<tr>
<td>Kocaeli, Turkey, 1999</td>
<td>27</td>
</tr>
<tr>
<td>Northridge, USA, 1994</td>
<td>24</td>
</tr>
<tr>
<td>Loma Prieta, USA, 1989</td>
<td>14.5</td>
</tr>
<tr>
<td>Chi-Chi, Taiwan, 1999</td>
<td>10.5</td>
</tr>
<tr>
<td>Cairo, Egypt, 1992</td>
<td>5.7</td>
</tr>
<tr>
<td>Umbria Marche, Italy, 1997</td>
<td>5</td>
</tr>
<tr>
<td>Bhuj, India, 2001</td>
<td>4.7</td>
</tr>
<tr>
<td>Athens, Greece, 1999</td>
<td>3.5</td>
</tr>
</tbody>
</table>

$^1$ Losses have been normalised to 2002 prices according to the change in Consumer Price Indices for the affected country and the change in US dollar exchange rates.
Figure 3-1: Variability within the earthquake database (a) Vulnerability (subjective rating based upon the degree of preparedness and the ability of the affected region to recover from an earthquake) vs. impact (the degree of damage and disruption and time taken to recover). (b) Distribution of earthquakes by location. (c) Distribution of earthquakes by magnitude.
Indicative ratings of the overall impact and vulnerability are included in Table 3-1, where these are both determined from the descriptions in the damage reports, and have been rated using a three point scale of low, moderate and high. Even where comprehensive economic data are available, these cannot be considered in isolation as an indicator of the impact of the earthquake, due to the significant differences between the economic status of the affected regions (Table 3-2).

3.2 Quality and variability of the available damage data

The quantity and quality of the available damage data for the earthquakes included in Table 3-1 varied considerably. The Northridge earthquake in 1994 seems to be the most widely studied earthquake of all time, with the number of published damage investigations, post earthquake analyses and other reports reaching the 1000s. Excellent resources of field reconnaissance include the Earthquake Engineering Research Institute’s (EERI) Learning from Earthquakes reports (http://www.eeri.org/lfel the Japan Society of Civil Engineers, the New Zealand National Society for Earthquake Engineering, and the Earthquake Engineering Field Investigation Team (EEFIT) from the UK, plus a large number of private and government institutions from around the world.

Even for the least investigated earthquakes, available information included:

- The magnitude, approximate depth and location of the event.
- Principal damage statistics in terms of the number of deaths and injuries, and the number or extent of damaged buildings.
- Discussion of causes of observed damage, such as design or construction flaws.
- Some discussion of geological features, and, if present, of the observed ground failures such as fault rupture, liquefaction and landslides.

In many, but not all, of the reports, additional relevant information included:

- Maps showing the spatial distribution of damage.
- Building-by-building damage surveys of small zones.
- Damage distribution information using damage scales such as the European Macroseismic Scale (EMS-98, Grünthal, 1998), which classifies damage from D0 (none) to D5 (complete collapse).
Economic loss data, in some cases broken down according to category and type of loss.

Of the 50 earthquakes in Table 3-1, an unprecedented level and quality of damage information was available for Loma Prieta (1989), Northridge (1994), Kobe (1995), Kocaeli (1999) and Chi-Chi (1999). The connection between these events is their significant impact (all feature at least once in Table 3-2) on highly urbanised and developed regions. Other earthquakes which were well reported include Costa Rica (1991), Erzincan, Turkey (1992), Hokkaido-Nansei-Oki (1993), Luzon, Philippines (1990), Colombia (1999), Bhuj, India (2001), Nisqually, Washington Sate (2001) and El Salvador (2001). There is less information available for the more recent earthquakes, particularly those which occurred in 2003, since the analysis and dissemination of information takes time. The quality and quantity of the damage reports for all 50 earthquakes are indicated in the final column of Table 3-1.

For a number of the earthquakes there are no economic data available and for many others only high level information on the direct costs has been published. There are often discrepancies or uncertainties in the reported costs. There are insufficient economic data or indeed any quantitative data to rate the relative damage caused by the various hazards, therefore a much more subjective scheme has been employed. The damage and disruption to each separate inventory category have been rated using a three point scale of None (1), Moderate (2) and High (3), similar to that used to rate the earthquake impact and the vulnerability of the region. Due to the variability of the data, which was sometimes very sparse, using a scale with any additional refinement would be unjustified and even misleading. Moderate damage was typically assigned when damage to the inventory category was mentioned in reports but was not widespread or very disruptive, whereas High damage was assigned when the damage to that category was widespread, or disruptive, or both. For example, approach settlement due to liquefaction of bridge embankments would register as Moderate damage, since the bridges could remain open with minimal repair, whereas collapse of one or more bridges due to liquefaction-induced ground displacement would be classified as High damage due to the cost and disruption of bridge closures.

Table 3-3 shows the distribution of collateral hazards reported for the 50 earthquakes, and also indicates which of these hazards was reported to cause damage to buildings, transportation lifelines and utility lifelines. The degree of damage is discussed later in this chapter.
3.2.1 Damage scales

The focus of this study is to understand the relative impact of ground failure on earthquake damage and losses. For this, invaluable field data includes the mapping of ground failure occurrence alongside the mapping of structural and lifeline damage. Qualitative mapping of ground failure occurrence and its influence on infrastructure has rarely been comprehensively gathered in earthquake reconnaissance studies. The Ground Failure Index (GFI) used by Bray and Stewart (2000) to present damage survey data for Adapazari, following the 1999 Kocaeli earthquake (see Chapter 5) allows a semi-quantitative comparison of different levels of building foundation failure. The consistent use of such a scale for ground failure in future earthquake field surveys, in the way that many European investigators have employed the 6 point EMS-98 damage scale (Grunthal, 1998) to define structural impact, would greatly improve the possibilities for comparing ground failure on a regional or inter-earthquake basis.

Loss estimation methodologies generally determine direct losses in terms of the damage ratio, \( D_r \), and a building which has suffered complete collapse will have \( D_r = 1.0 \). Damage ratios can usually be related to structural damage indices or descriptions, such as the EMS-98 scale, or the damage state descriptions presented in HAZUS (FEMA, 2003), which define damage in terms of cracking, damage or collapse of structural and non-structural elements. Such scales do not include descriptions of damage states brought about by permanent ground deformations such as those described in Section 3.4.2.

3.2.2 Ambiguity in reported data

Where there are no official government damage statistics, there is invariably some ambiguity in the damage reported by various individuals or teams. For example, the number of collapsed buildings after the 1999 Chi-Chi earthquake is variously reported as over 17,000 (EDM, 2000) and over 100,000 (Shin & Teng, 2001) – in cases such as these, some judgement had to be exercised regarding the perceived reliability of the source. By implication, there will be a similar variability related to those earthquakes for which only a single report was available, but it is not considered that such issues would have a major impact on the mainly qualitative trends reported herein.

3.3 Damage due to earthquake-induced landslides

Including landslides in remote areas, which did not lead to any damage, the total number of earthquakes where landslides were reported was over 50%. The different types of damage caused by landslides are discussed in more detail in the following sections.
3.3.1 Catastrophic landslides

Large landslides can potentially claim massive loss of life and leave an indiscriminate trail of devastation. Fatal landslides occurred in 10 of the earthquakes reviewed and in five of these, hundreds of people lost their lives; frequently entire villages were destroyed. These catastrophic landslides occurred in the earthquakes in Ardebil, Iran in 1997; El Salvador in January 2001; Luzon, Philippines in 1990; Manjil, Iran in 1990 and Paez, Colombia in 1994. The only easily identifiable feature linking these events is that all occurred in mountainous areas of high landslide susceptibility. In Iran, entire villages were reported to be destroyed; in the mountainous areas around Baguio in the Philippines, an unknown number of villagers lost their lives due to landslides; in the remote region of Paez in Colombia over 1000 people lost their lives and settlements were swept away by huge debris flows (Forero-Duenas, 1996); and the Las Colinas development in El Salvador, where more than 400 residents were killed, was located at the base of a tall and unstable slope (Bommer et al., 2002b). The other five fatal landslides caused tens rather than hundreds of deaths, usually involving the collapse of one building, such as the destruction of the Yo Yo Hotel in the Hokkaido-Nansei-Oki earthquake in 1994 (Harp & Youd, 1995), or the Degirmendere Hotel in the Kocaeli earthquake (Bardet & Seed, 2000).

Casualties due to landslides are typically caused by a building or a large number of buildings being completely destroyed by down-slope soil movement. Very few buildings are constructed to withstand this magnitude of permanent ground deformation. In the absence of other constraints, appropriate land use planning would be able to prevent new structures being sited on or close to potentially unstable slopes. Such catastrophes tend to affect hillside villages in rural or remote areas, where both the location and the type of building create highly vulnerable zones. Particularly vulnerable zones also exist on the periphery of major cities in developing countries, where there is a huge demand for land, little control, and only susceptible terrain is available. These areas are increasingly likely to face catastrophic future losses.

In the January 2001 El Salvador earthquake landslides were actually the primary cause of the earthquake losses in the entire region, over and above ground-shaking induced damage. This case is rare, but not unique, for example, in 1987 an earthquake in Ecuador triggered massive debris flows which were the cause of nearly all the earthquake induced damage. Over 40km of the trans-Ecuadorean crude-oil pipeline was severely damaged by these flows, causing US$800million of lost revenue during the five month repair period (Schuster, 1991). A combination of factors could lead to such cases where landslides are the most critical hazard: landslides can occur where ground shaking is relatively weak but slopes or cuttings are unstable, or heavy rainfall has occurred to destabilise slopes. Steep unstable areas are likely to be
sparsely populated in most parts of the world, whereas lifelines such as roads, railways and pipelines are often obliged to traverse such features.

Table 3-3: Occurrence of collateral hazards, and damage to the main inventory categories as a result of each hazard as indicated by damage reports

<table>
<thead>
<tr>
<th>No.</th>
<th>Name</th>
<th>Collateral Hazards</th>
<th>Buildings</th>
<th>Transportation</th>
<th>Utilities</th>
</tr>
</thead>
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FR = Fault Rupture; T = Tsunami; GS = Ground shaking; SF = Slope Failure; L = Liquefaction. Empty cell = hazard not reported, or reported but no mention of damage; o = minor or moderate damage reported, relative to overall damage; • = major damage reported relative to overall damage.
A further example of landslide-induced damage to buildings was in the Santa Cruz mountains following the Loma Prieta earthquake in 1989, where between 500 and 800 homes were destroyed due to slope failures in an area of known instability (Seed et al., 1991), fortunately with no loss of life.

Generally, when destructive landslides such as those described in the preceding section occur, they become the primary cause of damage in the affected areas, overshadowing any previous damage to buildings on or near the slope that may have been caused by ground shaking. Only one case was reported within the database where buildings were damaged by both ground shaking and slope movement: this was to homes located at the top of the Pacific Palisades landslide (Figure 3-2) after the Northridge earthquake (Norton et al., 1994). In all other cases, buildings in the path of destructive landslides rarely survived, therefore the level of damage caused by strong ground shaking prior to the landslide is practically irrelevant. This statement refers only to building damage; loss of life and injuries may well be caused by the ground shaking rather than the landslides, which may occur some time (from seconds to days) after the earthquake, allowing time for evacuation.

Figure 3-2: Slope failure and building damage at Pacific Palisades, Santa Monica following the Northridge Earthquake, 1994. (Courtesy EEFIT)
In this review, only one case of engineered structures or large industrial facilities being damaged by landslides was reported, this being an electrical substation damaged in the Chi-Chi earthquake. In general, major engineered structures are unlikely to be positioned in the vicinity of unstable slopes.

3.3.3 Landslide-induced transportation disruption

Although fatal landslides and the accompanying destruction of buildings are rare but catastrophic events, disruptive failures of road, rail or river embankments are far more common (e.g. Figure 3-3), occurring in 46% of the earthquakes reviewed, with five cases of major routes being blocked by landslides. All four roads leading into the mountain city of Baguio were blocked by landslides after the 1990 Luzon earthquake, causing significant delays to the rescue operations. Transportation disruption was the greatest effect of the Luzon earthquake, and this was caused in large part by the landslides in mountainous areas. A landslide blocking the busy Santa Monica freeway following the Northridge earthquake caused disruption and delay, leading to indirect losses to businesses in the area, as well as the direct costs to repair the road and stabilise the slope. However, it is expected that these losses would be a tiny fraction of the total losses of US$20bn incurred in the earthquake. Outside San Salvador, following the January 2001 earthquake, the Pan American highway was blocked by massive landslides in both the east and west direction (Figure 3-4), remaining partially closed for at least 10 months due to the need for extensive stabilisation works (Bommer et al., 2002b).

Figure 3-3: Rockslide blocking the Chivay–Arequipa highway in Peru, two days after the 23 June 2001 earthquake. (Photograph: Richard Bird)
Chapter 3  Earthquake Losses due to Ground Failure

Figure 3-4: Major landslide at Las Leonas blocking the Pan American Highway. (Photograph: Julian Bommer, Courtesy EERI)

The direct cost of repairing damage to roads following the El Salvador earthquakes was estimated at US$2.7 millions, less than 1% of the total earthquake losses. The long-term disruption would have caused additional indirect losses, but the magnitude of such losses has not been quantified. Some degree of transportation disruption ensued as a results of rock falls or slope failures onto roads in 46% of the earthquakes, and in 28% transportation disruption was the only effect of the landslides. In all these 28% of cases, the failures were in steep cut slopes or rock cuttings, probably due to the fact that either slope stabilisation or greater land-take were not feasible due to financial constraints.

3.3.4 Landslide-induced damage to lifelines

Landslides did not tend to cause significant damage to utility lifelines in the earthquakes reviewed, with a few notable exceptions. Landslide-induced damage to an electricity substation in Taiwan during the 1999 Chi-Chi earthquake contributed to the major power disruptions after the earthquake (Sitar & Bardet, 2001). These power cuts had far-reaching effects, since many of the high-tech manufacturing facilities, producing semi-conductors for the global electronics market, were disrupted, causing a significant drop in technology stock prices on international stock exchanges. Landslides had a small impact on the drinking water supply in southern Costa Rica after the 1991 earthquake due to increased turbidity of the river water used to supply water treatment plants. In two further earthquakes – El Salvador, 2001 and Northridge, 1994 - damage to electricity transmission towers located on failed slopes was reported.
There is no consistent trend amongst these few examples, and the only case where landslides were the cause of significant lifeline losses was in an indirect manner in the Chi-Chi earthquake. The lost business due to the power interruption was responsible for a significant portion of the US$600 million insured losses (Sitar & Bardet, 2001). However, these losses were more directly related to the lack of sufficient emergency power back up at the manufacturing plants.

From these 50 earthquakes, there is little evidence of the damaging potential that landslides pose to lifelines, although this conclusion cannot be extended to imply that landslide-induced damage is never a major component of earthquake impact. A very important example of landslide-induced lifeline damage was the 1987 Ecuadorian earthquakes. As mentioned in Section 3.3.1, over 40 km of the Trans-Ecuadorian crude-oil pipeline was destroyed due to massive debris flows and avalanches, related to the anomalously high rainfall in the period leading up to the earthquakes. This pipeline provided Ecuador with almost 60% of its export revenue, and loss of revenue during the five month repair period was US$800 million, 80% of the total earthquake losses (Schuster, 1991).

3.4 Damage due to Earthquake-Induced Liquefaction

3.4.1 Occurrence of liquefaction in earthquakes

Liquefaction was reported in 62% of the earthquakes reviewed, causing some degree of damage in all but three cases. Of those earthquakes where no liquefaction occurred, two were in Colombia (1994 and 1999) where it has been noted that the volcanic soils generally have too high a clay content for liquefaction to occur (Restrepo & Cowan, 2000). There was no liquefaction reported in the moderate magnitude events in Italy (1991, 1997, 2002) or in other $M_w \leq 6$ events in Newcastle (Australia), Ecuador, Iran and Turkey. Liquefaction did occur in other $M_w \leq 6$ events in Turkey, Greece and Egypt, causing only minor damage. Where liquefaction did occur, there were no great surprises; back analyses tended to show that liquefaction could have been predicted using state-of-the-practice methodologies (Section 2.9). Even in Kobe, where the liquefaction of the gravelly Masado material at Kobe Port in 1995 is frequently cited as being an unexpected occurrence, it was actually the strength of the ground shaking that was the surprise, being much greater than had been used in design checks for this material (Park et al., 1995).

Direct evidence of the occurrence of liquefaction comprises sand boils, lateral spreading, settlement or slumping; where none of these were observed, liquefaction beneath a building can frequently be inferred from the failure modes. Unprecedented levels of information regarding
the impact of liquefaction on densely populated urban environments are available from the 1995 Kobe and 1999 Kocaeli earthquakes.

Liquefaction frequently occurs in reclaimed soils in coastal areas (e.g. Kobe Port; Marine District of San Francisco; Manzanillo, Mexico) and is also a common occurrence in alluvial or deltaic deposits including old or existing river beds (e.g. Dagupan, Luzon; Ceyhan, Turkey; the Rann of Kachchh in Gujarat, India). Table 3-3 indicates which earthquakes had reported occurrences of liquefaction.

3.4.2 Liquefaction-induced building damage

Compared to ground-shaking, landslide and tsunami hazards, liquefaction is less likely to cause conventional collapse of buildings or fatalities. In fact, structural damage to buildings in the conventional terms of cracking, failure of structural members and collapse or partial collapse, such as those defined in the EMS-98 damage scale (Appendix B) is rarely reported for buildings affected by liquefaction. Liquefaction-induced damage to buildings typically includes foundation settlement or tilting, or displacement due to lateral spreading, although structures with poor foundations can collapse due to liquefaction-induced foundation movements. Loss of life and contents damage will be significantly less in the case of liquefaction-induced settlement and tilting than in shaking-induced collapse. Existing damage scales poorly represent such modes of building damage or failure.

Figure 3-5: Bulging of ground adjacent to settled building, Adapazari, Turkey, 1999. Courtesy Earthquake Engineering Field Investigation Team (EEFIT), UK
Foundation settlement occurs either where the soil beneath the building has settled due to volume change or where the strength of soil has decreased causing the structure to sink into the ground due to loss of bearing capacity. Bray and Stewart (2000) noted both types of settlement in their survey of damaged buildings in Adapazari after the 1999 Kocaeli earthquake; the former was the more common, the latter (bearing failure) being evidenced by bulging of the ground surface around the settled buildings (Figures 3-5 and 3-6). In such cases there is frequently no other significant damage to the building, either structural or non-structural and (provided the settlement is uniform) the principal disruption to the building owners may only be damage to service connections, or problems with doors not opening or closing properly. In Dagupan, after the 1990 Philippines earthquake, some buildings which had settled by as much as 1 or 2 m were reported to be still occupied after the earthquake (Schiff, 1991).

3.4.2.1 Observed building damage in liquefied zones

Damage related to liquefaction beneath a building's foundations includes failure of unreinforced ground floor slabs due to bulging of the material beneath as the footings, walls and columns settle. This type of damage was observed in Adapazari (EEFIT, 2003). In other cases basements have been filled with sand ejecta due to liquefaction, as was observed in the Sodo district of Seattle after the 2001 Nisqually earthquake (Bray et al., 2001a) and in Taiwan after the Chi-Chi event. Such damage is relatively easy to repair and is probably equivalent to light structural damage in terms of a damage ratio. Settled buildings with no structural damage can be effectively economic write-offs due to flooding, as was the case at the Hotel Sapanca, Turkey (Figure 3-7). The damage induced by uniform settlements has been observed to be very dependent on the type of foundations, as is discussed later. Another secondary effect of
building settlement is the dragging down of neighbouring buildings, which for various reasons have not settled by the same amount, as was observed in Dagupan, in the Philippines.

![Figure 3-7: Flooding of the Hotel Sapanca, Turkey, 1999 due to liquefaction induced settlement. (Courtesy EEFIT, UK)](image)

A second type of damage caused by liquefaction is differential settlement and tilting, also related to volume change or bearing capacity failure, where there is variability of the foundation soil. If the liquefied layer is of non-uniform thickness, then there is every possibility of differential settlement occurring. The worst liquefaction-related damage in the Marina District of San Francisco in the 1989 Loma Prieta was on the boundary between the liquefied and non-liquefied zones, due to the settlement differential (Bardet & Kapuskar, 1993). As in the case of uniform settlement, when buildings tilt, there is very often no other observable damage; even window panes often remain unbroken. However, even a small angle of tilt can render a building unusable, and beyond a certain angle, a building may be beyond repair just as much as a building which has been reduced to rubble by ground shaking, rendering the direct losses the same. A secondary effect of tilt is the impact on to neighbouring buildings which can cause significant damage to both buildings (Figure 3-8).

There are presently no established guidelines as to how much a building can tilt or settle before it becomes irreparable; obviously this will depend to a large extent on the function and value of the building. The building in Figure 3-8 is clearly beyond repair. Re-levelling of houses which have undergone some differential settlement through the use of grouting or underpinning to correct buildings which are structurally sound has been reported (Ueng et al., 2001). As discussed in Section 3.2.1, and underlined by the information collated for this chapter, there is at present a notable absence of an index that classifies building damage due to permanent ground deformation (such as that shown in Figure 3-8) in terms of the repair cost ratio, or damage ratio.
Lateral spreading is potentially the most destructive liquefaction-related hazard to buildings: there have been cases where structures are heavily damaged by the lateral movement of the foundation soil. Such damage is unlikely to occur in piled buildings, or those with continuous tied reinforced shallow foundations. Buildings at the greatest potential risk to lateral spread-induced damage are those which are of poor design or construction quality, and located on or near to a slope or a free face in an area of liquefiable soils. Figure 3-9 shows a school building located close to a river bank, which was damaged beyond repair due to non-uniform settlement and lateral spreading towards the river during the Chi-Chi earthquake (Lew et al., 2000).
A canteen in the Naval Station in Guam was destroyed when half of it was carried towards the harbour due to lateral spreading (Comartin, 1995). At Hokkaido Harbour, in 1993, there was slight damage due to lateral spreading, and damage to piles at a primary school in Oshamanbe as a result of lateral spreading was reported by Berrill et al. (1993), although it is important to note that there was no associated damage to the superstructure in this case. The Hotel Sapanca (Figure 3-7) was damaged beyond repair due to a combination of settlement and lateral spreading carrying the building towards the lake (Cetin et al., 2002). The Marine Science Laboratory at Moss Landing, shown in Figure 3-10, was demolished subsequent to lateral spreading beneath its foundations in the 1989 Loma Prieta earthquake (Boulanger et al., 1997).

![Figure 3-10: Damage to the Marine Laboratory at Moss Landing, Monterey Bay, California during the 1989 Loma Prieta, 6.9 Mw, earthquake. Lateral spreading caused the buildings' pad foundations to spread apart. The laboratory was subsequently demolished. (Photograph: L.F. Harder, Jr.)](image)

### 3.4.2.2 Relative frequency of building damage due to liquefaction

Although some important examples of building damage due to liquefaction have been described in the previous section, it is important to consider these in conjunction with the relative frequency of such damage, both in individual earthquakes, and overall within the database.

In the 1999 Chi-Chi earthquake, the number of buildings reported to be severely damaged by liquefaction was 900 (Lee et al., 2001), which, although not insignificant, is only approximately 1% of the 100,000 buildings reported to be damaged in the earthquake. The spectacular liquefaction-induced damage in Adapazari following the 1999 Kocaeli earthquake caused ‘hundreds’ of buildings to tilt or settle (Youd et al., 2000) as compared to the tens of thousands of buildings reported damaged beyond repair over the entire region (e.g. Erdik, 2000). Of the 31 earthquakes where liquefaction was reported, damage to structures as a result was reported
for 14 events. Of these, nine were isolated cases, or small zones of building damage, such as the damage to one airport in Alaska, 2002, or to several buildings on coastal and lakeshore areas in the El Salvador 2001 events (Bommer et al., 2002b). The other five cases (i.e. 10% of the total database) were: Chi-Chi, Kocaeli, Luzon, Manjil and Manzanillo, Mexico. In each of these earthquakes, large areas of liquefaction-induced damage to buildings were reported, as in central Adapazari in Kocaeli (see Chapter 5) and in Dagupan in Luzon. Kobe is not included in this list, since despite the widespread damage and disruption, particularly in the port area, related to liquefaction, structures were actually not seriously affected, primarily due to the fact that the port buildings were mostly piled.

3.4.2.3 Influence of building foundations on response to liquefaction

The field reports studies herein clearly show that good foundations can significantly reduce damage due to liquefaction. In the Port of Kobe, where port buildings were mostly piled, settlement of up to half a metre of the surrounding soil was observed with no damage to the piled structures. Figure 3-11 shows a similar occurrence, although for piled bridge piers rather than building foundations. The Port Island Ferry terminal buildings were newly completed, supported on piles and strongly tied together to prevent differential horizontal movements. Despite severe liquefaction the building was reported to be undamaged and back in use within a few days of the earthquake (EERC, 1995). Hotels with piled foundations in Dagupan suffered little damage in the 1990 Luzon earthquake, and were able to remain functioning (Hopkins et al., 1991).

Figure 3-11: An elevated transport link in Kobe following the 17th January 1995 earthquake. The bridge piers were piled, and the structure is generally undamaged despite the settlement of the surrounding soil by up to 1m. (Photograph courtesy R.W. Boulanger, UC Davis)
Buildings in the Kobe region on piles with their toes anchored in firm ground suffered more damage due to ground deformations than those on floating piles founded in the liquefied layer due to differential movements between the two layers (Tokimatsu & Asaka, 1998). This illustrates how well-designed foundations for non-seismic loading conditions may not necessarily be the best solution in the event of an earthquake. Cases were reported of damage to piles due to lateral spreading of the surrounding soil, which did not translate into damage of the superstructure, such as the previously mentioned Oshamanbe Elementary School in the 1993 Hokkaido earthquake (Berrill et al., 1993). An industrial facility in the Kobe region, founded on piles, had a floating floor slab which settled around the piles, causing damage to machinery and pulling down the ceiling with it (EQE, 1995).

Where buildings are on either tied or continuous stiff shallow footings, as was generally the case in Adapazari, observations confirm that they are likely to settle uniformly or tilt as rigid bodies in response to liquefaction. Isolated spread footings or flexible tied footings are at risk of settling along with the columns whilst the non-load bearing walls remain at the higher level, or of settling by varying degrees. In the 1998 Adana-Ceyhan, Turkey, earthquake it was noted that buildings with well-designed and constructed continuous mat foundations and strong reinforced base slabs performed noticeably better in liquefied areas (Adalier & Aydingun, 2000). Following the Hokkaido-Nansei-OkI earthquake in 1993 it was observed that many buildings in areas of liquefaction were undamaged due to the fact that their foundations were tied together (Youd et al., 1995). Similarly, in Kobe, houses built on tied shallow footings, such as those in the Ashiya district, suffered little damage (Figure 3-12).

Figure 3-12: Undamaged houses very close to lateral spreading, Ashiya district, Kobe. (Courtesy National Information Service for Earthquake Engineering, University of California, Berkeley)
3.4.3 Liquefaction-induced transportation disruption

3.4.3.1 Roads and bridges

Damage to roadways due to liquefaction comprises settlement or lateral spreading of embankments, causing cracking and uneven surfaces. Such damage is generally minor compared to other potential damage to roads such as large slope movements or surface fault rupture. Settlements are usually easy to repair, at relatively low costs, and the disruption is short-lived. Some degree of damage to roads due to liquefaction occurred in at least 19 of the earthquakes in the database. In some of these, the damage was quite extensive, such as the widespread disruption to roads in Costa Rica in the 1991 earthquake, where transportation disruption was the most significant feature of the earthquake: Youd (1993) estimated that approximately 30% of the highway network within the affected region was disrupted due to liquefaction. After the Bhuj earthquake in 2001 the estimated cost of settlement and cracking damage to highways was US$125 million, less than 4% of the total direct losses, but nonetheless significant (Singh et al., 2002). Damage to transportation was one of the most significant features of the 1990 Luzon earthquake (see Section 3.3.3) but the contribution of liquefaction-induced road pavement damage to this was relatively small.

Bridge embankments are vulnerable to lateral spreading, due to the combination of susceptible river channel soils and a sloping or free face. Minor damage to bridges caused by liquefaction was reported in 30% of the earthquakes and this typically comprised disruption but no collapse, such as the Old Surajbadi highway bridge in Gujerat which was closed for several days after the January 2001 earthquake because of lateral spread-induced movement of the pier, and the Chi-Lu bridge in Taiwan where damage but no collapse due to liquefaction was reported (Wallace et al., 2001). The most common form of minor damage to bridges due to liquefaction was the settlement of the approach embankment fill, causing a difference in levels between the bridge deck and the approach road. Repair of such damage is very straightforward using temporary ramps or earth fill. In four of the earthquakes, bridge collapse was caused by liquefaction-induced movement, such as the failure of three spans of the Magsaysay Bridge in Dagupan (Luzon earthquake, 1990) shown in Figure 3-13, due to a combination of excessive movement of the bridge piers caused by lateral spreading and poor design (insufficient bearing width). At least five other bridges in Luzon were damaged due to liquefaction; this damage contributed significantly to the total transportation disruption, which caused significant economic losses by cutting off agricultural areas from their main markets in Manila (Booth et al., 1991).
Road closures due to extensive liquefaction-induced bridge damage in the banana growing region of Costa Rica after the 1991 Limón earthquake cost an estimated US$0.25 million every day in lost exports (Ballantyne et al., 1991). One span of an old railway bridge, more recently used for road vehicles, collapsed due to lateral movements of up to 1.2m in the January 2001 El Salvador earthquake. However, an adjacent bridge, built in 2000 was undamaged, so the resulting disruption was insignificant (Orense et al., 2002). Collapse of part of the Nishinomaya Bridge after the Kobe earthquake (Figure 3-14) was related to liquefaction-induced deformation (Park et al., 1995).

Figure 3-13: Collapse of the Magsaysay Bridge in Dagupan, after the 1990 Luzon earthquake. (Courtesy EEFIT, UK)

Figure 3-14: Collapsed span of the Nishinomaya Bridge following the 1999 6.9 Mw Kobe earthquake in Japan resulting from liquefaction-related foundation deformations. Ground cracks behind the quay walls and parallel to the water edge are indicative of the lateral ground movements that occurred. (Photograph: L.F. Harder, Jr)
3.4.3.2 Ports and harbours

Ports and harbours tend to be particularly vulnerable to liquefaction due to the combination of recently-placed fill and high water tables. The most significant case of losses directly or indirectly resulting from liquefaction in this study was the widespread damage to the Port of Kobe, which caused direct losses of US$11 billion (1995 dollars). 90% of the port’s 187 berths were destroyed due to the liquefaction of reclaimed soil used to construct Port and Rokko Islands. Most of the operations were transferred elsewhere, and the effect on the local economy was devastating, including loss of employment for many port workers. Prior to the earthquake, the Port of Kobe was the busiest port in Japan and the fourth largest container port in the world. Large lateral movements of the sea walls (up to 3m) occurred, combined with significant settlements that damaged tanks, silos, cranes and crane tracks (e.g. Figure 3-15). The shortage of usable space in Japan, and the location of Kobe between the mountains and the sea, necessitated the extensive reclamation of ground from the sea that led to these losses. Two important observations from the Port of Kobe were the general absence of damage to piled structures, and the effectiveness of ground improvement in mitigating the risk of liquefaction.

![Figure 3-15: Liquefaction of reclaimed fills at the Port of Kobe in 1995 caused complete suspension of operations. The lateral displacement of the quay walls in this picture pulled apart the crane legs causing collapse (Photograph: L.F. Harder, Jr)](image)

Of the 31 earthquakes where liquefaction was reported, 26 affected coastal areas. Of these 26, there were 12 cases of minor liquefaction-induced damage to ports and harbours, with no disruption of operations, and four cases of significant disruption to port operations as a direct result of liquefaction (including the Port of Kobe). The scale of the damage to the Port of Kobe is unparalleled. The port facilities at Nuweiba, in Egypt, which were significantly disrupted following the 1995 Gulf of Aqaba earthquake, comprised only four berths, of which two were
destroyed, and the facilities at San Fernando port which were damaged by the 1990 Luzon earthquake were also on a small scale. The port at Manzanillo suffered moderate damage in the 1995 Manzanillo earthquake, which had a significant economic impact on the city due to the reduced operations (Juarez Garcia et al., 1997). The same port was shaken again during the 2003 Colima earthquake, where it was noted that liquefaction occurred, but mainly in non-critical areas, probably as a result of the ground improvement measures implemented since the 1995 earthquake (EERI, 2003a).

3.4.4 Liquefaction-induced damage to lifelines

The continued functioning of lifelines after an earthquake is of vital importance for both the emergency response and the recovery of a community (Lund, 1996). The failure of the telephone system in the mountain city of Baguio after the 1990 Luzon earthquake meant that there was a significant delay before the authorities in Manila were able to fully appreciate the scale of the disaster. The disruption of power, gas or water supplies can extend much further than the areas of direct damage. Lund (1996) noted for example that in the wider area of Los Angeles after the Northridge earthquake, the lifeline disruption was not great, and this was a significant factor in the rapid recovery of normal routines for the local population. Liquefaction-induced lateral permanent ground deformation, which can vary from a few centimetres to tens of metres (in the case of flow failure), is one of the most pervasive causes of earthquake pipeline damage (Hamada & O’Rourke, 1992; O’Rourke & Hamada, 1992). The strains induced in lifelines crossing liquefied soils will be largest at the boundary between liquefied and non-liquefied materials.

Liquefaction can be particularly significant in the damage caused to lifelines, for a number of reasons:

- The nature of regional lifeline networks means that there are limited options for locations, and areas of poor soils are difficult to avoid.
- Deep foundations or ground improvement are rarely feasible along great lengths of lifeline networks.
- Buried pipelines in or close to liquefiable deposits are particularly susceptible to damage due to even small amounts of differential displacement of the ground surrounding the pipeline.
- Localised areas of liquefaction, which would not cause significant damage to most buildings, may be sufficient to cause pipes to bend or rupture, and a single failure in a
pipeline may be enough to cause an outage in the water or gas supply. The economic cost of repairing such a leak may be low, but the resulting loss of business in the area during the downtime may be significant.

Some disruption to lifelines as a result of liquefaction was reported for 30% of the earthquakes reviewed (i.e. about 50% of the earthquakes where liquefaction occurred). As noted above, it is generally understood that ground deformations are the main cause of pipeline damage (e.g. Schiff & Tang, 2000). Utility companies can also suffer significant losses due to the loss of their customer base where a large number of houses have collapsed, as was the case for gas companies in Taiwan in 1999 (Schiff & Tang, 2000).

Examples of lifeline disruption due to liquefaction include the Piti Power Station in the 1993 Guam earthquake where differential settlement of the turbines, plus failure of buried pipelines, caused major disruptions to the power supply for at least a week after the earthquake (Comartin, 1995). Breakage of buried gas or water pipes due to liquefaction settlement occurred in Oshamanbe following the 1993 Hokkaido-Nansei-Oki event (Yamazaki et al., 1995), in Sapanca and Adapazari following the 1999 Kocaeli earthquake, in Kobe in 1995 and also in the 1990 Luzon and 1991 Costa Rica events. In the case of the Kobe earthquake, water supplies were interrupted for several months. Additional disruption in the Kobe area included the damage to wastewater plants located on reclaimed ground where liquefaction occurred, and, to a lesser scale, tilting of electricity distribution poles due to ground failure. Flotation of buried tanks is a further liquefaction-related hazard to utility systems.

In the city of Adapazari, affected by both liquefaction and strong ground shaking in the 1999 Kocaeli earthquake, lifelines fared very badly: the destruction of the water distribution network was cited as being “one of the most remarkable examples of system failure in any historic earthquake” (O’Rourke et al., 2000b).

In terms of financial losses, the Osaka Gas Company, which supplies some 5.7m households in the Kobe and Osaka area, reported US$170million in repairs, and US$68 million in lost revenues.

A significant hazard related to lifeline damage, which does not arise within this database but should not be discounted on that basis alone, is that of fire following earthquake. Gas pipeline rupture can be the trigger for fires, and water pipeline rupture can seriously impede the ability to control fires (Figure 3-16). Both of the aforementioned types of damage are frequently associated with the occurrence of liquefaction or surface fault rupture. The damaging potential of a rapidly spreading fire is very significant.
3.5 Damage caused by fault rupture

3.5.1 Surface fault rupture trace

Buildings, structures and lifelines can be damaged by fault rupture. A spectacular example is the complete failure of the Shihkang Dam in Taiwan, due to approximately 8m differential vertical surface fault rupture in the 1999 Chi-Chi earthquake (Figure 3-17). Of the 50 earthquakes in the database, fault rupture was mentioned in damage reports for only nine (18%). It is possible that surface fault rupture did occur in some of the other events, but without causing significant damage.

Figure 3-16: Liquefaction-induced ground deformations ruptured gas and water pipelines beneath Balboa Boulevard after the 1994 Northridge earthquake. In this case the fire was controlled and did not therefore cause significant damage.

Figure 3-17: Failure of the Shihkang Dam due to vertical fault displacement, Chi-Chi, Taiwan earthquake, 21 September 1999.
Of the nine earthquakes with reported surface fault rupture, there was no resulting damage in two. In Khalari, India (1992) the observation of surface fault rupture was remarked upon since it was surprising for a stable continental region earthquake (Gupta et al., 1998). In Manjil, Iran (1990) there was 100 km of surface fault rupture, with horizontal offsets of up to 20 cm, and vertical offsets of up to 50 cm. There is no discussion of any damage caused by these relatively small displacements (e.g. Ishihara et al., 1992).

In only one event, the 2002 Denali earthquake in Alaska, could it be said that the fault rupture was the primary reason for the damage and losses. The remote location of the earthquake meant that there was relatively little structural damage observed. The earthquake was of great engineering interest in being the first test of a structure built to withstand major surface fault rupture displacements, in this case the Trans Alaskan Pipeline System (TAPS). The fault rupture was 335 km long, and the average slip was about 5 m (EERI, 2003b). Fault rupture poses a greater hazard for lifelines than for structures in general, since it is possible to relocate structures away from known faults, where the relocation of a linear feature such as a road, railway or pipeline is often not feasible. The TAPS performed very well in that the very large displacements were accommodated without failure. There was some damage to the support structures, which required minor repairs, but no pipeline damage. The pipeline was shut down for two days for safety checks, costing approximately US$60 m in lost oil deliveries (EERI, 2003b). Fault rupture damage to roads in Alaska was also reported. Although overall the damage caused by this earthquake was minor and its impact on the region was low, the TAPS is arguably the most economically significant facility in the region, and since the only damage to it was due to surface fault rupture, it is unique within the database in that surface fault rupture was the primary contributor to the total earthquake losses.

Figure 3-18: Displacement of the trans-Alaskan pipeline system at the crossing of the Denali fault following the 3 November 2002 Mw 7.9 Denali earthquake
In Chi-Chi (1999) there was significant damage along the entire 90 km of surface fault rupture, including the Shihkang Dam (Figure 3-17) and the collapse of four bridges due to large displacements between their supports (Schiff & Tang, 2000). Roads were heavily damaged by the vertical fault offsets and there was substantial damage to water pipes crossing the fault trace. Whilst initially disruptive, vertical road offsets can at least be rapidly repaired using temporary ramps. Buildings directly on top of the fault collapsed, although buildings very close by were frequently undamaged, as was the case in the city of Fengyuan (JSCE, 1999). The overall impact of the earthquake was high, and damage was caused by ground shaking, landslides and liquefaction as well as fault rupture. Despite the fact that the damage caused by the fault rupture was unprecedented, it seems that the damage caused by each of the other hazards was equal or greater. However, if there had been catastrophic flooding due to the Shihkang Dam failure, then there might have been a very different conclusion.

In the 1999 Kocaeli earthquake, approximately 120 km of surface fault rupture was mapped with between 2 and 4 m horizontal displacement. One bridge collapse was directly caused by the fault movement, as was damage to roads and railways. Further surface rupture of the same fault in the 1999 Düzce event was a contributing factor, along with ground shaking, to the damage of the Bolu Viaduct of the Great Anatolian highway (Priestley & Calvi, 2002). In fact, fault rupture was the main cause of damage to roads and railways in the Kocaeli earthquake, and bridge damage due to other hazards was minor. Buried water pipelines were severely damaged, including those supplying industrial facilities, which added to the indirect economic losses of the earthquake. Buildings directly on the fault were destroyed – the number of buildings affected in this way was reported to be “a large number” (JSCE, 2000), “many” (Erdik, 2000), or “over a hundred” (Lettis et al., 2000), including the Gölcük Naval Base and the Ford-Otosan factory under construction in Gölcük (Figure 3-19). Since around 80 000 buildings were destroyed and a further 80 000 heavily damaged in the earthquake (Elnashai, 1999), it is concluded that fault rupture was not a primary contributor to building collapse. As in the Chi-Chi earthquake, it was noted that the structures very close to the fault rupture were sometimes completely undamaged.

A secondary fault-related feature was the significant tectonic subsidence of coastal areas of Gölcük, resulting in up to 3 m drop in level of a 4 km long coastal strip (Erdik, 2000) and an associated loss of buildings due to flooding. Such losses are difficult to predict and have not been observed in any other earthquake reviewed in this study. Interestingly, in many areas, the boundary of the subsided zone appeared to follow existing boundaries between orchards and pastures or timber forest, suggesting that it may be a reactivated feature (EEFIT, 2003).
Fault rupture extended over 100 km in the 1990 Luzon earthquake, with around 6 m horizontal and 2 m vertical displacements, causing structural damage to buildings lying on the fault in the small town of Rizal. As is often the case, there was a marked absence of damage close to the fault (EQE, 1990; Booth et al., 1991). Surface fault rupture caused the collapse of one bridge and damage to some roads, in particular to the economically significant Dalton Pass in the Central Plains region, linking Manila with the agriculture and industry of North Luzon. However, the majority of the bridge and road damage in the Luzon earthquake was caused by either liquefaction or landslides. Damage and disruption to the transportation network was undoubtedly the most significant feature of the Luzon earthquake and fault rupture was a contributing factor, but was not the primary cause. The effects of this earthquake would have been almost as disruptive without the surface fault rupture.

In many cases, appropriate foundation design can limit or prevent damage to buildings crossing a fault. For example, it was observed that massive concrete structures such as bunkers at Gölcük Naval Base influenced the path of the surface fault rupture in the Kocaeli earthquake (Lettis et al., 2000). Lifelines of major importance can be specially designed to accommodate large displacements as in the case of the Trans Alaskan Pipeline System. For most pipelines, such design is unlikely to be economically viable and therefore for these, and for roads and railways, it appears that emergency planning and allowing for some redundancy are the most relevant mitigation approaches.
3.5.2 Tsunamis

Tsunamis caused by earthquake faulting, although not falling into the category of ground failure, are still collateral hazards, in the same way as the three types of ground failure hazard discussed previously. Therefore tsunami damage is briefly included in this chapter for completeness.

The extremely damaging potential of large tsunamis is well known, notable examples within the database include the Flores Island, Indonesia earthquake in 1992, where a large tsunami devastated much of the island, damaging houses, schools and hospitals as well as large sections of road, causing 2200 deaths in combination with the ground shaking (Pribardi & Soenmardi, 1996). The Hokkaido-Nansei-Oki event of 12 July 1992 was followed within a few minutes by tsunami waves up to 30m high, causing 208 of the total of 231 fatalities, and being also the main cause of building damage and the cause of significant disruption to earthquake rescue and recovery. In Papua New Guinea, an otherwise non-damaging earthquake in 1998 produced a devastating tsunami which left no structures standing in the two worst hit coastal villages. An important difference between the two latter events was the degree of preparedness – residents in Hokkaido immediately evacuated the low-lying coastal regions, due in a large part to their own memories of tsunami damage. This saved many lives even though the destruction was still extensive. In Papua New Guinea on the other hand, the community was very poorly prepared, despite having over 20 minutes between the earthquake and the tsunami waves. The last tsunami in that region had been in 1935, beyond the living memory of most of the residents.

Of the earthquakes reviewed, tsunamis were reported in only 14%. Of these, two were non-damaging, such as the tsunami that reached the south coast of the Balearic Islands following the Algerian earthquake in May 2003. In Manzanillo, Mexico, structures were destroyed by a tsunami after the 1995 earthquake, including the collapse of one beach front hotel already weakened by ground shaking (Juárez García et al., 1997) but overall the tsunami-related damage was less significant than that due to ground shaking. In four cases (Flores Island, Hokkaido-Nansei-Oki, PNG and Peru, 2001) the tsunamis were undoubtedly either the primary cause of damage, or at the least equal to ground shaking in terms of the damage and destruction caused. These numbers suggest that damaging tsunamis are rare events, but when they do occur, they tend to overshadow all other hazards, to devastating effect.

A perceived difficulty in incorporating tsunamis into loss estimations is that they can affect areas very far from their point of origin, and the seismic sources may therefore not be considered within the study area. This was not a feature of the limited number of tsunamis
reviewed herein, since all the damaging tsunamis were generated by earthquake faults in the vicinity of the worst affected zones.

3.6 Ground failure and ground shaking as causes of damage

Ground failure, rather than direct ground shaking, is the focus of this chapter, as well as of the thesis overall. Nonetheless, the relative contribution of ground failure cannot be usefully evaluated without including ground shaking by way of comparison.

3.6.1 Damage caused by ground shaking

Collapse of buildings due to ground shaking is widely believed to be the principal cause of earthquake casualties; this is clearly supported by this review. Of the 50 earthquakes reviewed, hazards other than shaking were the principal cause of fatalities in only seven events, four of which involved massive loss of life due to catastrophic landslides (Ardebil, Iran, 1997; El Salvador, 2001; Paez, Colombia, 1997 and Luzon, Philippines, 1990) and three of which triggered devastating tsunami that claimed all or most of the earthquake casualties (Flores Island, 1992; Hokkaido-Nansei-Oki, 1993 and Papua New Guinea, 1998).

Ground shaking causes damage to commercial, industrial, residential and public buildings. Shaking can destroy lifeline systems from supply (generating, processing or treatment plants) through to demand (collapse of houses, offices or factories). Shaking damage to utilities most often comprises damage to the associated structures at water treatment plants, telephone control centres, power plants or electricity substations. The damage to the power plant at Ilo following the 23 June 2001 earthquake in southern Peru was caused entirely by strong ground shaking (Rodríguez-Marek & Edwards, 2003). The most significant transport disruption is frequently caused by shaking-induced damage to bridges, such as the notable cases of the collapse of the elevated Hanshin Expressway in Kobe (1995) and the Nimitz Freeway collapse in Loma Prieta (1989). Roads and railways have a low vulnerability to ground shaking and direct damage to road pavements and railway tracks tends to be related to permanent ground deformations due to lateral spread, settlement, slope movements and surface fault rupture. Other than bridge damage, ground shaking can indirectly disrupt transportation networks through the structural damage to stations or tolling booths, collapse of buildings onto city streets, or by damaging power generation and transmission facilities, where power failures halt railway operations and cause disruption to traffic control systems.

The transient loading caused by ground shaking is potentially damaging to pipelines although it is generally acknowledged that the permanent ground deformations caused by ground failure are
typically more damaging to pipelines. Restrictions on route alignments combined with pressure to locate major pipelines away from built up areas frequently lead to their passing through remote, mountainous terrain or other areas where either topographic or soil amplifications of ground shaking are probable.

The 1990 Luzon earthquake and the 1999 Chi-Chi earthquakes were particularly noted for the widespread geotechnical effects, but even in these events, all the available damage reports suggest that shaking dominated the damage. In Luzon, there was widespread shaking-induced building collapse, particularly in the mountain city of Baguio where ground shaking was amplified by topographical effects. In the Chi-Chi earthquake, at least 10,000 buildings were reported to have collapsed, and although there are no breakdowns of the various causes, it is clear from the damage reports that a significant number of these were a direct result of very strong ground shaking combined with common deficiencies in design and construction. In the Kobe earthquake, liquefaction damage had a significant impact on the earthquake losses: however, in the breakdown of direct costs given by Hagiwara et al. (1997), building losses were responsible for 58% of the total direct losses, and losses to roads and railways, almost entirely due to ground-shaking related bridge collapses a further 10%, whereas damage to ports caused only 10% of the total losses. These statistics suggest that ground shaking was the dominant cause of the massive earthquake losses. EQE (1995) noted that the business interruption losses would ultimately be higher than the repair costs for the Kobe earthquake, in which case the relative contribution of ground failure is likely to be larger.

Naeim and Lew (2000) summarise the current state of seismic engineering, highlighting lessons that should have been learnt from recent earthquakes, but which continue to be the main reasons for injury, death and destruction. Of these lessons, two (improper land-use and -zoning, and improper construction on liquefiable soil) relate to geotechnical failures, the other eight are all related to the design and construction of structures to resist shaking. Shaking-induced damage continues to be the principal issue related to earthquake damage. Shaking is judged to be the main, or only, issue in well over 50% of the earthquakes reviewed, as well as the four more recent damaging events in Algeria (May 2003), Bingöl, Turkey (May 2003), Colima, Mexico (January 2003) and Bam, Iran (December 2004). When the entire affected region is considered, ground shaking is nearly always the main cause of building damage. For the 1990 Luzon earthquake, if smaller zones are considered, then Dagupan city was affected mainly by liquefaction, the town of Rizal mainly by fault rupture, and some of the remote mountain villages mainly by slope failure. However, considering the affected region as a whole, then liquefaction, fault rupture and slope failure each impacted only small areas, whereas ground
shaking caused damage over the entire zone, including within Dagupan, Rizal and the mountainous area. Tsunamis are an exception to this observation, since they have the potential to destroy very large areas.

3.6.2 Interaction between ground shaking- and ground failure-induced damage

An important issue for the incorporation of ground failure into earthquake loss estimations is how to combine ground failure-induced damage with damage caused by ground shaking. In the HAZUS methodology described in the next chapter for example, the two hazards are assumed to be independent, i.e. the probability of collapse due to shaking is not affected by the probability of collapse due to ground failure. Practically, this means that a building on liquefiable soil or an unstable slope will have a higher overall probability of collapse, but this collapse is only caused by one hazard – there is no allowance for a building to be partly damaged by ground shaking and partly damaged by ground failure, thus producing an overall state of severe damage. These assumptions are explored further in the light of the data gathered for this review.

As demonstrated by Berrill and Yasuda (2002) and Lopez (2002), a significant amount of shaking can occur before liquefaction occurs, so a structure may, at least theoretically, suffer ground shaking-induced damage followed by ground failure-induced damage. The alternative scenario is that liquefaction effectively damps out much of the strong shaking in the high to mid-frequency range, acting as a base-isolator for overlying structures. The case histories have been carefully reviewed to identify the empirical evidence in this respect.

Many field reports note a beneficial effect to structures in liquefied zones. For example, in Kobe (1995) "residents of a house in a liquefied area of Nishinomiya City reported that, during the earthquake, their furniture remained upright and they did not feel severe shaking" (Shibata et al., 1996). Also in Kobe, an ‘Earthquake Disaster Zone’ was identified where over 30% of the traditional wooden houses collapsed (Matsui et al., 2001) but the areas of the most severe liquefaction and lateral spreading were outside this zone (Figure 3-20). Structures in liquefied areas tended to suffer little damage (e.g. Figure 3-11 and Figure 3-12). An observation of the damage in Adapazari in the 1999 earthquake was that the liquefied soil "acted as an isolator, dissipating the energy at the foundation level and avoiding shaking damage to buildings" (Erdik, 2000). This observation is possibly supported by the hundreds of settled and tilted buildings with little or no structural damage (e.g. Figure 3-5, Figure 3-6 and Figure 3-8). On the other hand, where buildings in Adapazari suffered complete collapse, as many did, it is impossible to determine the extent of ground failure beneath the collapsed building. A more
detailed discussion of the interaction between shaking and liquefaction in Adapazari is presented in Chapter 5.

**Figure 3-20:** Map showing areas with heaviest building damage versus areas of liquefaction in the Kobe region following the 1995 earthquake (Matsui & Oda, 1996)

In Dagupan, after the 1990 Philippines earthquake, some structural damage such as collapse of roofs and walls occurred outside the central zone, but within this zone, where up to 90% of the buildings settled and tilted due to liquefaction, there was very little structural damage (EQE, 1990). Similarly, in Guam, after the 1993 earthquake, it was clearly observed that the major structural damage occurred to hotels where no ground failure was observed, whereas at the Piti Power Plant, ground failure occurred and there was little structural damage (Comartin, 1995). In the Marina District of San Francisco, much attention was given to the occurrence of liquefaction in the Loma Prieta earthquake, but "the high concentration of structural damages in this district was primarily due to strong shaking" (Seed et al., 1991).

There are occasional references in the field reports to cases of ground shaking and ground failure damaging the same structure. In Wu-feng, Taiwan, a number of 5-storey condominium buildings collapsed due to a soft ground storey, and adjacent buildings showed evidence of foundation damage due to ground failure "which may have contributed towards the structural damage" (Stewart et al., 2001). Observers in the 1990 Luzon earthquake reported that 45-50 seconds of strong shaking caused wooden structures to collapse before liquefaction-induced settlement took place (Schiff, 1991), and elsewhere in the Philippines, the collapse of the important Carmen bridge was reported to be due to a combination of liquefaction and ground shaking (Hopkins et al., 1991). The AIJ (2001) report a building affected by settlement, which, although appearing undamaged structurally from the outside, revealed "severe damages to the in-filled walls and the finishing, which were caused by the seismic lateral vibrations in addition
to the soil liquefaction, were observed by the inspection to the inside of the building”. A further case was reported by Juárez Garcia et al. (1997), shown in Figure 3-21.

Figure 3-21: A damaged building in Jaluco, following the 1995 Manzanillo, Mexico, earthquake (Juárez Garcia et al., 1997). The caption accompanying this photograph reads “Structural damage caused by strong motion in combination with liquefaction and subsidence”. Although not clarified, the ground shaking damage referred to could be the fallen debris in the foreground.

For landslide-induced damage it is feasible for a building to be damaged by shaking prior to the failure of the slope on which it is located. However, in nearly all cases, the damage caused by landslides is complete, and the occurrence or absence of damage due to ground shaking is immaterial to the affected structures, although, as noted before, not to the number of casualties. With respect to lifeline damage, the cause of damage at a given location is generally either shaking damage to ancillary structures, or ground deformation damage to pipelines. Across a lifeline network or the length of a major pipeline, damage can be due to any or all of the hazards considered in this chapter.

In conclusion, there is certainly a mechanism for buildings to be strongly shaken before the onset of liquefaction or landslides, their final damage state thus being caused cumulatively by ground shaking and ground failure, contrary to the assumption in HAZUS (FEMA, 2003). A few case histories supporting such an occurrence have been identified. Nonetheless, of the 23 earthquakes in the database where liquefaction caused damage to buildings or bridges, only five
isolated observations of combined damage have been found. In the remaining cases there tends to be a reasonably clear delineation between zones of liquefaction, with damage to foundations or settlement or tilting of the building, but relatively light structural damage, zones of landslide-induced damage, and zones of structural damage with no evidence of ground failure, with the casualties, repair costs and disruption all being significantly higher in the two latter cases than in the former. It is true that where buildings have suffered pancake collapse it is very difficult to spot evidence of ground failure, and therefore in susceptible zones it is impossible to determine whether ground failure contributed to the collapse. Where landslides have damaged buildings, any previous damage caused by ground shaking is largely immaterial since most buildings are unlikely to survive the landslide. For these cases it would be reasonable to assume that damage is caused by one or the other hazard, particularly in the light of the other uncertainties related to estimating earthquake losses. The relative likelihood of cumulative damage due to combined shaking and liquefaction is something that merits further investigation.

3.7 Summary of earthquake hazards and damage to the built environment

The earthquake review presented in this chapter has focussed on damage and disruption caused by the earthquake hazards of landslide, liquefaction, fault rupture and tsunami, and contrasts these with the direct impact of ground shaking. The empirical data from the review of 50 recent damaging earthquakes is summarised in this section to provide useful guidance on the relative significance of different hazards to different damage scenarios. Figure 3-22 shows the distribution of the causes of damage to buildings for each earthquake. This figure does not distinguish the extent of damage, i.e. whether the overall damage to buildings was slight, moderate or severe, it just considers the cause of the damage. In 88% of the earthquakes reviewed, ground shaking was the most significant cause of building damage. Ground failure was the primary cause in 6% of the cases, and tsunamis in another 6%. These statistics include several earthquakes where two or more hazards were considered to have equally significant impacts. For example, in Manjil, Iran, 1990, ground shaking damage to adobe structures, liquefaction damage in the deltaic regions leading to the Caspian Sea, and the two major landslides were all responsible for the overall devastation of the region (Ishihara et al., 1992). These individual hazards cannot easily be separated sufficiently to identify the main one. In just under half of the earthquakes, more than one hazard combined to cause the overall damage to the building stock, but in these cases it was possible to identify which hazard caused the most damage and which was the secondary cause. If damage due to ground shaking is discounted, then ground failure becomes an important hazard, with landslides and liquefaction being the
second most significant cause of building damage in 32% of the cases (Figure 3-22b). However, in 54% of the earthquakes, ground shaking was the single cause of building damage.

Figure 3-22: Causes of earthquake damage to building stock, a) primary cause; b) secondary cause. Where there are two causes of damage, there is generally a clear delineation between the different zones. In some earthquakes there are more than one equal causes of damage, and in others there is only one cause, in which case the secondary cause is defined as 'None'.

Figure 3-23a shows the primary causes of damage and disruption to transportation networks in 12 earthquakes where transportation damage was significant, i.e. where bridges collapsed or where major routes were closed following the earthquake. This category also includes major damage to ports and harbours. In some earthquakes two or three hazards were assigned equal importance. Shaking, landslides and liquefaction are approximately equally distributed in this figure. Where the damage or disruption was caused by shaking, this was typically due to either collapse or major damage to bridges, whereas the landslide-induced damage and disruption was typically in the form of blocked major routes, sometimes for many months (e.g. January 2001 El Salvador). In 36% of all the earthquakes, there was some disruption to the transportation system, which was mainly temporary and repairable, and therefore categorised as moderate. In these cases, shown in Figure 3-23b, liquefaction was the dominant hazard, closely followed by landslide. Moderate damage caused by liquefaction can include displacement of piers with no collapse, lateral spreading of road and rail embankments and settlement of roads or railway tracks. Moderate damage caused by landslides includes rock falls or slope failures in cuttings or embankments, temporarily or partially blocking routes.

For clarification, the term 'secondary cause' refers to the second most significant cause of earthquake damage to building stock in each earthquake, not to the cause of secondary damage in individual buildings, and the distribution is on an earthquake by earthquake basis, rather than for the total number of damaged buildings across all 50 earthquakes.
Utilities networks suffered significant damage in 14 earthquakes, and moderate damage in 19. Where damage was significant, ground shaking was again the main factor, governing the lifeline damage in 45% of cases, with liquefaction governing in 37% (Figure 3-24a). Ground shaking was also the most significant hazard for utilities in earthquakes where the overall utilities damage was moderate (Figure 3-24b). This shaking damage to lifelines typically comprised damage to the associated buildings, plant or infrastructure, for example at water treatment plants or at power generation facilities.
3.8 Implications for loss modelling

As shown in Chapter 4 of this thesis, most earthquake loss estimations focus exclusively on losses induced by ground shaking. Where secondary hazards such as ground failure and tsunamis are incorporated, the methodologies are shown to be poorly developed in comparison to those used for ground shaking assessments. Particularly in the case of ground failure, the incorporation of such hazards into a loss estimation requires either considerable additional data and analysis, or a number of significant assumptions and simplifications to be made regarding the susceptibility of soils to ground failure and the resulting damage to buildings and infrastructure. All of these issues are addressed in greater detail in later chapters.

On the basis of this review, guidelines for the planning of an earthquake loss estimation study are developed. Firstly, it is suggested that for a loss study with a regionally distributed portfolio, seeking a preliminary estimate of building damage, it would be sufficient to consider only ground shaking-induced building damage. This simplification would generally produce estimates within the correct range, since it has been shown that ground shaking almost invariably dominates the building damage over the entire affected region (Figure 3-22). As the study area becomes smaller and more focussed the potential of a secondary hazard such as landslide, liquefaction or fault rupture dominating the nature and distribution becomes greater. The potential for liquefaction to dominate in a smaller region was observed in central Dagupan or downtown Adapazari. Although rare, when tsunamis do occur it is very likely that they will cause devastation to the coastal area. A loss estimation for a coastal zone which ignored the possibility of tsunami occurrence could be virtually meaningless.

The importance of building foundations has been highlighted in this chapter. Where landslides, liquefaction or fault rupture have occurred there has been a clear pattern for buildings which are either piled or on continuous or tied shallow foundations to perform significantly better than those on flexible shallow footings. Other parameters observed to influence the performance of buildings subjected to ground failure, particularly liquefaction, include the height, aspect ratio and proximity to other buildings, as well as the variability of the liquefied soil (e.g. Sancio et al., 2002). Knowledge of foundation systems for existing buildings may be difficult to obtain but is essential for the correct modelling of building performance due to ground failure. The difficulty lies in the fact that even visual inspection cannot reveal the foundation type. There are some assumptions that could potentially be made regarding the foundation type, for example that a structure which has been designed to meet modern code requirements will have appropriate foundations and resistance to ground deformation. In loss estimation methodologies, building inventory classes are usually related to both the age and the height of
the building. In order to represent different levels of design and classification of foundation systems for the purposes of estimating future losses due to ground failure, the same variables should be considered. With local knowledge and judgement, the foundation system, and the associated degree of uncertainty can be estimated.

Comprehensive loss estimations include transportation damage and disruption. Financially, the losses (at least to roads) are less significant than those due to collapse of buildings, but the performance of the transportation system can have a vital role in the emergency response and the subsequent recovery of a region. In the El Salvador earthquakes of 2001, the total cost of repairing damage to roads and bridges was reported to be $2.7 m and $12.4 m respectively (JSCE, 2001). However, the damage to roads was major and widespread, whereas there were only a few reports of bridge damage, which was classified as minor, indicating the difference in terms of direct losses. Indirect losses will also be affected by transportation disruption. Damage to ports can potentially be very expensive and disruptive, and ports are particularly susceptible to liquefaction due to their combination of high water levels and man-made fill.

Unlike the case of damage to buildings, it is not appropriate to consider only ground shaking-induced damage for the transportation category. Situations where transportation losses may be particularly important include indirect losses caused by road blockage in mountainous regions, or both direct and indirect losses caused by bridge damage in liquefiable zones. In such cases, consideration of the three main hazards of ground shaking, liquefaction and landslide is essential. Fault rupture can also be important, causing significant damage in 9% of the cases. Where existing facilities, bridges or lifeline networks within a loss portfolio are identified to cross a fault, the probabilistic fault displacement hazard analysis (PFDHA) approach described by Youngs et al. (2003) can be incorporated. Tsunamis must be considered in susceptible coastal regions.

Estimations of future losses to lifeline networks also cannot afford to ignore the contribution of landslide and liquefaction, particularly where indirect losses or emergency planning are important. Functioning lifelines have been shown to be essential for both emergency response and for post-earthquake recovery, and will also affect the indirect losses (see Figure 3-25). Damage to lifeline systems can be a combination of the damage to the associated structures, plant and pipelines. These components are vulnerable primarily to ground shaking but also to ground failure hazards, particularly to permanent ground deformation in the case of buried pipelines.
Figure 3-25: Does a preliminary loss model need to incorporate the complex components of ground failure in addition to considering ground shaking? This simplified decision tree shows some of the key factors that will influence this. For a single site loss model, it is expected that ground failure would always be included. Other factors not included in this figure include quality of available data, time and resources available, knowledge of foundation types and end user of loss results.
The continued functioning of businesses, industry and critical facilities such as schools and hospitals relies not only on good structural performance under earthquake loads, but also on the performance of the surrounding environment. Examples have been presented of the importance of ground failure on the transportation and utility systems. A complete picture of the losses a region will experience needs to include the interaction of the different categories and the indirect losses that the blockage of a road or the closure of a port will cause. The potential for such indirect losses is illustrated by the New York blackouts in August 2003, which cost the city an estimated $1.1 billion, even without the associated damage costs that an earthquake would incur. The interaction that leads to indirect losses is a complex system. However, what becomes clear is that whilst shaking damage is invariable the primary factor leading to earthquake-induced losses, failure to appropriately model the effects of ground failure, particularly on transportation and lifelines, could lead to an unrealistic picture of the post-earthquake state of a region.
4 EARTHQUAKE LOSS ESTIMATION METHODS

This chapter presents a critical review of methods used for loss estimation studies around the world. The history of loss estimation methodologies is still relatively young, and there is no coherent approach globally recognised as being 'standard practice'. One reason for the diversity in current methods is the dependence upon the motivation for the study: whether this comes from a financial (insurance industry) perspective, a parochial view such as local planning organisations, or from nationally or internationally funded research. The potential applications of earthquake loss estimation have been briefly presented in Chapter 1.

This chapter considers each of the first three elements of a loss estimation methodology (hazard, inventory and damage, see Figure 4-1) in turn, and how each of these can be dealt with, with reference to published methodologies. Not all published methodologies include all of the components required to produce a complete estimation of earthquake losses, as will be shown. Full loss models need to incorporate many disciplines, including financial modelling, social issues, and emergency response. However, this chapter deals primarily with the elements related to engineering aspects of loss estimations, i.e. the hazard, the vulnerability and the expected damage of the exposed infrastructure.

![Figure 4-1: Components of an earthquake loss estimation (after FEMA, 1994)]
As explained in Chapter 1, techniques used for site-specific design cannot be employed in regional studies, because the demands of time and data acquisition would be prohibitive. Therefore methods are required that are capable of dealing with the issues of spatial uncertainty and variability in ground conditions and exposed building stock – these features can not be eliminated.

This chapter only considers issues relevant to regional loss estimations, although it should be noted that loss estimations may also be carried out for specific buildings, whose owners require the information for the purposes of managing their future risks (e.g. Kiremidjian & Porter, 2001). In these cases the uncertainties are of a different nature, being related to performance and repair of individual structural and non-structural components. Furthermore, only the estimation of future losses is considered in this chapter, as opposed to post-earthquake damage evaluation for either financial or emergency planning purposes.

4.1 The historical development of loss estimations

Much of the early work on regional earthquake loss estimations was carried out in the western United States, particularly California. Of the regions of the world exposed to high seismic hazard, California is one of the wealthiest and most technologically advanced. As such, it is not surprising that the concerns extending beyond basic life safety and prevention of building collapse to protection of assets and control of economic losses were first raised here.

Earthquake loss estimation began in 1972, with a study by the National Oceanic and Atmospheric Agency for San Francisco by Algermissen et al. (1972). Between 1972 and the early 1990s, a large number of loss estimation studies were carried out for regions, cities, districts or particular lifeline systems of the USA, without arriving at a clear and defined methodology (Reitherman, 1985). One of the most influential publications was ATC-13 (1985) "Earthquake Damage Evaluation Data for California", which has been used as the basis for many subsequent regional studies (e.g. McCormack & Rad, 1997; Rojahn et al., 1997; Escuela Politecnica Nacional et al., 1994). ATC-13 set a new standard through its comprehensive methodology for the evaluation of earthquake damage caused by ground shaking and collateral hazards. The recognised drawback of ATC-13 was the lack of observational data with which to correlate evaluated losses. In the absence of such data, the methodology relied almost entirely upon expert judgement. Since the publication of ATC-13, the 1989 Loma Prieta and the 1994 Northridge earthquakes provided significant quantities of observational data and allowed major advances to be made in the field of loss estimation for California.
The massive data handling and storage requirements of a loss study have also been one of the limiting factors in the development of methodologies. The earliest studies used hard copy storage of data, which was very time consuming, liable to errors and difficult to update. Subsequent developments used database storage, and more recently, the development of Geographical Information Systems (GIS) software has proved an ideal tool for storage and management of large databases, enabling rapid assessment of many earthquake scenarios, and presenting the results in a useful manner (King & Kiremidjian, 1994). GIS also allows different types of hazard such as ground shaking and ground failure to be combined through the use of ‘layers’ of data. The principle advantage of using GIS is undoubtedly the ability to present results visually and graphically, since many of its other features, as shown in Figure 4-2, are not exclusive to GIS and could also be implemented into database or even spreadsheet-based systems. Nonetheless, the importance of the visual presentation should not be undermined, since it is this that enables the end result to be understood, disseminated and also compared to other areas or hazards (Figure 4-3).

Figure 4-2: The role of GIS in regional seismic hazard and risk analyses (after King & Kiremidjian, 1994)

Once a database is in GIS, it can be easily updated with changes to anything from the building stock to the damage assessment methodology. Common GIS files can also be used by all bodies involved in both the routine management of infrastructure and disaster response. There is a clear trend within the methodologies studied for this chapter towards the use of GIS platforms.

Since 1994, efforts led by the US Federal Emergency Management Agency (FEMA) and the National Institute of Building Services (NIBS), have been focused on the derivation of a
comprehensive national methodology, implemented in GIS, which resulted in HAZUS (FEMA, 2003).

![Figure 4-3: Example of the use of GIS for the assessment and presentation of loss estimation for the metropolitan area of Lisbon. Left-hand figure shows the distribution of macroseismic intensity values for a repeat of the 1755 M8.7 earthquake, and the right-hand figure shows the predicted number of completely damaged buildings in each district (Sousa et al., 2004)](image)

### 4.1.1 HAZUS

HAZUS is a program for the complete assessment of earthquake-induced losses developed for the US by the Federal Emergency Management Agency (FEMA, 1999; 2001; 2003). HAZUS is the only complete and transparent methodology that is both publicly available and fully documented, and represents a major advance in earthquake loss estimations (Khater et al., 2003). The primary intended users of this methodology are state, regional and community governments. The software can be used at various levels of detail, and allows preliminary loss estimations using only the default databases included with the programme.

The scale of HAZUS is very large, covering the entire US, and as such contains some simplifications in order to achieve general applicability (Whitman et al., 1997). One significant advantage of the HAZUS methodology is that both the method and the underlying assumptions are fully documented, unlike most commercial and proprietary software, which tends to comprise ‘black boxes’ with little documentation provided. There are a number of published technical papers describing the theory and development of the methodology (e.g. Whitman et al., 1997; Kircher et al., 1997a,b) and many more presenting case studies of its application (e.g. Tantala et al., 2002; Bouabid et al., 2002). Furthermore, because of the aforementioned transparency, many studies carried out outside the US have adopted a HAZUS-based approach (e.g. Bommer et al., 2002a; Pappin et al. 2004; Yeh & Loh, 2001). These are the reasons for the extensive reference made to HAZUS within this thesis.
Figure 4-4 shows a flowchart of the HAZUS methodology. Some of the areas in which HAZUS is perceived to be better than previous published studies include its treatment of additional aspects such as service outages, fire ignition and spread, and indirect economic losses. As such it contains elements that bring it much closer to being truly regional and comprehensive. The most recent release of HAZUS was HAZUS-MH (multi-hazards), completed in 2003. This is the version to which reference is made in this thesis, although it is worth noting that the significant components discussed herein have not changed since the first release in 1999.

HAZUS is a complex and powerful technology (Khater et al., 2003) and moreover is provided free of charge, thus filling a significant void in the field of loss estimations. The developers of HAZUS acknowledge that the “approximations and simplifications that are necessary for comprehensive analyses” (FEMA, 2003) will produce significant uncertainties in the results, of “at best” a factor of two or greater.

4.1.2 DBELA

The DBELA (Displacement-Based Earthquake Loss Estimation) methodology is a probabilistic displacement-based vulnerability assessment procedure, currently under development by researchers at the ROSE School, University of Pavia, Italy, in collaboration with Dr J. Bommer,
Chapter 4 Earthquake Loss Estimation Methods

Imperial College London, and others. The principles underlying DBELA are presented in Section 4.8. An extension of the methodology to estimate damage caused by ground deformations has been undertaken as part of this research, see Chapter 8.

4.1.3 Commercial software

The three main developers of commercial loss estimation software are EQECAT, RMS (Risk Management Solutions Inc), and AIR (Applied Insurance Research). Development in this field is strongly led by the insurance and reinsurance industry. As noted previously, there is only limited information in the public domain as to how these packages calculate seismic hazard and building damage levels.

4.2 Seismic hazard

The primary input to earthquake loss estimation is the definition of the seismic hazard, in terms of the size, type and location of the earthquake, the behaviour of waves as they travels through the Earth to the study area, and how the earthquake effects are translated into a direct hazard at a site. The built environment is at risk from a number of hazards directly caused by earthquakes; these hazards and their assessment have been described in some detail in Chapter 2. Each of these can potentially cause damage to the main inventory components of buildings, transportation lifelines and utilities lifelines, leading to both direct and indirect losses.

4.2.1 Forecasting model

The choice between deterministic earthquake scenarios and uniform hazard levels defined through probabilistic seismic hazard assessment depend largely upon the purposes of the loss estimation model.

Probable Maximum Loss (PML) has been traditionally used as an indicator to insurance companies or building owners of their financial exposure and to enable them to manage their long-term risks. PML is calculated on the basis of a single deterministic scenario, or a series of scenarios, which are typically either repetitions of historic events (as in the example in Figure 4-3) or the expected worst case scenario that could affect the exposed region. The calculated losses will clearly be a single outcome or a series of outcomes for that region. Annualised average losses (AAL), or expected annualised losses (EAL) are the aggregate losses of all possible combinations of earthquake magnitude and distance affecting the exposed region, and are compatible with probabilistically determined hazard levels. For example, in California,
over 80% of the average annual losses are due to frequent, low magnitude, events (Windeler et al., 2004).

For a probabilistically derived hazard, if damage calculations are based upon spectral response, scenario earthquakes obtained either through disaggregation, or through the use of synthetic catalogues (Section 2.6.4), are essential. The magnitude and distance of the scenario earthquakes will not only influence the shape of the elastic response spectra (e.g. Figure 2-5), but also two important variables in displacement-based design; the period beyond which spectral displacements are constant, and the scaling factor for damping values other than 5% (Bommer & Mendis, 2004).

There is a current trend towards the use of stochastically simulated earthquake catalogues for loss estimations (e.g. Zolfaghari, 2004; Trenafiloski & Milutinovic, 2004), since these satisfy requirements both for defined probabilities of exceedance, and realistic demand parameters for determining the response of engineered structures. The individual earthquake scenarios contributing to each point on the aggregate curve of loss versus probability of exceedance can also be identified. The potential downside of this approach is the additional requirements in computing time, particularly if all uncertainties are to be fully incorporated; however, this is unlikely to present a significant constraint in the near future.

HAZUS (FEMA, 2003) was developed primarily with a view to using deterministic scenarios, although probabilistic hazard maps for the United States may also be used as input. There is no reason why simulated events could not be used with the HAZUS methodology.

A very general conclusion from published literature would be that earthquake loss estimations use deterministic models where they have been developed for social or infrastructure planning, for use by public bodies and for vulnerability assessment and risk mitigation purposes (e.g. Escuela Politécnica Nacional, 1994; Cardona & Yamin, 1997; Ménéroud et al., 2000; Siegel et al., 2002). For financial planning, risk management and risk transfer and insurance purposes, probabilistic hazard models with stochastically simulated events (e.g. Andrea, 2004; Zolfaghari, 2004; RMS; EQECAT) or multiple deterministic scenarios with assigned frequencies of occurrence (e.g. Bommer et al., 2002a; Grossi, 2000) are more common.

4.2.2 Earthquake intensity measure

The magnitude of the ground shaking may be represented either by macroseismic intensity scales or by strong-motion parameters, i.e. acceleration, velocity or displacement, represented either by a single peak value (i.e. PGA) or by response spectra.
Some of the perceived advantages and disadvantages of these various options for application to loss estimations have been discussed in Section 2.4. Despite the shortcomings related to the use of macroseismic intensities, their use is still common in published loss estimations. Loss estimation models for New Zealand, for example, have used MMI due to the large body of empirical MMI-based data for the prediction of ground-motions and for correlation of observed and predicted damage (e.g. Dowrick & Rhoades, 1999, 2002). Certainly one of the strongest arguments in favour of the use of MMI is its close relationship to observed damage, and the large body of data for which MMI can be determined, compared to very limited survey data having instrumentally measured strong-motion parameters. The noted disadvantage that macroseismic intensities are discrete, non-uniform indices and are therefore not suited to prediction through regression-based attenuation relationships is dealt with in some models (e.g. Onur et al., 2002 for British Columbia) using appropriate strong-motion prediction relationships (Section 2.5) or existing hazard maps in terms of PGA, and subsequently converting these to MMI using empirical conversions such as those presented by Trifunac & Brady (1975). However, this approximate procedure will significantly increase the uncertainty level.

PGA as a parameter for loss estimation models has no significant advantage over MMI, other than the fact that there is a larger number of predictive relationships, and many published hazard maps are defined in terms of PGA. Nonetheless, it has a very poor relationship with structural damage.

![Image](image_url)

**Figure 4-5:** Example of hazard definition used in HAZUS (FEMA, 2003). Smoothed spectra are obtained from the spectral accelerations at periods of 0.3 and 1.0s, plotted against spectral displacement, with the radial lines showing constant period. Site class definitions are after FEMA (1997). WUS = Western United States

Current state-of-the-art methods use spectral displacements, having the closest relationship with the deformations and hence damage levels in a building (e.g. AIR, 2000; Crowley et al., 2004a;
Kircher et al. 1997a; Ordaz et al., 2000). HAZUS (FEMA, 2003) defines the hazard in terms of acceleration versus displacement, where the two are related through the pseudo-spectral relationships given in Equation 2-3, as shown in Figure 4-5. Other methodologies e.g. DBELA, employ displacement spectra plotted against period (Figure 4-6); the only difference between the two is in terms of presentation, where Figure 4-5 can be directly related to the capacity spectrum approach used in HAZUS to evaluate damage levels. Ordaz et al. (2000) define the hazard in terms of acceleration spectra, but subsequently incorporate Equation 2-3 into their building damage estimation, and therefore the difference is again in terms of presentation only.

Strong-motion parameters such as spectral accelerations and spectral displacements, estimated using ground-motion prediction relationships as described in Section 2.5, are defined in terms of both their median value and their standard deviation, thus allowing the uncertainty associated with demand definition to be explicitly included in the model.

![Figure 4-6: Median displacement spectra used to define ground shaking hazard in DBELA (Crowley et al., 2004b)](image)

**4.2.3 Site effects**

Where hazard is defined in terms of macroseismic intensity, there are limited options available for incorporation of local soil effects. Estimated intensities can be modified in an approximate manner for different soil conditions, as explained in Section 2.7.1 (e.g. Jones et al., 1995) or by predicting site dependent PGA values that are subsequently converted to intensities. Either approach has a high associated uncertainty. An alternative approach is to leave the demand unmodified, but to modify the expected performance of the building according to the expected amplification, landslide or liquefaction hazard, as done by McCormack and Rad (1997). This
approach is preferable in that it is more transparent and provides users with a clearer picture of how these effects contribute to the overall damage.

For hazard defined in terms of response spectra, the NEHRP site classifications and amplification factors, presented in Tables 2-2 to 2-4 are well suited to the requirements of a regional loss estimation. The classifications are clearly defined and allow consistent classifications across large areas. The amplification factors can be applied to any demand defined in terms of bedrock spectral accelerations, and their use is amenable to large databases or GIS models. For the United States, site zoning maps using the NEHRP classifications are widely available, and where they are available, the incorporation of site amplification effects is very straightforward. HAZUS employs the NEHRP system, where the short- and long-period amplification factors correspond to the standardised spectral shape shown in Figure 4-5, as illustrated in Figure 4-7.

![Figure 4-7: Amplification of bedrock spectra due to local site conditions in HAZUS (FEMA, 2003)](image)

Classification schemes other than NEHRP may be used where there is sufficient knowledge of local geology to do so. For example, the HAZUS-based study for South Carolina used five site categories divided according to the amplification characteristics of the different surficial deposits, and amplification factors were computed by site response analysis for a range of profiles and ground motions (Silva et al., 2002). The study for Hong Kong discussed in Chapter 6 adopted the NEHRP classification system, but developed new, region-specific amplification factors, from typical soil profiles and site response analyses.

The basis for soil zoning is very dependent upon the size of the study area and the availability of geotechnical data. The preferred approach is to use in situ measurements, either of penetration
resistance or shear wave velocities, from a distributed database of boreholes, but this is clearly very demanding in terms of time and resources. For example, the site classification map for Hong Kong used over 1200 borehole logs in conjunction with detailed geological maps (Pappin et al., 2004). Hong Kong is a relatively small territory, and due to its highly built up nature, the ground conditions are remarkably well investigated. Furthermore, the correlation between the site classifications and the near-surface geology assisted in the zonation procedure. In many other regions, such a detailed procedure may not be possible, and studies have to rely more heavily on published geological data. This was the case in the loss model for the whole of Turkey carried out for the Turkish Government (Bommer et al., 2002a).

A sensitivity study has been carried out to explore the influence of various epistemic uncertainties within a loss estimation model on the predicted damage distributions, full details of which are presented by Crowley et al. (2004b). Geological criteria were used to develop the base model for site classifications, in the Marmara region of Turkey, assuming that:

- Holocene (the youngest Quaternary) deposits and other Quaternary deposits in river valleys and coastal areas are site class E.
- All other Quaternary deposits are site class D.
- Pre-Quaternary formations in mountainous regions are site class B.
- All other pre-Quaternary deposits are site class C, on the basis that some soil cover is likely to overly the rock in non-mountainous areas.

This very crude assumption is illustrative of the type of decisions that must be made when there is no scope for detailed geotechnical investigations. Nonetheless, it was found that adjusting these assumptions could lead to fairly significant changes in the damage estimations, thus confirming that investment in data retrieval for built up areas in order to confirm and refine site classifications is worthwhile to improve a loss model (Crowley et al., 2004b).

The same study also considered the influence of using the NEHRP amplification factors compared to directly estimating surface response spectra from the ground-motion prediction relationships. In this particular case it was shown that the change had only a small influence on the results, although it was also noted that this conclusion would not necessarily hold for stronger levels of shaking than were considered, or for longer building periods (this is illustrated in Figure 2-7). The study of site effects for Hong Kong (Chapter 6; Pappin et al., 2004) clearly illustrated two important issues: the difference between the NEHRP factors and those calculated specifically for the seismicity and soil conditions in the region; and the large scatter within
computed amplification factors, which is not represented by the use of the NEHRP factors. These are uncertainties that need to be given greater consideration in future models, as soil amplification effects can be as significant as the bedrock seismic hazard model.

Microtremor data, i.e. measurements of weak motions such as ambient noise (e.g. wind, waves, traffic vibration) or small magnitude/large distance earthquakes at the ground’s surface are an increasingly popular means of characterising site effects on a regional basis. The reason for their popularity is that measurements can be undertaken using portable instruments and therefore large geographical areas can be covered at a relatively low cost in comparison to drilling boreholes for sampling and testing or for geophysical testing. Two common methods of interpreting microtremor measurements are; the standard spectral ratio (SSR), a ratio of the measured Fourier spectra at the ground surface of a soil site to that at a nearby reference rock site; and the horizontal to vertical spectral ratio (HVSR) for a single recording at a site (commonly referred to as the “Nakamura method” after Nakamura, 1989). The latter method is more adaptable because it does not require a reference site, and has been found to give a good indication of a site’s natural period (e.g. Martirosyan et al., 2002). In some cases the HVSR is suggested to provide an estimation of the spectral amplification factors of a site (Tromans, 2004), although this method is far from reliable. Notwithstanding this unreliability, microtremors, particularly when correlated to geotechnical and geological data, provide a useful tool for the classification of near-surface soils over large areas.

Spectral Analysis of Surface Waves (SASW) is another non-invasive, and therefore less costly than boreholes, method to estimate the shear wave velocity profiles of near-surface deposits, and thus assist in site classification for regional hazard studies. Brown et al. (2002) showed good agreement between $V_s$ profiles obtained in this manner and those obtained from downhole seismic testing, to a depth of about 50m. Even though this technique lacks physical sampling to confirm the nature of the soils, it will provide a suitable $V_{S\,30}$ for regional site classification according to NEHRP.

4.2.4 Ground failure hazard

Figure 4-8 shows the requirements of regional assessments of seismic hazard and risk in terms of ground failure. Ultimately, for any earthquake scenario, the expected displacement of the various susceptible soil units can be estimated.
Current methods and published loss estimations vary significantly in their approach for dealing with ground failure. However, it appears that in many cases where liquefaction or landslides are not incorporated, this is not based upon careful decision-making, but rather on the basis that these components are too complex. A large number of studies reviewed as background to this thesis make no mention of ground failure hazards (e.g. Schwarz et al., 2004; Dhu et al., 2004; Chrysostomou et al., 2004). Others mention the definition of the hazard with no details of how, or even if, this is translated into building damage (e.g. Windeler et al., 2004). In some cases this may be appropriate, as, unlike ground shaking and soil amplification, ground failure hazard is not always an essential component of a loss estimation. In the light of the additional data and resources required to incorporate it, careful decision-making should be carried out in order to balance the requirements of the model, the regional topography and geology, and the extra costs related to incorporating ground failure-induced losses. The findings of Chapter 3 will support this decision-making process. Notwithstanding the above comments, such decisions should be based upon the requirements and considerations of the model in question, not upon the lack of suitable methodologies. If ground failure is to be neglected from a model, there must be a rational justification presented for doing so.

The hazard due to liquefaction and landslide may be defined either in terms of the relative likelihood or probability of failure, with the damage modified accordingly, or, in more detailed calculations, by defining both the probability of failure and the expected permanent ground deformation. Methods for the classification of ground failure hazard are plentiful, and liquefaction and landslide hazard maps, identifying zones that are susceptible to these hazards
have been produced for many regions (Broughton et al., 2001; Power et al., 1991). The larger the volume of data available, the more sophisticated the techniques that can be used to generate such maps (e.g. Siegel et al., 2002), and the higher the degree of quantitative information that can be included. A recent publication by Rodriguez-Marek et al. (2001) presented an approach for the assessment of the probability of liquefaction over spatially distributed areas. This is done by using probabilistic relationships for liquefaction potential (see Section 2.9.3) combined with stochastic treatment of the probability density functions of the random variables such as $(N_1)_60$ and CSR. Despite its computational complexity, this approach is suited to regional damage estimations, where the uncertainty related to the hazard is a fundamentally important parameter.

Regardless of the level of complexity used to define the hazard, it is the evaluation of the demand that this places on the built environment, and the associated levels of damage, that are important for the final results of a regional damage study. These elements, discussed in Sections 4.9 to 4.11, are much more poorly defined (particularly the resulting damage). Few liquefaction hazard maps for example contain sufficient information to determine the thickness of the liquefied layer, yet in terms of building response, this is an important variable. In keeping with the "principle of consistent crudeness" (Elms, 1985) whereby the reliability of results will be no greater than the reliability of the most uncertain element, it is debatable whether there is presently any advantage to be gained from the use of sophisticated techniques for mapping ground failure hazard.

4.3 Inventory

The collation, storage and processing of the inventory databases and their level of detail are of great significance to the accuracy of loss estimations. To develop a comprehensive database can be very expensive and time consuming; FEMA (2001) suggest that up to two years may be required to compile a detailed loss estimation inventory of a new study area for a new HAZUS model. The level of detail of an inventory depends upon the requirements of the loss estimation study. Direct economic losses will be caused primarily by damage to the general building stock, whereas emergency response planning is also influenced by the disruption to lifelines. Knowledge of the relative importance of different factors therefore enables database collation to focus on the critical areas.

A complete inventory should cover the following elements;

**General building stock.** This will include residential, commercial and industrial buildings.
Figure 4-9: An example of inventory data, broken down according to age, use and construction type for the damage city of Armenia (Colombia) following the January 25, 1999 earthquake. Damage assessment is also shown, where a red-tagged building is damaged beyond repair, and a green-tagged building is safe for immediate occupation (courtesy J.Bommer).

**Lifelines.** Lifelines include all distribution networks such as power supply, water distribution, communications and transport. A transport inventory would include road and rail bridges, tunnels, and ports and airports.

**Critical Facilities.** Critical facilities such as schools, hospitals and emergency shelters may be designed to have higher seismic resistance than general building stock. Furthermore, they generally come into a separate category due to their importance to the recovery of a region.

**High potential loss facilities.** This category includes dams, nuclear installations or industrial or military facilities with potential for release of hazardous materials. Such facilities generally merit a full site-specific risk assessment of their own, but sufficient detail of expected risk and losses due to an earthquake must be incorporated into a regional loss estimation.

**Economic and social data.** Inventories need to define the statistical distribution of given building types with each category. As well as the structural type, the occupational category (e.g. residential, commercial, light industry) of each group of buildings must be known in order to assess economic losses.
Figure 4-9: An example of inventory data, broken down according to age, use and construction type for the damage city of Armenia (Colombia) following the January 25, 1999 earthquake. Damage assessment is also shown, where a red-tagged building is damaged beyond repair, and a green-tagged building is safe for immediate occupation (courtesy J.Bommer).

**Lifelines.** Lifelines include all distribution networks such as power supply, water distribution, communications and transport. A transport inventory would include road and rail bridges, tunnels, and ports and airports.

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**Economic and social data.** Inventories need to define the statistical distribution of given building types with each category. As well as the structural type, the occupational category (e.g. residential, commercial, light industry) of each group of buildings must be known in order to assess economic losses.
An important limit with HAZUS, as with all loss models, is the resources available for the inventory compilation. Since HAZUS contains extensive default data directories, there is possibly a higher danger of users overlooking the importance of the inventory. The intended purpose of the default data is to allow a very crude analysis for a region to be carried out in a matter of weeks as a first step in assessing earthquake risk. Figure 4-10 illustrates the significant differences between the HAZUS default data and detailed inventory data for the Wall Street area of New York. For example, HAZUS assumed that 100% of the buildings were less than three storeys high, whereas in reality, 81% were more than 8 storeys. The authors of that study found that by changing HAZUS default data for both buildings and soil, the estimated losses for a $M_w$ 5.0 earthquake at approximately 20km, dropped from an estimated $59.3$ million to $4$ million (Nordensen et al., 2000). For a $M_w$ 7.0 earthquake on the other hand, the difference was a factor of less than 1.5.

![Figure 4-10: Comparison of HAZUS default inventory and defined building inventory for the Wall Street census tract, New York (Nordensen et al., 2000). It should be noted that the building stock in the Wall Street district represents an extreme example, and for a larger study area, the effect of these differences could be expected to be diluted.](image)

The classification system requires careful consideration such that it adequately represents the variability of building types in a region, whilst balancing this criterion with the need to avoid creating so many categories that the calculations become unreasonable, or data collection unrealistic. Table 4-1 shows the building classification system used for Turkey in a loss estimation model for the entire country. Not only do different structural types have to be grouped into representative categories, but variables relating to their age and quality as shown on the right hand side of Figure 4-10 have to be collated into groups. Even if it were possible to gather complete details of each and every building in a study region, thus (theoretically) reducing the epistemic uncertainty to almost zero, it would still be necessary, within the framework of a loss model, to group these buildings. The alternative would be to analyse each
individual building, which would then become a very large number of site-specific studies. This type of modelling uncertainty, may, for convenience, be modelled as random, in that it is assumed it can not be reduced within the confines of the model.

Table 4-1: Turkish building classifications and typical percentage in both urban and rural locations (after Spence et al., 2003)

<table>
<thead>
<tr>
<th>Structural Type</th>
<th>Code</th>
<th>Urban</th>
<th>Rural</th>
</tr>
</thead>
<tbody>
<tr>
<td>Timber frame</td>
<td>TW1</td>
<td>&lt;1</td>
<td>&lt;1</td>
</tr>
<tr>
<td>Weak Masonry (adobe, rubble masonry)</td>
<td>TURM1</td>
<td>7</td>
<td>9</td>
</tr>
<tr>
<td>Brick/block Unreinforced Masonry with timber floors</td>
<td>TURM2</td>
<td>19</td>
<td>26</td>
</tr>
<tr>
<td>Brick/block Unreinforced Masonry with concrete floors</td>
<td>TURM3</td>
<td>17</td>
<td>21</td>
</tr>
<tr>
<td>RC frame with masonry infill – 1 to 3 storeys, Poor Seismic Design</td>
<td>TC1 LP</td>
<td>44</td>
<td>36</td>
</tr>
<tr>
<td>RC frame with masonry infill – 4 to 7 storeys, Poor Seismic Design</td>
<td>TC1 MP</td>
<td>4</td>
<td>3</td>
</tr>
<tr>
<td>RC frame with masonry infill – 8 or more storeys, Poor Seismic Design</td>
<td>TC1 HP</td>
<td>6</td>
<td>1</td>
</tr>
<tr>
<td>RC frame with masonry infill – 1 to 3 storeys, Good Seismic Design</td>
<td>TC1 LG</td>
<td>1</td>
<td>&lt;1</td>
</tr>
<tr>
<td>RC frame with masonry infill – 4 to 7 storeys, Good Seismic Design</td>
<td>TC1 MG</td>
<td>&lt;1</td>
<td>0</td>
</tr>
<tr>
<td>RC frame with masonry infill – 8 or more storeys, Good Seismic Design</td>
<td>TC1 HG</td>
<td>&lt;1</td>
<td>0</td>
</tr>
<tr>
<td>RC shear wall – 4 to 7 storeys</td>
<td>TC2 M</td>
<td>1</td>
<td>&lt;1</td>
</tr>
<tr>
<td>RC shear wall – 8 or more storeys</td>
<td>TC2 H</td>
<td>2</td>
<td>1</td>
</tr>
<tr>
<td>Single storey industrial shed with light steel truss roof pinned to cantilever RC or steel stanchions</td>
<td>TS1</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>Single storey industrial shed with pre-cast concrete roof</td>
<td>TS2</td>
<td>0</td>
<td>0</td>
</tr>
</tbody>
</table>

Crowley et al. (2004b) explored the relative impact of building height classifications to estimated damage distributions, and found their results to be relatively insensitive to grouping buildings from one to nine storeys high into three ranges, represented by the capacity of the median height (i.e. 1 to 3 storeys represented by a 2 storey building), rather than nine individual categories. Importantly, this simplification needs to consider the distribution of building heights within the inventory data; Crowley et al. showed that the behaviour of buildings between 1 and 9 storeys could be approximately represented by a single category of 3 storeys, being the mean value of the actual building stock used for their study, whereas 5 storeys, the median, was found to be a poorer representation of the more detailed results. Furthermore, the shape of the demand curve is relevant; the demand representation used by Crowley et al. (2004b) is shown in Figure
4-6, and is linear over the relevant period range, as is the capacity of the buildings, defined in terms of their height, which allows a single category to approximately represent a larger range of storey heights. Should the constant displacement plateau in the demand spectrum occur within the relevant period range for the building stock, these simplifications would be less effective.

A further important variable in the collation of an inventory database for an earthquake loss estimation is the size of the area used. Studies are generally divided into zones, referred to, at least in a GIS context, as georefs. The seismic hazard is determined at the centre of each georef, and damage estimations carried out for the assumed distribution of inventory data within this area. Such divisions need to be relatively homogeneous in terms of the available data. Where a high level of data is available, as in urban areas, for modelling convenience, calculations may be carried out on a 100 m or 500 m grid, assuming uniform soils and a single building distribution within each grid square. For less well-defined data or lower requirements in terms of accuracy, larger zones may be used, which are more likely to be irregularly-sized and -shaped due to being based upon district or municipality boundaries. Insurance studies frequently use CRESTA (Catastrophe Risk Evaluating and Standardising Target Accumulations) zones, whose size will vary greatly depending on the country for which they are defined, but can be very large; for example, El Salvador is divided into only two CRESTA zones (www.cresta.org).

### 4.4 Damage definitions

Damage to buildings or infrastructure can be quantified on a continuous scale from zero (no damage) to one (collapse) where values relate to the ratio of damage repair to total rebuilding costs. For expedience, damage levels are grouped into damage states (DS) reflecting the ranges of damage levels buildings can experience. The number of damage states defined depends upon the needs of the study (Jones et al., 1995). Typical damage states are defined as slight, moderate, severe and complete (e.g. FEMA, 2003), which can then be qualitatively described. A descriptive definition of the damage state is important so that from the end results there can be an understanding of the nature and extent of the predicted damage (Whitman et al., 1997). Descriptions of structural damage states of RC frame buildings are shown in Table 4-2.

Other commonly used damage definitions include:

\[
\text{Damage Factor (DF) or Damage Ratio (D_R) } = \frac{\text{cost of repair}}{\text{total building value}} \quad (4-1)
\]
Mean Damage Factor (MDF) = \[ \frac{\sum_{i=1}^{N} (DF_i)}{N} \] \hspace{1cm} (4-2)

where \( N \) is the number of buildings. For regional studies, the Loss Ratio (LR) is also used, where:

\[
\text{Loss Ratio (LR)} = \frac{\text{Total Loss}}{\text{Total Exposure}} \hspace{1cm} (4-3)
\]

where Total Exposure is the value of all building stock in the region. Insurance companies generally consider their 'effective' loss ratio (ELR), where the total loss in Equation 4-3 is replaced by the total insured loss.

**Table 4-2:** Structural damage state descriptions for RC frame buildings (adapted from HAZUS, FEMA, 2003 by Crowley et al., 2004a)

<table>
<thead>
<tr>
<th>Structural Damage Band</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>None to Slight</td>
<td>Linear elastic response, flexural or shear type hairline cracks (&lt;1.0mm) in some members, no yielding in any critical section.</td>
</tr>
<tr>
<td>Moderate</td>
<td>Member flexural strengths achieved, limited ductility developed, crack widths reach 1.0mm, initiation of concrete spalling.</td>
</tr>
<tr>
<td>Extensive</td>
<td>Significant repair required to building, wide flexural or shear cracks, buckling of longitudinal reinforcement may occur.</td>
</tr>
<tr>
<td>Complete</td>
<td>Repair of building not feasible either physically or economically, demolition after earthquake required, could be due to shear failure of vertical elements or excess displacement</td>
</tr>
</tbody>
</table>

HAZUS, and HAZUS-based approaches use the *mean damage ratio* (MDR) to define the estimated losses by a single value, where:

\[
MDR = 0.02 \cdot P(DS = \text{slight}) + 0.1 \cdot P(DS = \text{moderate}) + 0.5 \cdot P(DS = \text{extensive}) + 1.0 \cdot P(DS = \text{complete}) \hspace{1cm} (4-4)
\]

An important observation relating to damage state descriptions that is considered in the following chapters is their inadequacy in terms of describing damage that is frequently seen in zones of soil liquefaction (described in Section 3.4.2.1). The descriptions in Table 4-2, in common with those used in HAZUS, Coburn and Spence (2002), Grünthal (1998), and others, focus on typical structural response to ground shaking, in terms of crack width, and collapse of walls, floors or entire buildings. They do not reflect the response of buildings on stiff
foundations to uniform or differential ground movements, even though, as noted in Chapter 3, such buildings must still be considered damaged in terms of their loss of functionality.

This study focuses on damage states rather than costs of damage. Loss estimations must be able to produce reliable estimations of damage distributions in order to get correct costs. The author believes that directly presenting results in financial terms, as is frequently done, camouflages an important element of the results, and adds additional uncertainties before those of the previous stage have been resolved. Repair costs, for example, are dependent upon local construction methods, so by focusing on damage states, some regional dependencies may be removed.

4.5 Options for the definition of vulnerability

The definition of a relationship between the ground motion and the damage state is probably the biggest variable between different loss estimation approaches. Vulnerability is defined as "the degree of loss to a given element at risk resulting from the occurrence of a specified earthquake" (Coburn & Spence, 2002). The requirement of vulnerability models in loss estimations is to determine the range of behaviour expected for a given building classification subject to a defined level of hazard.

The three generic options available for the definition of vulnerability are discussed in the following sub-sections. These three methods should converge to some extent in the future, as more good quality loss data become available following future earthquakes, analytical tools improve and so does the ability to analyse non-engineered structures, and expert judgement will reflect these improvements.

4.5.1 Empirical

Where there is sufficient existing loss data available, users can take empirical data of damage and losses from previous earthquakes, and develop relationships between the ground motion and the degree of damage for building types in the region. For the most common building categories in regions with a history of seismic activity, this approach is increasingly relevant as more data become available (Coburn & Spence, 2002). One problem inherent with the approach is that following damaging earthquakes, structures suffering extensive damage or collapse are unlikely to be rebuilt in exactly the same manner. This is part of the process of improving earthquake resistant construction and learning from earthquakes. The building types that performed badly in the earthquake should therefore be rendered obsolete in the rebuilding process and available damage data is of limited use. However, it is frequently shown that vulnerable structures, which have been observed to perform poorly in past earthquakes, still
exist in many earthquake prone regions, and so the assumption that damage data is rendered obsolete by the fact that it exists is somewhat idealised.

Detailed loss or damage data from earthquakes in recent years are very limited (see Chapter 3), and the older the data, the less relevant they are in terms of current building practices and economics. Many regions for which loss estimates are carried out have not suffered damaging earthquakes in recent times; Hong Kong (Chapter 6) is an example of a region for which there is no available data, and where the seismicity and the nature of the building stock render comparisons with other regions very difficult. Even where extensive data are available such as in the Western USA and Japan, they can be of very variable quality, or incomplete. Insurance data are of limited use as they rarely provide any breakdown of losses or structural details, although there is probably potential to make better use of insurance loss data than is currently done (Shephard, 1997). Furthermore, use of empirical data cannot provide guidance on the cost benefits of undertaking improvements or retrofitting, unless these improvements have also been tested by past earthquakes.

Despite its limitations, the use of existing loss data is the only method available to calibrate loss models, and there are therefore many loss estimations that are essentially uncalibrated.

4.5.2 Analytical tools

Where the engineering response of a building type to earthquake loading is well understood, it is possible to predict the structural response to ground motions, and hence the damage level, using analytical tools. Relationships can be established between parameters such as inter-storey drift and an expected degree of damage. The HAZUS and DBELA approaches are described in Sections 4.7 and 4.8.

Pushover analysis has become increasingly common as a tool for the performance evaluation of buildings subjected to seismic loading. Pushover curves represent the force-displacement behaviour of a non-linear, single degree of freedom structure with degrading stiffness as the load monotonically increases, thus representing the response of a structure in a pseudo-static manner. One of the significant advantages of pushover analysis is that it is efficient and easy to use, in comparison to full dynamic time history analysis, and provides some important structural response information (Antoniou & Pinho, 2004). In this respect it is suited to the needs of regional damage estimations for simple procedures using accessible input data. However, disadvantages noted by Antoniou and Pinho (2004) include the fact that higher modes of response are generally ignored, as is the change in period and modal characteristics as a
structure degrades. Therefore, pushover analysis cannot be considered as an appropriate replacement for non-linear dynamic analysis in terms of accuracy.

4.5.3 Expert judgement

The use of expert opinion to develop vulnerability relationships and damage distributions is a useful technique. ATC-13 (ATC, 1985) makes extensive use of expert opinion, and represents "one of the most thorough, best-documented efforts to compile seismic vulnerability functions for an exhaustive set of structure categories" (Porter, 2003). Expert judgement is most appropriate to structures whose engineering performance is reasonably well understood, but for which there is limited damage data (Coburn & Spence, 2002). The performance of non-engineered structures, such as unreinforced masonry buildings, is difficult to predict analytically, and complete, good quality loss data is rarely available for such structures. In cases such as this, expert judgement may be the only available resource. Although the use of expert judgement may appear to be the most uncertain approach of the three listed, due to its subjectivity and lack of analytical support, there is an argument that the use of a single analytical model, which is, after all, selected using judgement, could have even greater uncertainties. These are compounded by the fact that most users would not see the need to consider the uncertainty (e.g. conservatism) associated with their choice of modelling approach.

The evaluation of other components of a loss estimation model, such as the loss of functionality of a building and the time required to restore functionality, is complex. To the author's knowledge, this has only ever been done by expert judgement. HAZUS relies on ATC-13 in many areas where there are insufficient data, knowledge or resources to improve upon it, including restoration times for transportation and utility lifelines, pipeline damage functions and even casualty rates, as well as much of the economic data such as the cost ratios used to evaluate overall direct economic losses. This is in spite of the fact that ATC-13 was developed 20 years ago for Californian conditions and therefore classification systems and damage estimations are for the particular infrastructure of California, which is generally designed for earthquake loading. However, this has not precluded its use as the framework for a number of studies outside the United States, such as the Quito Project (Escuela Politécnica Nacional et al., 1994) and a loss estimation study for Bogotá, Colombia (Cardona & Yamin, 1997).

Expert judgement can be a highly variable commodity, strongly dependent upon the confidence, experience, knowledge and motivation of the expert in question. The definition of 'expertise' is the subject of much interesting debate. The product required from expert judgement is decision-making, in this case regarding building vulnerability and resistance to
earthquakes, because of the uncertainty associated with lack of data. As pointed out by Vick (2002) "If it weren't for uncertainty, ... decisions would follow automatically from prescriptive rules and expertise would hardly be required." Biases in expert judgement arise from a wide range of sources, including motivation and pre-conceptions, and in full elicitation of expert judgement, formulaic measures may be put in place in order to eliminate, or compensate for, such biases. A further disadvantage of the use of expert opinion is that it is not 'testable'.

4.6 Damage probability matrices

ATC-13 (1985) was one of the first attempts to comprehensively develop earthquake damage evaluation data and loss estimation for industrial, commercial and residential properties, utilities and transportation networks, in California. Some of the important developments in ATC-13 were the combination of damage due to ground shaking and ground failure, the development of a complete structural inventory for California, and the use of extensive expert opinion, in a controlled manner, to develop damage probability matrices (DPMs), such as that shown in Table 4-3 for each building classification.

Table 4-3: Example damage probability matrix (DPM) from ATC-13 for reinforced concrete shear wall structures with moment resisting frames

<table>
<thead>
<tr>
<th>Damage Level</th>
<th>Modified Mercalli Intensity I % of buildings at each damage level</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>VI</td>
</tr>
<tr>
<td>None</td>
<td>19.1</td>
</tr>
<tr>
<td>Slight</td>
<td>62.9</td>
</tr>
<tr>
<td>Light</td>
<td>18.0</td>
</tr>
<tr>
<td>Moderate</td>
<td>***</td>
</tr>
<tr>
<td>Heavy</td>
<td>***</td>
</tr>
<tr>
<td>Major</td>
<td>***</td>
</tr>
</tbody>
</table>

ATC-13 made extensive use of expert engineering judgement; consulting over 70 experts in the field of earthquake engineering representing "more than a thousand man-years of professional experience in earthquake engineering" (ATC, 1985). A three-round procedure was used to establish experts' mean, low and high damage estimations for each inventory classification and each level of shaking (defined in terms of MMI). This approach was probably more rigorous than anything that had been done before, but one of the acknowledged shortcomings of the study was the insufficient observational data with which to correlate the damage estimations (ATC, 1985). Of a total of 70 structural and lifeline classifications used in ATC-13, useful
observed damage information was available for only six structural types, hence use of expert opinion was almost certainly the best option available.

One of the undoubted appeals of DPMs is their straightforwardness. A direct relationship between the level of shaking and the type of building precludes the need to define a large number of structural parameters, which can be very difficult to evaluate over large study areas (Booth et al., 2004).

4.7 The capacity spectrum approach

Figures 4-11 and 4-12 show the process used in HAZUS (FEMA, 2003; Kircher et al., 1997). This approach, using inelastic displacement-based capacity curves and fragility curves to define the proportion of each building type in each damage state, related to the interstorey drift, is recognised as being a significant improvement upon previous loss estimation techniques. The demand is specified as a continuous response spectrum (Section 4.2.2). Capacity is defined as a function of yield strength, stiffness, and ductility (Figure 4-11), therefore the structural response to ground shaking is obtained in an analytical manner, and the vulnerability parameters can be changed to reflect local building behaviour, unlike DPMs.

The essence of the methodology illustrated in Figure 4-12, is that building damage states are a function of the maximum inter-storey drift. The amount of drift can be determined by comparing the displacement capacity; presented as acceleration (proportional to force) versus displacement from a pushover analysis, and the demand; which is defined in terms of acceleration versus displacement. This definition of the capacity, which is a function of the yield strength (allowing for overstrength, redundancy, code conservatism etc.) and the ultimate strength, is referred to as the capacity curve. The parameters shown in Figure 4-11 are tabulated in the HAZUS methodology for all combinations of building classification, height, age and seismic design level. The displacement at which the demand curve (modified for inelastic behaviour) and capacity curves intersect, referred to as the ‘performance point’, is then used to obtain a distribution of expected damage from fragility curves defined in the methodology. These fragility curves define the probability of a particular building exceeding a given damage state. They are lognormal cumulative distributions, and their ‘spread’, represented by the lognormal standard deviation of the curves, is a function of the combined variability in the demand and capacity. The fragility curves in HAZUS are partly empirically based and partly developed from expert opinion (Porter, 2003).
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Figure 4-11: Building capacity curve used in the HAZUS methodology (Kircher et al. 1997a)

Figure 4-12: The building damage estimation process used in HAZUS (FEMA, 2003). The upper diagram shows the capacity curve and the demand curve super-imposed to obtain the performance point, and the lower diagrams illustrate how the probability of each damage state (P[D|S]) is obtained.

In Figure 4-12 above, three sets of fragility curves are presented for one building type, to represent the main structural components, non-structural components that are sensitive to displacement (e.g. partition walls, windows, cladding) and non-structural components that are
sensitive to accelerations (e.g. mechanical and electrical systems). A total of almost 780 fragility curves are presented in HAZUS, which illustrates the level of detail and complexity of damage vulnerability estimation.

4.7.1 Validation of a HAZUS-based model for Turkey

A study by Spence et al. (2003) (in which this author collaborated) compared observed damage to estimated damage obtained using a model developed for Turkey, and based upon the HAZUS methodology, modified to Turkish conditions (Bommer et al., 2002a). The observed damage data comprised building-by-building surveys over small zones in the cities of Gölcük and Izmit, both heavily damaged by the 17 August 1999 earthquake. The study revealed significant over-prediction of damage levels, by 60 to 100% of the MDR (Equation 4-4). The observed damage recorded many more buildings with slight to moderate damage, and fewer extensive or complete damage states compared to the predictions. Some sensitivity studies were carried out, and it was tentatively concluded that the main contributing factors to the noted discrepancies were (i) the definition of the demand curve and (ii) the shape of the capacity curve, which failed to represent the real behaviour of the buildings. Fairly large changes of ± 25 to 50% to the ultimate capacity, the assumed drift ratios for different damage states, or the damping coefficient led to only very small changes in the predicted MDR (less than 7.5%). However, a set of model parameters selected to represent the capacity curve of a shear wall structure showed a 38% reduction in the overall damage, a significant change which meant that the results were much closer to the observations. It was observed in the field investigations after the Kocaeli earthquake that the infill walls, although not designed as shear walls, did in fact contribute to the lateral resistance, particularly in the case of the mid-rise concrete frame buildings, and vulnerability parameters that ignored this contribution would be unrealistic.

4.7.2 Steepness and spread of fragility curves

The fragility curves defined in HAZUS all produce similar shaped damage distributions. These distributions are modal in nature, in that they have a peak and a spread either side of the peak. If the peak coincides with the lower or higher end of the damage scale then only one half of such a distribution will be seen. This general shape can be observed in typical fragility curves used for capacity spectrum approaches (Figure 4-13) and also in the damage probability matrices used in intensity-based methods such as ATC-13.
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Figure 4-13: Illustration of shapes of damage distributions obtained using HAZUS-based fragility curves for mid-rise RC frame buildings with infill walls. Charts on right are for different levels of demand, in terms of spectral displacement (Sd) in metres.

The reason that the capacity spectrum approach as used in HAZUS, predicts such damage distributions is that they are based upon the ‘performance point’, without taking account of how close that point is to brittle collapse. The spread of the fragility curves is obtained by assuming that variability of the displacements is log-normally distributed based on the assumption that both demand and capacity are either normally or log-normally distributed. Although the latter assumption may be reasonable, in a highly non-linear brittle system, where brittle failure may occur very rapidly after the onset of structural damage, the assumption that response is also log-normally distributed is unlikely to apply. This would be the case for many poorly confined European reinforced concrete buildings (see Figures 4-14 and 4-15), and may also occur due to the contribution to resistance of the masonry infill walls mentioned in the previous sub-section.

Figure 4-14: ‘Brittle’ behaviour capacity spectrum for poorly confined European reinforced concrete buildings (Booth et al., 2004).
4.8 DBELA

Despite the significant progress it represents in terms of estimating building damage using analytically based approaches, there are shortcomings associated with the capacity spectrum approach described in the previous section, due primarily to the fact that real structural behaviour can differ significantly from that represented by simplified pushover curves (Crowley et al., 2004a). Furthermore, despite the apparent computational simplicity of the procedure shown in Figure 4-12, this is only the case if the capacity curves and fragility curves are already defined, as they are for the 36 building categories for the United States used in HAZUS. In order to use the same approach for other regions or other building categories, users must employ a combination of analysis and judgement to obtain the necessary parameters.

The DBELA approach presents a solution to a number of the aforementioned issues. This procedure was initially proposed by Pinho et al. (2002) and subsequently developed in a deterministic framework by Glaister and Pinho (2003). Crowley et al. (2004a) refined the approach for reinforced concrete moment resisting frames and extended it into a fully probabilistic framework.

The central tenet of the DBELA methodology is that a relationship between displacement capacity and period can be used to define limit states for any building class, with similar material and geometric properties, and this can then be directly related to the ground motion demand in terms of a displacement response spectrum. The capacity is defined using the principles of displacement-based assessment, combined with semi-empirical and semi-
mechanical formulations for the capacity of a frame to resist lateral drift. The uncertainty in both demand and capacity are incorporated within a fully probabilistic framework such that the final damage state definition includes all the uncertainty related to the building period, building displacement capacity and ground motion demand. Ongoing research related to DBELA includes the development of solutions for ground failure-related demand, presented in Chapter 8 of this thesis, and exploration of the sensitivity of damage estimations to epistemic uncertainties (Crowley et al., 2004b) and to the representation of seismic hazard.

The principal advantages of DBELA over HAZUS are:

i. The capacity is defined using mechanical principles, and the capacity functions are therefore more consistent with the behaviour of real structures (Crowley et al., 2004a).

ii. Each parameter that defines the displacement capacity is treated in a probabilistic manner, allowing the uncertainty to be directly quantified with respect to the available data from which to define the building stock.

The methodology is still in the development stages, and therefore does not as yet constitute a complete alternative to existing methods. Only RC frame buildings are presently considered.

4.9 Damage due to ground failure

As noted previously, many published methods do not include ground failure-induced damage, for reasons that are not always clearly explained. Of those that do, other than those which follow the HAZUS-based approach, discussed in the next section, most tend to use some form of weighting system to represent the occurrence of ground failure. In ATC-13, a probability of ground failure is assigned to different types of soil deposits as shown in Table 4-4. The additional damage on poor ground (PG) is then estimated from Equations 4-5 and 4-6.

\[
MDF(PG) = MDF(S) \times P(GF) \times 5 \\
MDF(PG) = MDF(S) \times P(GF) \times 10
\]

for surface facilities

for buried facilities

where MDF is the mean damage factor (see Equation 4-2), due to either poor ground (PG) or shaking (S), and P(GF) is the ground failure probability, defined in Table 4-4. The factors of 5 and 10 in Equations 4-5 and 4-6 are obtained through judgement and empirical observations, related to the fact that in the 1906 San Francisco earthquake, it was observed that damage on poor ground was reported to be 5 to 10 times greater than that on firm ground. The final
damage level is the sum of the estimated damage caused by ground shaking (see Section 4.6), ground failure, and landslide (obtained in a similar manner).

Table 4-4: Ground failure probability \( P(GF) \) in ATC-13 (after ATC, 1985)

<table>
<thead>
<tr>
<th>Zone</th>
<th>Type of Deposit</th>
<th>Probability of ground failure, ( P(GFI) ) in % by MMI and Soil Type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>VI</td>
</tr>
<tr>
<td>1a</td>
<td>Stream channel, tidal channel</td>
<td>5</td>
</tr>
<tr>
<td>1b</td>
<td>San Francisco Bay Mud</td>
<td>3</td>
</tr>
<tr>
<td>2a</td>
<td>Holocene Alluvium (water table &lt; 10ft)</td>
<td>2</td>
</tr>
<tr>
<td>2b</td>
<td>Holocene Alluvium (water table &gt; 10ft)</td>
<td>.5</td>
</tr>
<tr>
<td>3</td>
<td>Late Pleistocene Alluvium</td>
<td>.1</td>
</tr>
</tbody>
</table>

The ATC-13 method described above has a very tenuous basis. This reflects the lack of appropriate data for calibration at the time it was developed. Nonetheless, it is important for two reasons, in raising the issue of damage caused by hazards other than ground shaking, and by providing a precursor to other methods that used some form of weighting factor, or increment, on the ground-shaking induced damage to represent the impact of ground failure. In principle, this concept presents a reasonable compromise between data requirements and realistic results. On the other hand, it does not satisfy the stated requirements of loss estimations (Section 1.5) to produce realistic and meaningful results.

For Turkey, Bommer et al. (2002a), nominally allowed for the effect of liquefaction by assigning the softest soil class, NEHRP Class E (FEMA, 1997), to liquefiable deposits, where identified. In this way “losses due to liquefaction are approximated by a corresponding increase in the losses due to ground shaking”.

McCormack and Rad (1997) proposed Equation 4-7 for the estimation of damage likelihood:

\[ P(DF \geq 60\%) = 10^{-S} \]  

(4-7)

where a damage factor (DF) of 60% is assumed to represent ‘major damage’ and \( S \) is a ‘structural score’ reflecting the basic type of structure, reduced for particular features that may increase its vulnerability, and further reduced for the presence of liquefiable soils or potential slope instabilities. Other published approaches that apparently use a weighting factor include that of Lai et al. (2004), who mention, in passing, that “soil liquefaction is considered as a damage weight".
Kiremidjian (1994) presents an approach for the estimation of the damage factor DF due to liquefaction, using empirical relationships between the extent of lateral spreading and damage, shown in Table 4-5. This is based upon a scheme presented by Youd and Perkins (1987a).

**Table 4-5:** Relationship between Liquefaction Severity Index (LSI), which is equivalent to the lateral spread in inches, and the Damage Factor (from Kiremidjian, 1994).

<table>
<thead>
<tr>
<th>LSI</th>
<th>Description</th>
<th>Damage Factor (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>Very sparsely distributed; some sand boils; fissure openings up to 0.1m and ground settlements of up to 25mm</td>
<td>0-1</td>
</tr>
<tr>
<td>10</td>
<td>Sparsely distributed ground effects; sand boils with aprons up to 1m; ground fissures; slumps of few tenths of a metre at steep banks</td>
<td>1-10</td>
</tr>
<tr>
<td>30</td>
<td>Generally sparse but locally abundant ground effects; sand boils up to 2m; some fences and roadways noticeable offset; ground settlement as much as 0.3m</td>
<td>10-30</td>
</tr>
<tr>
<td>50</td>
<td>Abundant effects; sand boils with aprons up to 3m; fissures up to 1.5m wide; fences and roadways are offset or pulled apart as much as 1.5m; ground settlement of more than 0.3m</td>
<td>30-60</td>
</tr>
<tr>
<td>70</td>
<td>Abundant effects; many large sand boils; long fissures with openings as wide as 2m; frequent ground settlements of more than 0.3m</td>
<td>60-100</td>
</tr>
<tr>
<td>90</td>
<td>Very abundant ground effects; numerous sand boils covering as much as 30% of an area with fresh sand; many long fissures; large slumps; widespread settlements of more than 0.3m</td>
<td>100</td>
</tr>
</tbody>
</table>

Limitations of the above approach include the fact that the data were from a particular geological environment, the relationships are defined from a limited dataset and consider lateral spread only. Other than these limitations the approach is both clear, simple to implement and supported by empirical observations.

In terms of accuracy, the optimum approach for the ground failure component would be to predict the probability of liquefaction, the expected ground deformation and the resulting damage to buildings using in situ geotechnical data and detailed building inventory data including foundation type. This option requires specialist input, and above all, analysis to determine the building response, since there are no existing methodologies for this part of the calculation. Therefore, the requirements in terms of input data, analysis and time are significant, often prohibitively so.

As a result of the low level of geotechnical and building inventory data that most loss estimations have to use, a simplified approach, wherein the level of input data and the simplicity of the analysis are commensurate with other aspects of the study, undoubtedly answers the
needs of loss modellers. The HAZUS methodology attempts to answer these needs: it is the only published method for evaluating building damage due to ground failure that does so in a detailed manner, taking account of the expected mode of failure and foundation type. A comprehensive, step-by-step approach is described in the HAZUS technical manuals, which seeks to resolve the contradictory demands of minimising the amount of additional data required and producing realistic estimates of resulting damage. Nonetheless, it has been found that many additional uncertainties are generated through the simplifying assumptions made, and these are poorly constrained and considered, as shown in Section 4.10.

4.10 Evaluation of losses due to liquefaction in HAZUS

The HAZUS (FEMA, 2003) methodology comprises a default and an ‘expert-generated’ ground failure assessment. The latter suggests the use of in situ data, combined with expert input, to define the liquefaction hazard; having done so, the building damage evaluation methodology is the same as for the default approach and therefore, as discussed in Section 4.2.4, there may be little to gain in terms of reduction of uncertainty through this approach.

The default approach makes a number of (necessary) simplifications in order to eliminate the need for in situ geotechnical data, in terms of the susceptibility of ground failure, the expected permanent ground deformation (PGD) and the relationship between PGD and damage. The input data take the form of landslide or liquefaction susceptibility, based upon the geological descriptions in the case of liquefaction, and upon criteria related to topography, geology and ground water conditions for landslides. Such maps are separate to and distinct from the primary input for site amplification, which use the well recognised NEHRP site classification system (See Section 2.7.3). This poses a problem for users in that additional data collection and analysis is required to prepare the input for the ground failure components, despite the significant uncertainties related to the output. Furthermore, the output, in terms of the probability of a building being in a certain damage state due to ground failure, is simplistic in comparison to the ground shaking model described briefly in Section 4.7 and so there is an apparent inconsistency between the complexity of the input and the approximate nature of the output.

Figure 4-16 illustrates the principal stages of defining liquefaction-induced damage according to the HAZUS methodology (FEMA, 2001), described in more detail in the following subsections.
4.10.1 Liquefaction susceptibility

Geological criteria are used to determine the regional susceptibility to liquefaction. The criteria adopted are those originally proposed by Youd and Perkins (1978), shown in Table 2-5. These relate the likelihood of liquefaction to the type and age of deposit, independent of the seismicity of the region. Therefore the input must be presented in the form of a map defining the different soil zones in a study area in terms of the six categories, from none to very high. A degree of judgement is necessary in assigning these categories.

The zones will not necessarily coincide with the site classification zonation using, for example, the NEHRP classification scheme (Section 2.7.3), since liquefaction susceptible soils could be either site class D or E, and similarly, site class E includes soft clays, which are not vulnerable to liquefaction. Youd and Perkins (1978) state that their classification system is “a useful guide for preliminary assessments and generalised regional mapping of susceptibility”. Whether such a guide is therefore sufficient to be used as the basis for all of the subsequent calculations of liquefaction-induced losses is questionable.
4.10.2 Proportion of map unit

Where the above criteria are applied to determine the susceptibility of a particular mapped deposit to liquefaction, it is further acknowledged that due to natural variability, each deposit will not uniformly liquefy under a given level of shaking. The assumption is made that non-susceptible portions of a particular type of deposit are larger for lower susceptibilities, and a factor to represent this is applied, as shown in Table 4-6.

<table>
<thead>
<tr>
<th>Mapped Relative Susceptibility</th>
<th>Proportion of Map Unit, $P_{ml}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very High</td>
<td>0.25</td>
</tr>
<tr>
<td>High</td>
<td>0.2</td>
</tr>
<tr>
<td>Moderate</td>
<td>0.1</td>
</tr>
<tr>
<td>Low</td>
<td>0.05</td>
</tr>
<tr>
<td>Very Low</td>
<td>0.02</td>
</tr>
<tr>
<td>None</td>
<td>0.00</td>
</tr>
</tbody>
</table>

Although it is appropriate to state that there will be natural (or man-made) variability in any deposit and uniform occurrence of liquefaction is unlikely, the factors presented in Table 4-6 have been developed from "judgments developed based on preliminary examination of soil properties data sets compiled for geologic map units characterized for various regional liquefaction studies" (FEMA, 2003). However, they have a significant effect on the overall results, being applied to the overall probability of liquefaction (Equation 4-8). The assumption that the proportion of a given deposit that will liquefy can never be greater than 25% seems inappropriate. Where the amplitude and duration of shaking are sufficient, there is no reason why an entire zone of liquefiable ground should not liquefy. At Port Island and Rokko Island, in the 1995 Kobe earthquake, for example, liquefaction was almost universal (with the exception of the improved ground, and this particular variation should be captured by the susceptibility rating). The validity of these factors is given further consideration in the context of the case studies presented in Chapters 5 and 6.

If a reduction factor is to be applied to the probability of liquefaction, it should consider the uniformity of the soils, the strength and duration of shaking, and the thickness and extent of the liquefiable layers. If in situ geotechnical data were to be used, then the variability of soil deposits would be adequately represented by the scatter in the penetration resistance for example, and no further modification would be required. In the absence of such data, sensitivity of the final results to the uncertainties in the $P_{ml}$ factor must be considered.
4.10.3 Conditional probability of liquefaction

The overall probability of liquefaction is obtained from Equation 4-8:

$$P[Liquefaction_{SC}|PGA=a] = \frac{P[Liquefaction_{SC}|PGA=a]}{K_M \cdot K_W} \cdot P_{ml} \quad (4-8)$$

where $P[Liquefaction_{SC}|PGA=a]$ is the conditional probability for a given susceptibility category (SC) at a specified peak ground acceleration (PGA)

$K_M$ is a correction factor for magnitude

$K_W$ is a correction factor for ground water depth

$P_{ml}$ is the proportion of map unit susceptible to liquefaction (Table 4-6)

The conditional probability term in Equation 4-8 is based upon probabilistic relationships developed by Liao et al. (1988) through regression of an empirical dataset of events where liquefaction either did or not occur, in terms of $(N_1)_{60}$ and cyclic shear stress ratio (CSR) (defined in Section 2.9.3), or in terms of magnitude and distance of the earthquake as an alternative to the local CSR. Their base model (all fines contents) relationship is given in Equations 4-9 and 4-10 for information.

$$Q_L = 10.167 + 4.193 \ln(CSRN) - 0.24375(N_1)_{60} \quad (4-9)$$

$$P_L = \frac{1}{1 + e^{(-Q_L)}} \quad (4-10)$$

where $P_L$ is the conditional probability of liquefaction and CSRN is CSR normalised to M 7.5.

In HAZUS, these relationships are simplified to plot the probability of liquefaction versus PGA for each susceptibility category (Figure 4-17). Implicit in this simplification is an assumed SPT N value for each susceptibility category as well as an assumed depth for the calculation of CSR (see Equation 2-5). Since these assumptions are not fully explained, it is not possible for users to understand the uncertainty associated with these assumptions. Furthermore Seed et al., (2001) observe that the database used in the study by Liao et al. (1988) contained some unreliable data, and their work has largely been superseded by more recent studies (Seed et al., 2001, 2003).
Figure 4-17: Conditional probability relationships used in HAZUS

‘Conditional probabilities’, whether they are calculated using the HAZUS approach or more detailed and up-to-date methods such as Seed et al. (2003), relate the probability of liquefaction to the level of shaking. These relationships have to be normalised to a single magnitude (typically M=7.5) and a single ground water level (in HAZUS \( d_w = 5 \text{ ft} \)). Correction factors are therefore required for other moment magnitudes and ground water levels. A number of magnitude correction factors have been developed; the one adopted in HAZUS is that of Seed and Idriss (1982), which, although widely used, is now superseded by the work of the NCEER workshop (Youd & Idriss, 2001). There is an additional minor, but nonetheless important, uncertainty relating to this use of ‘mix and match’ correction factors. By using a different magnitude correction factor to that used by Liao et al. in their database, an additional uncertainty is introduced.

The application of the magnitude correction factor to the calculated probability (Equation 4-7) means that the probability of liquefaction can only be equal to unity for \( M=7.5 \) or higher. Where an earthquake is of lower magnitude, but is closer to the site, and the soils are highly susceptible there is no practical reason why the probability of liquefaction should not be equal to 1, again considering the case of Port and Rokko Islands in Kobe in 1995 (\( M_w=6.9 \)). One potential solution to this would be to use the energy-based liquefaction model of Davis and Berrill (1982), where the ‘load’ is defined in terms of magnitude and distance rather than PGA. The probabilistic relationships of Liao et al. (1988) are additionally presented in terms of a magnitude and distance related load factor. The author is not aware of any regional liquefaction risk models being developed using this approach.
The use of a probabilistic approach as opposed to a deterministic evaluation of the factor of safety against liquefaction is an essential component of a model like HAZUS, since the overall building damage distribution is expressed probabilistically. However, it seems that the decisions made in order to produce Figure 4-17 are made on a deterministic basis, since there is no consideration of the uncertainty behind the assumptions. Fully probabilistic treatment of the liquefaction potential would require consideration of the natural variability of the PGA and of the SPT N data and the uncertainty related to the depth and thickness of liquefaction. The impact upon the built environment of a probability of liquefaction of unity would be very different if this applied to a 1 m thick layer 8 m below the ground surface or a 6 m layer at the ground surface, for example. Figure 4-17, with reason, seeks a probability of liquefaction for all study regions, based upon the same definition of relative susceptibility for all regions. However, in the light of the assumptions that must be made, and the uncertainties that are ignored, it is suggested that to achieve this without the use of in situ data is not possible.

4.10.4 Permanent ground deformation

4.10.4.1 Lateral spreading

A number of published relationships exist for the estimation of lateral spreading displacement (see Table 2-8). HAZUS uses the relationships for Liquefaction Severity Index (LSI) of Youd and Perkins (1987b), derived from a database from six earthquakes in the Western United States and Alaska:

\[ \log LSI = -3.49 - 1.86 \log R + 0.98 M_w \]  \hspace{1cm} (4-11)

For consistency with the definition of hazard in terms of PGA and spectral accelerations, Equation 4-11 is rewritten in terms of LSI and PGA only, by combining it with an attenuation relationship of the form given in Equation 4-12 after Sadigh et al. (1986), and assuming magnitude \( M_w = 7.5 \), thus eliminating distance from the equation.

\[ \ln PGA = a + bM + c_i (8.5 - M)^{r_i} + d \ln[r + h_i \cdot e^{h_i M}] \]  \hspace{1cm} (4-12)

where the coefficients in Equation 4-12 are defined according to the fault rupture mechanism (strike-slip or reverse-slip) and rock or soil sites. A magnitude correction factor, \( K_M \), having a different definition to \( K_M \) is then applied.
The expected lateral spreading is then calculated from Equation 4-13 and Figure 4-18:

\[
E\{PGD_{SC}\} = K_\Delta \cdot E\{PGD\left(\frac{PGA}{PL_{SC}}\right) = a\} \tag{4-13}
\]

where \(E\{PGD\left(\frac{PGA}{PL_{SC}}\right) = a\} \) is the expected permanent ground deformation (PGD) from Figure 4-18.

**Figure 4-18:** Expected lateral spreading (FEMA, 2003). PGA(t) is a threshold PGA to trigger liquefaction for each susceptibility category (the intersection with the x-axis in Figure 4-17).

Without exploring the minor uncertainties that now begin to accumulate in the methodology, such as the assumption of a particular fault rupture mechanism for Equation 4-12, it is worth noting the major ones, such as the arbitrary combination of four different empirical relationships (Youd & Perkins, 1978; Liao et al., 1988; Youd & Perkins, 1987b and Sadigh et al. 1986) derived from different empirical databases, even though they are mainly from the Western United States. Again there is no attempt to define the uncertainty associated with Equation 4-13, which can be expected to be significant.

As noted in the previous section, the use of magnitude and distance as opposed to PGA would improve this approach, since Equation 4-11 could then be used directly.

**4.10.4.2 Settlement**

The degree of vertical settlement is estimated with reference to published relationships such as Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992). These relationships relate volumetric strain to measured relative density (e.g. Figure 4-19). Therefore if \((N_i)_{60}\) and the thickness of liquefied layer are assumed, as before, the amount of vertical settlement as a
function of CSR can be determined. HAZUS makes the additional assumption that, given the occurrence of liquefaction, the amplitude of the settlement actually has very little dependence on the CSR, and settlements are estimated according to the values in Table 4-7.

![Figure 4-19: Determination of volumetric strain for saturated sands (Tokimatsu & Seed, 1987)](image)

**Table 4-7:** Ground settlement amplitudes for liquefaction susceptibility categories (after FEMA, 2003)

<table>
<thead>
<tr>
<th>Mapped Relative Susceptibility</th>
<th>Settlement (mm)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very High</td>
<td>305</td>
</tr>
<tr>
<td>High</td>
<td>150</td>
</tr>
<tr>
<td>Moderate</td>
<td>50</td>
</tr>
<tr>
<td>Low</td>
<td>25</td>
</tr>
<tr>
<td>Very Low</td>
<td>0</td>
</tr>
<tr>
<td>None</td>
<td>0</td>
</tr>
</tbody>
</table>

* Converted from inches in original table, and rounded to nearest 5mm.

Once again, there are uncertainties related to the assumed SPT N values and the thickness of the layer. 300 mm of settlement for example, according to Figure 4-19, would require a 6 m thick liquefied layer with an \(N_{100}\) of the order of 3 throughout the layer (5% strain). This is an extreme scenario, which is unlikely to be representative of many zones assigned very high susceptibility. The accuracy of most empirical methods for estimating either settlement or lateral spread, have only been shown to be within a factor of two or three (Glaser, 1994), and again this uncertainty is ignored.

### 4.10.5 Calculation of building damage

The first four stages in Figure 4-16, described in sub-sections 4.10.1 to 4.10.4, produce a probability of liquefaction for a scenario earthquake event, and an estimated permanent ground
deformation (PGD) at the ground surface. Simplified fragility curves, relating PGD to damage
distribution have been developed (Table 4-8 and Figure 4-20). The only distinction made with
respect to building classifications is between shallow and deep foundations.

Table 4-8: Fragility functions for buildings on failed ground (after HAZUS, FEMA, 2003)

<table>
<thead>
<tr>
<th>Foundation Type</th>
<th>P[E or C][PGD]</th>
<th>Settlement PGD (inches)</th>
<th>Lateral PGD (inches)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Shallow</td>
<td>0.1</td>
<td>2</td>
<td>12</td>
</tr>
<tr>
<td>Shallow</td>
<td>0.5 (median)</td>
<td>10</td>
<td>60</td>
</tr>
<tr>
<td>Deep</td>
<td>0.01</td>
<td>2</td>
<td>-</td>
</tr>
<tr>
<td>Deep</td>
<td>0.05</td>
<td>10</td>
<td>-</td>
</tr>
<tr>
<td>Deep</td>
<td>0.05</td>
<td>-</td>
<td>12</td>
</tr>
<tr>
<td>Deep</td>
<td>0.25</td>
<td>-</td>
<td>60</td>
</tr>
</tbody>
</table>

P[E or C][PGD] is the probability of extensive or complete damage for a given permanent ground
deformation; the values in Table 4-8 are assumed to be lognormally distributed, with a standard
deviation of 1.2 (see Figure 4-20). The justification given in the HAZUS manuals (FEMA,
2003) for the above values, are that:

The assumptions are based on the expectation that about 10 (i.e., 8 Extensive
damage, 2 Complete damage) out of 100 buildings on spread footings would be
severely damaged for 2 inches of settlement PGD or 12 inches of lateral spread
PGD, and that about 50 (i.e., 40 Extensive damage, 10 Complete damage) out of
100 buildings on spread footings would be severely damaged for 10 inches of
settlement PGD or 60 inches of lateral spread PGD.

The lognormal standard deviation of 1.2 therefore comes from nothing more than the factor of 5
between the 10th and 50th percentiles in the above statement, i.e. from judgement. These
fragility curves are used for permanent ground deformations caused by liquefaction, slope
failures or fault rupture, despite the very different effect that these can have on building stock.
If anything, the assumptions and judgements underlying these curves are more representative of
liquefaction-induced damage than either slope failures or fault ruptures.

The primary assumptions that have been made to produce these fragility curves are:

- Buildings will be either undamaged, or severely damaged due to ground failure. Slight
  or moderate damage due to ground failure is assumed to be included in estimations of
damage due to ground shaking.

- Lateral displacements would need to be significantly larger than vertical settlement to
  cause the same degree of building damage.
Damage due to ground failure is the same for all building categories provided they have the same foundation type and buildings on deep foundations such as piles will perform significantly better than those on shallow foundations such as spread footings.

Piled foundations will be more effective in reducing damage due to settlement than due to lateral spreading.

![HAZUS fragility curve for shallow foundations on ground failures (liquefaction or landslide). Curves show a cumulative lognormal distribution with a standard deviation of 1.2 and median values from Table 4-8. $P(E \text{ or } C|PGD) = \text{Probability of extensive or complete damage states due to permanent ground deformation (PGD).}$](image)

With respect to the first assumption above, detailed correlation of HAZUS predictions to empirical observations would be required to explore this assumption, but in the light of the review presented in Chapter 3, it seems that light to moderate building damage (such as minor to moderate settlements, basement damage or tilting within repairable limits) has a higher probability than severe damage due to liquefaction, although the assumption does generally hold true for landslide-induced damage.

The suggestion that lateral displacements are less damaging than vertical ones does not necessarily coincide with observations in the field; structures can be pulled apart by differential lateral displacements, as was observed at Moss Landing after the 1989 Loma Prieta earthquake. However, considering the average behaviour of all buildings on lateral spreads, it may be reasonable. It is understood that the numbers in Table 4-8 are intended to be representative of the fact that on a large lateral spread, many buildings in the middle of the spread would not actually suffer structural damage; the proportions being obtained from a database of wooden houses on lateral spreads in Japan (J. Eidinger, pers. comm., 2004). Whilst this observation is realistic, to attempt to apply such data to all building stock throughout the United States comprises a very crude modelling approach.
A more realistic approach (but also more complex) would be to consider the percentage of buildings that would experience differential ground movements and those that would experience uniform ground movements, and then determine the mode of damage of each of these subgroups depending upon their foundation type. HAZUS does not distinguish between rigid and ‘flexible’ shallow foundations, a fundamentally important variable.

Considering the third point in the list above, particularly for buildings on flexible foundations such as spread footings, to ignore the details of the superstructure is also inappropriate, since, an unreinforced masonry building would respond differently to differential foundation movements than a reinforced concrete frame for example. Variables such as the building height, the quality of construction and the type and quality of the shallow foundations have been shown to influence a building’s resistance to ground deformation-induced damage.

The final assumption, regarding piled foundations, is reasonable in the light of observations in the field. Settlement will only lead to a potential reduction in static bearing capacity due to loss of skin friction, and is very unlikely to reduce the static factor of safety to less than one. The load applied to piles by lateral spreading soil, on the other hand, can potentially damage them.

4.10.6 Summary

The HAZUS approach for determining liquefaction potential is simple, and straightforward to implement within a GIS framework, where the probability of liquefaction for a given earthquake scenario could be determined in a few simple steps, using a map of soil types and their susceptibility as input. However, there are many flaws and simplifications in the approach described. Despite the apparent use of published data and logical analysis to determine the expected ground deformations, it seems that at the end, the numbers presented are due in a very large part to subjective judgement, and therefore have no particular advantages over others, such as the empirically based ATC-13 approach. In fact the implication that analytically defined hazard and fragility curves are used is misleading to users, who may reasonably assume that this has been done to a standard commensurate with the treatment of ground-shaking damage in HAZUS. Even if the uncertainties related to the liquefaction probability and the expected ground deformation could be reduced using in situ geotechnical data, and more considered analysis, the fact that these ground deformations are then used as input to the subsequent damage calculation means that the overall uncertainty cannot be significantly reduced. These vitally important issues are the focus of subsequent chapters of this thesis.
Most of the criticisms made of the methodology relate to the lack of data and the simplifying assumptions made because of that. These issues highlight some of the difficulties in attempting to simplify the ground failure component of a loss estimation model so that it can be undertaken without large volumes of additional data and detailed analysis. There is certainly potential to improve understanding of the relative importance of these uncertainties with respect to the overall results of a loss estimation, such that users can concentrate on reducing those which have the most impact. One of the objectives of the case studies presented in Chapters 5 and 6 is to understand better what these uncertainties are, and their impact on loss modelling.

### 4.11 Combined ground shaking and ground failure

The final estimation of the damage state must consider the combination of ground shaking and ground failure. In HAZUS for example, the probability of a particular damage state being exceeded is assumed to be:

\[
P(DS \geq X) = P(DS \geq X)_{GS} + P(DS \geq X)_{GF} - P(DS \geq X)_{GS} \cdot P(DS \geq X)_{GF}
\]  

(4-14)

where \(P(DS \geq X)\) is the probability that the damage state is greater than or equal to damage state \(X\); \(GS\) signifies damage due to ground shaking and \(GF\) damage due to ground failure. The two hazards are thus assumed to be independent (see also Section 3.6.2).

In ATC-13, the final damage state was assumed to be the sum of the damage caused by each individual hazard, i.e.:

\[
MDF(\text{Total}) = MDF(\text{shaking}) + MDF(\text{poor ground}) + MDF(\text{landslide}) + MDF(\text{fault rupture}) + MDF(\text{inundation})
\]  

(4-15)

with \(MDF(\text{Total}) \leq 1\)

McCormack and Rad (1997), at the other extreme, take the larger of the damage amplification factor (\(S\) in Equation 4-6) considering either soil amplification or liquefaction-induced damage "because it was felt that Liquefaction/Lateral Spread and Ground Motion Amplification hazards are unlikely to occur concurrently, since liquefied soil will most likely lose the shear strength needed to significantly amplify shaking."

In ATC-36, a development of ATC-13 for Salt Lake County, Utah (Rojahn et al., 1997), Equation 4-15 was deemed over-conservative and therefore the square root of the sum of the squares (SRSS) was used to combine the various hazards.
In fact, as noted in Section 3.6.2, the interaction between ground shaking and ground failure is a complex feature, and although examples of combined damage are rare, they are nonetheless relevant. A rational approach for modelling the combined hazards is clearly an issue that warrants further investigation.

4.12 Discussion

Loss estimation is still a relatively new field, and certain aspects, such as the target end user and the location, have influenced the development of methodologies more strongly than others. This has led to a lack of balance in some existing methods, where certain components are considered in much more detail than others. The best example of this is HAZUS, where the advanced treatment of displacement-based building performance is inconsistent to say the least with the crude incorporation of landslide and liquefaction, as shown in this chapter.

Clearly the ground failure component of loss models is currently significantly less advanced than the ground shaking component. By specifically incorporating ground failure, the HAZUS methodology presents a very valuable step forward in the field, and it is particularly useful by virtue of being freely and publicly available. The methodology attempts to fill the gap noted by FEMA in 1994 in their review of current loss methodologies, where it was observed that damage due to collateral hazards was not thoroughly treated in the literature, existing methods being "crude and incomplete". However, it is contended that the HAZUS approach has not actually improved this gap in loss estimation methodologies to any great extent, other than possibly by raising awareness that it should be included. One reason for this must relate to the fact that certain elements of the HAZUS methodology were carried out on a remarkably low budget (C.F. Kircher, pers. comm. to J. Bommer, 2002). The need to improve upon HAZUS, or to develop alternative procedures to HAZUS for this particular component was supported by Professor R.V. Whitman, the chairman of the HAZUS earthquake committee, who commented that "The methods HAZUS uses to evaluate settlement and lateral spread certainly are very crude. I hope you come up with procedures that might improve HAZUS's methodology." (R.V. Whitman, pers. comm., 2003).
Chapter 5

5 LOSSES DUE TO GROUND FAILURE IN ADAPAZARI, TURKEY

The city of Adapazari in northwest Turkey suffered extensive ground failure during the 17 August 1999 Kocaeli earthquake. The manifestation of ground failure was primarily in the form of liquefaction-induced settlement and sand boils. Damage to buildings in liquefied zones was observed to take the form of tilting, settlement and lateral displacement, as well as structural damage due to ground shaking in the form of cracking, deformation and collapse. As noted previously in Chapter 3, in terms of assessing earthquake losses due to building damage, a structure that has undergone excessive tilting without significant structural damage such as that shown in Figure 5-1 is suitable only for demolition, and its damage state may therefore be described as complete in the same way as for a building which has undergone total collapse (Figure 5-2). The costs of the two different modes of failure differ in that loss of life, injury, damage to contents and non-structural elements would all be less for the first mode, whereas in terms of functionality, both buildings are beyond repair, so the structural damage costs are essentially the same, and furthermore, the building in Figure 5-1 would incur additional demolition costs.

Figure 5-1: Tilted building due to liquefaction in central Adapazari. (Courtesy Earthquake Engineering Field Investigation Team (EEFIT), UK)

The earthquake review presented in Chapter 3 led to the conclusion that ground failure effects will only really begin to dominate earthquake damage and losses when the study area being considered is relatively small. In most cases, when a large region is considered, damage due to ground shaking is most likely to dominate the damage patterns. The downtown area of Adapazari is a prime example of a zone where ground failure influenced the observed damage;
this is why it has been selected for further investigation. The damage survey data for Adapazari are considered in the context of future loss modelling in similar zones.

Figure 5-2: Collapse due to ground shaking, Nato Caddesi, Adapazari. (Courtesy EEFIT, UK)

The aim of this chapter is to understand how the damage distribution was affected by the widespread occurrence of ground failure, in terms of the overall behaviour of the building stock, and how existing methodologies perform in seeking to predict this behaviour. Having identified a number of shortcomings related to existing methodologies in this field in the previous chapter, it is appropriate to consider the significance of these in the context of real data.

Figure 5-3: Map of Turkey (CIA, 2004)
5.1 The 1999 Kocaeli earthquake

The Turkish earthquake of 17 August 1999, known as the Kocaeli earthquake, had a magnitude $M_w = 7.4^3$, a strike-slip fault rupture mechanism and a focal depth of 17 km. The epicentral co-ordinates were 40.702° N, 29.987°E. The earthquake ruptured 126 km of the North Anatolian Fault, and was followed within 3 months by the $M_w$ 7.1 Düzce earthquake (Figure 5-4). The fault rupture was almost pure right-lateral strike-slip with a near vertical fault plane.

The affected region in northwest Turkey is the industrial heartland of the country, home to over 15 million people and 40% of Turkish industry (Youd et al., 2000). The direct economic losses have been estimated to be $9 billion, with more than 17,000 fatalities and approximately 50,000 hospitalised injuries (Erdik, 2000).

Figure 5-4: Regional tectonic setting of the North Anatolian Fault and the ruptures of the two earthquakes in 1999 (Lettis et al., 2000)

The intensity distribution of the earthquake is shown in Figure 5-5, the total affected area was approximately 10,000 km$^2$ (Youd et al., 2000). Strong motion recordings from within 100 km of the fault rupture are shown in Table 5-1 and the station locations are in Figure 5-6.

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3 The Harvard CMT Catalogue reports $M_w=7.6$ for this earthquake. Despite the reliability of this source, the more commonly reported value of $M_w=7.4$ (USGS) has been adopted herein for consistency with other studies such as Sancio (2003). This variability leads to an additional uncertainty.
Chapter 5  
Losses due to ground failure in Adapazari, Turkey

5.1.1 Strong-motion data

The strong-motion records listed in Table 5-1 were downloaded from the Internet Site for European Strong-motion Data (ISESD) (Ambraseys et al., 2002). Acceleration-distance pairs for 21 records at fault distances of less than 100 km, for which the local geology is approximately known, are shown in Figure 5-7. These data are plotted at T=0s (PGA), and T=0.3s and 1.0s. Particular features of Figure 5-7 include the relatively low amplitudes at shorter distances, the large scatter in the data at greater distances (70 to 100 km), and the scarcity of records in the middle distance range of 20 to 60 km.

Figure 5-6: Location of strong motion stations in the Sea of Marmara region
### Table 5-1: Strong-Motion Recordings from the 17 August 1999 Kocaeli mainshock (Ambraseys et al., 2002; Anderson et al., 2000; Kudo et al., 2002, Sucuoglu, 2002)

<table>
<thead>
<tr>
<th>Station Name</th>
<th>Code</th>
<th>Owner</th>
<th>Co-ordinates</th>
<th>Local geology</th>
<th>Epicentral dist. (km)</th>
<th>Fault dist. (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ambarli-Termik Santrali</td>
<td>ATS</td>
<td>Kandilli²</td>
<td>40.981</td>
<td>28.693</td>
<td>Soft soil</td>
<td>113</td>
</tr>
<tr>
<td>Bursa-Sivil Savunma Mudurluga</td>
<td>BRS</td>
<td>GDDA³</td>
<td>40.183</td>
<td>29.131</td>
<td>Stiff soil</td>
<td>93</td>
</tr>
<tr>
<td>Bursa-Tofa Fabrikasi</td>
<td>BUR</td>
<td>Kandilli</td>
<td>40.261</td>
<td>29.068</td>
<td>Soft soil</td>
<td>92</td>
</tr>
<tr>
<td>Cekmece-Kucuk</td>
<td>CNAK</td>
<td>Kandilli</td>
<td>41.024</td>
<td>28.759</td>
<td>Stiff soil</td>
<td>110</td>
</tr>
<tr>
<td>Duzce-Meteoroloji Mudurlugu</td>
<td>DZC</td>
<td>GDDA</td>
<td>40.85</td>
<td>31.17</td>
<td>Soft soil</td>
<td>101</td>
</tr>
<tr>
<td>Fatih-Tomb</td>
<td>FAT</td>
<td>Kandilli</td>
<td>41.02</td>
<td>28.95</td>
<td>Soft soil</td>
<td>94</td>
</tr>
<tr>
<td>Gebze-Arcelik</td>
<td>ARC</td>
<td>Kandilli</td>
<td>40.824</td>
<td>29.361</td>
<td>Rock</td>
<td>55</td>
</tr>
<tr>
<td>Gebze-Tubitak MAM</td>
<td>GBZ</td>
<td>GDDA</td>
<td>40.82</td>
<td>29.44</td>
<td>Stiff soil</td>
<td>48</td>
</tr>
<tr>
<td>Goynuk-Devlet Hastanesi</td>
<td>GYN</td>
<td>GDDA</td>
<td>40.381</td>
<td>30.734</td>
<td>Stiff soil</td>
<td>73</td>
</tr>
<tr>
<td>Heybeliada-Senatoryum</td>
<td>HAS</td>
<td>Kandilli</td>
<td>40.869</td>
<td>29.088</td>
<td>Rock</td>
<td>78</td>
</tr>
<tr>
<td>Istanbul-Atakoy</td>
<td>ATK</td>
<td>ITU⁴</td>
<td>40.989</td>
<td>28.849</td>
<td>Stiff soil</td>
<td>101</td>
</tr>
<tr>
<td>Istanbul-Bayindirlik ve Iskan Mudurlugu</td>
<td>IST</td>
<td>GDDA</td>
<td>41.08</td>
<td>29.09</td>
<td>Stiff soil</td>
<td>86</td>
</tr>
<tr>
<td>Istanbul-Maslak</td>
<td>MSK</td>
<td>ITU</td>
<td>41.104</td>
<td>29.019</td>
<td>Rock</td>
<td>93</td>
</tr>
<tr>
<td>Istanbul-Mecidiyekoy</td>
<td>MCD</td>
<td>ITU</td>
<td>41.065</td>
<td>28.997</td>
<td>Stiff soil</td>
<td>93</td>
</tr>
<tr>
<td>Istanbul-Zeytinburnu</td>
<td>ZYT</td>
<td>ITU</td>
<td>40.986</td>
<td>28.908</td>
<td>Stiff soil</td>
<td>96</td>
</tr>
<tr>
<td>Izmit-Meteoroloji Istasyonu</td>
<td>IZT</td>
<td>GDDA</td>
<td>40.79</td>
<td>29.96</td>
<td>Rock</td>
<td>10</td>
</tr>
<tr>
<td>Iznik-Karayollari Sefili Muracaati</td>
<td>IZN</td>
<td>GDDA</td>
<td>40.437</td>
<td>29.691</td>
<td>Soft soil</td>
<td>39</td>
</tr>
<tr>
<td>Sakarya-Bayindirlik ve Iskan Mudurlugu</td>
<td>SKR</td>
<td>GDDA</td>
<td>40.737</td>
<td>30.384</td>
<td>Rock</td>
<td>34</td>
</tr>
<tr>
<td>Yapi-Kredi Plaza Levent</td>
<td>YKP</td>
<td>Kandilli</td>
<td>41.081</td>
<td>29.011</td>
<td>Rock</td>
<td>92</td>
</tr>
<tr>
<td>Yarimca-Petkim</td>
<td>YPT</td>
<td>Kandilli</td>
<td>40.764</td>
<td>29.762</td>
<td>Soft soil</td>
<td>20</td>
</tr>
<tr>
<td>Yesilkoy-Havaalanı</td>
<td>DHM</td>
<td>Kandilli</td>
<td>40.982</td>
<td>28.82</td>
<td>Stiff soil</td>
<td>103</td>
</tr>
</tbody>
</table>

¹ Closest horizontal distance to the surface projection of the fault, also known as Joyner & Boore distance (Rjb)² Kandilli Observatory and Earthquake Engineering Research Institute of Bogazici University; ³ Earthquake Research Department of the General Directorate of Disaster Affairs; ⁴ Istanbul Technical University
The accelerations recorded during the Kocaeli earthquake have been observed to be lower, particularly in the near-fault region, than the predicted accelerations using relationships developed for shallow crustal earthquakes (e.g. Anderson et al., 2000; Erdik & Durukal, 2001). The relationships of Boore et al. (1997), for example, which have been used in previous studies for the region (e.g. Bommer et al., 2002a), were found to give a reasonable fit to the measured accelerations.
data from the Kocaeli earthquake at distances greater than about 30 km, and it was also observed that agreement was better for spectral ordinates than for PGA (Figure 5-8). Suggested reasons for the lower than expected near-source accelerations include the smoothness of the rupture, and possibly the fact that the fault rupture broke the ground surface. Somerville (2000) has suggested that the ground motions from earthquakes where surface rupture occurs may be weaker than those where the rupture remains below the Earth’s surface.

*Figure 5-8: Comparison of Boore et al. (1997) random horizontal component (left hand side) and Güikan and Kalkan (2002) largest component (right hand side) with PGA, 0.3s and 1.0s spectral accelerations (5% damping) for the 17/08/99 Kocaeli earthquake for rock sites (solid line) and stiff soil sites (dashed line). Dotted lines show +/- 1 logarithmic standard deviation from the logarithmic mean for rock sites. SKR station is stiff soil, all others are rock.*

Güikan and Kalkan (2002) have derived ground-motion prediction relationships using Turkish strong motion data; these are also compared to the measured data in Figure 5-8. Their dataset
was heavily dominated by the two earthquakes in 1999. This dependency makes it a potentially unsuitable choice for future seismic hazard assessments, but gives an improved fit to the measured near-field data from the Kocaeli event and is therefore appropriate for this study, which seeks to reduce as far as possible the uncertainty in the estimated ground motions from the Kocaeli earthquake.

Preliminary checks were also carried out using the relationships of Spudich et al. (1999) and Ambraseys et al. (1996). The non-Turkish relationships consistently over-estimated the measured accelerations at short distances, which suggests they would not be suitable for the prediction of the ground-motions in downtown Adapazari, at 7km from the fault. More recent relationships for Turkey have been published by Ozbey et al. (2004). Figure 5-9 shows that at short distances there is relatively little difference between the two Turkish relationships.

![Figure 5-9: Comparison of Boore et al., (1997), Ozbey et al., (2004) and Gülkan and Kalkan (2002) to measured data on rock sites. Note that the first two predict the random horizontal component whereas the latter predicts the larger.](image)

5.1.2 Adapazari strong-motion

The uncertainty related to the estimated shaking demand in central Adapazari is significantly less than it would be for many studies of future losses, because the magnitude and location of the earthquake are known, and there is a strong-motion record from within four kilometres of the study area. Furthermore the local ground conditions in the city centre are known, due to the extensive investigations carried out following the earthquake (Sancio, 2003).
Figure 5-10: Location of Adapazari and other affected cities, with respect to the fault location, earthquake epicentre and strong-motion stations (Spence et al., 2003)

Sakarya station (SKR) is on the outskirts of the city, located approximately 3 km from the fault rupture and 4 km from the downtown area where extensive liquefaction was observed. Figure 5-10 shows the location of the city and the station with respect to the fault rupture and other affected cities. Figure 5-12 shows the location of the station with respect to the main damaged areas and also the four survey lines considered in this case study. The recorded motion in Figure 5-11 will differ from those in central Adapazari due to both site conditions and distance from the fault rupture. The station is located on dense soil to soft rock, with a measured shear wave velocity ($V_{330}$) = 470 m/s (Rathje et al., 2003). The PGA at ground surface was 0.41 g.

Figure 5-11: 5% damped elastic response spectra, EW component, recorded at Sakarya (SKR) station, southwest of Adapazari. Instrument malfunction meant no NS component was recorded. EW direction is approximately fault parallel.
The central area of the city is underlain by alluvial sediments, extending to depths of greater than 300m. The near surface soils are recent deposits created by the Sakarya and Cark rivers that bound the city (Figure 5-12). Published estimates of the surface PGA in the central area are
summarised in Table 5-2, PGA being significant in this case since it is the required input parameter for liquefaction assessments. The two approaches to estimate the surface acceleration for this study were:

(i) The Gülkan and Kalkan (2002) relationships were used to estimate ground motions for a fault distance of 7 km, $M_w=7.4$, for a $V_{s30}$ of 200 m/s, the average value obtained from geotechnical investigation data from the PEER Center (http://peer.berkeley.edu/turkey/adapazari/) and shear wave velocity profiles from Cox (2001).

(ii) The same relationships of Gülkan and Kalkan were used to estimate the bedrock accelerations for NEHRP site class B ($V_{s30} = 760$ to 1500 m/s). The NEHRP amplification factors for short period accelerations ($T = 0.3s$) were then applied in order to estimate the surface accelerations. In accordance with the $V_{s30}$ value of 200 m/s, an intermediate site class D/E was assigned. The assumption that the NEHRP factors for $T=0.3$ s may also be applied to PGA is in line with the HAZUS methodology.

Table 5-2: Estimated surface PGA in central Adapazari (7 km from fault rupture), not including the effects of liquefaction.

<table>
<thead>
<tr>
<th>Source</th>
<th>Surface PGA (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bakir et al. (2001); Gülkan et al. (2004)</td>
<td>0.43</td>
</tr>
<tr>
<td>Sancio et al. (2002)</td>
<td>0.3 to 0.4</td>
</tr>
<tr>
<td>Bray et al. (2004)</td>
<td>0.3 to 0.5</td>
</tr>
<tr>
<td>Yasuda et al. (2001)</td>
<td>0.4</td>
</tr>
<tr>
<td>Sancio (2003)</td>
<td>0.35 to 0.5 (best estimate 0.4 to 0.45)</td>
</tr>
<tr>
<td>This study (i)</td>
<td>0.41</td>
</tr>
<tr>
<td>This study (ii)</td>
<td>0.33 to 0.4</td>
</tr>
</tbody>
</table>

None of these estimates consider the influence of liquefaction on the surface ground motion (e.g. Figure 2-15). The published values have been obtained in a number of ways, including equivalent linear 1-d site response analyses (Bakir et al., 2001; Bray et al., 2004). The estimated values obtained for this study show reasonable agreement with each other and with the other published values.

5.2 Background to case study

Following the two catastrophic earthquakes in northern Turkey in August and October 1999, the Turkish Government found itself unacceptably exposed to carrying much of the resulting
US$20 million in costs. One of the direct results of this was a decision, by the Turkish Government, to establish a compulsory insurance policy for all householders to offset future reconstruction costs. This scheme is known as the Turkish Catastrophic Insurance Pool (TCIP), and is based upon similar lines to the California Earthquake Insurance scheme. A national loss estimation study was carried out to establish the economic risk for Turkey's building stock due to future earthquakes (Bommer et al., 2002a; Spence et al., 2002). The results of the study were then used to evaluate the benefits of insuring Turkish property and to design the reinsurance scheme.

Subsequent to the completion of this study, Spence et al. (2003) undertook a comparison of predicted damage using the TCIP model and the results of detailed damage surveys carried out in the cities of Gölcük and İzmit (Figure 5-10) following the Kocaeli earthquake. Some of the conclusions of this study were briefly discussed in Section 4.7.1. In summary, the findings suggested substantial over-prediction of damage in the near-source region, for mid-rise reinforced concrete (RC) frame buildings, and masonry buildings with RC slabs and roofs. Sensitivity studies suggested that these differences could be partly explained by the uncertainty in the estimated level of ground shaking. More significant however, was the sensitivity to the assumed capacity curve, which failed to represent the real behaviour of the buildings in the case of frames with brittle infill walls or soft storeys.

As noted by Spence et al. (2003), it was not possible to draw any firm conclusions regarding the ground shaking demand and its influence on the predicted results. Without a strong-motion recording from exactly the same location as the damage data, the uncertainty related to this component cannot be fully determined. The dependency of the predicted results on the various input variables can only be fully evaluated by holding at least some of these variables fixed at their known values, so as to investigate the influence of the remaining unknown variables. Without this opportunity, while the sensitivity of the predicted damage can be investigated, the source of the incorrect results can not be established.

As noted above, the ground shaking in central Adapazari can be estimated with some confidence. Furthermore, the ground failure demand, quantified in terms of the permanent ground deformation, has been directly measured in the city centre as part of the damage survey carried out by Bray and Stewart (2000). The available data are therefore considered to be sufficient to correlate observed damage with demand, and to make a useful comparison between the estimated demand and the observed building performance, using simplified approaches.
The case study in this chapter makes combined use of the loss estimation framework for Turkey described by Bommer et al. (2002a), the survey data presented by Bray and Stewart (2000) and the geotechnical investigation and detailed studies of building response carried out by Rodolfo Sancio at the University of California, Berkeley from 2000 to 2003 (Sancio, 2003) under the auspices of the Pacific Earthquake Engineering Research (PEER) Center.

5.3 Earthquake damage in Adapazari

Adapazari has a population of approximately 190,000. The official loss of life there due to the Kocaeli earthquake was 2,627, although the actual number was probably much higher (Bray & Stewart, 2000). Some 27% of the city’s buildings suffered heavy damage or collapse. Proximity to the fault rupture and the presence of deep alluvial sediments undoubtedly contributed to Adapazari suffering the greatest loss of life and gross building damage of all the cities affected by the earthquake (Bray & Stewart, 2000).

Damage was concentrated in the central areas of the city, whereas relatively less damage was reported in the southern areas, where the bedrock is nearer to the surface (see Figure 5-12). In the central zone there is a greater density of mid-rise buildings (4-6 storeys), whereas towards the outskirts, buildings are reportedly less concentrated and there are more low-rise residential buildings. Lifelines also fared very badly, particularly the destruction of the water distribution network (Section 3.4.4). Visible structural damage was in the form of cracks, deformation or collapse (e.g. Figure 5-13). Foundation damage was in the form of settlement, tilting or lateral displacements.
The most notable feature of earthquake damage in Adapazari, as reported by numerous investigators was the "spectacular and extensive occurrence of liquefaction" (Bardet & Seed, 2000).

5.3.1 Damage in zones of ground failure

Ground failure-related building damage in Adapazari typically fell into one of the three categories described below:

- Uniform vertical displacement of foundations. Some settled buildings showed evidence of bulging in the surrounding pavements (Figure 3-5) indicating bearing capacity failure of the foundation soils. Others settled with no observable bulging (Figure 3-6).
- Non-uniform vertical displacement, causing tilts of typically between 1° and 3°, sometimes much more.
- Lateral translation of buildings by as much as one metre.

5.3.2 Geotechnical investigation data

An extensive geotechnical investigation consisting of soil borings with standard penetration tests (SPT) and cone penetration tests (CPT) was carried out in Adapazari following the earthquake, with the objective of characterising the subsurface conditions at sites where ground failure was or was not observed (Bray et al., 2001b). 90 CPTs and 14 soil borings were performed along the lines of the damage surveys discussed in the proceeding section. Most of the borings were no deeper than 10 m, but several were extended to depths as great as 30 m, in order to characterise the deeper soils.

Sancio et al. (2004) showed that local variations in the characteristics of the alluvial sediments in Adapazari appear to have played an integral role in the occurrence and non-occurrence of ground failure and associated building damage. The degree of ground failure observed in the study area was principally controlled by soil condition, with ground failure occurring in zones that are susceptible to liquefaction. No clear evidence was found that the type of structure significantly influenced the degree of ground failure but the localisation of settlements around buildings and the absence of observations of sand boils etc away from buildings suggested that ground strains associated with soil-structure interaction may have contributed to the triggering and severity of ground failure (Bird et al., 2004).
5.3.3 Building damage data

Rapid damage surveys were performed for 725 buildings along the four survey lines shown in Figure 5-14 by Bray and Stewart (2000), within three weeks of the earthquake. The survey data for each building comprised the number of storeys, presence of basements, type of building, structural damage, settlement, tilt, lateral movement and presence of sand boils as well as any other notes of interest. Damage levels were quantified using the structural damage index originally presented by Coburn and Spence (1992). The HAZUS damage state descriptions, used in the TCIP methodology, are similar, except that in HAZUS, partial collapse and complete collapse are combined into the single damage state complete, since they are both indicative of 100% loss (Table 5-3).

The correlation of the two damage scales is important, since the lack of correlation introduces an additional uncertainty in the comparison of predicted versus observed damage distributions. However, as Table 5-3 shows, in this case the two scales do not directly correlate. The structural damage index D1, as applied to the Adapazari survey data, does not include any visible structural damage (J.D. Bray, Pers. Comm., January 2004) whereas the damage state slight used in the TCIP/HAZUS model includes hairline cracks to beams and columns. The relationships between the first three damage states are therefore slightly skewed.
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Table 5-3: Structural damage index of Coburn and Spence (1992), as adapted by Bray and Stewart (2000) for their damage survey, correlated to the HAZUS damage scale (FEMA, 2003) used for the damage predictions presented in this chapter.

<table>
<thead>
<tr>
<th>Structural Damage Index (SDI)</th>
<th>Description</th>
<th>Equivalent HAZUS damage state</th>
<th>Description of structural damage for RC frame or URM buildings</th>
</tr>
</thead>
<tbody>
<tr>
<td>D0</td>
<td>None; No observable damage</td>
<td>Undamaged</td>
<td>None</td>
</tr>
<tr>
<td>D1</td>
<td>Light Damage; architectural damage only</td>
<td>Slight</td>
<td>Some hairline cracks in beams and columns, some hairline cracks on masonry wall surfaces</td>
</tr>
<tr>
<td>D2</td>
<td>Moderate damage, some cracks in load-bearing elements</td>
<td>Moderate</td>
<td>Hairline cracks in most RC beams and joints, some larger flexural cracks, cracks in most masonry walls</td>
</tr>
<tr>
<td>D3</td>
<td>Heavy damage, cracked load-bearing elements with some deformation across the cracks</td>
<td>Extensive</td>
<td>Large cracks in some RC frame elements, shear failures in some non-ductile frames. Extensive cracking in most masonry walls.</td>
</tr>
<tr>
<td>D4</td>
<td>Partial collapse, collapse of a portion of the building</td>
<td>Complete</td>
<td>At least 10-20% (25% for URM structures) of the structure’s floor area has collapsed.</td>
</tr>
<tr>
<td>D5</td>
<td>Collapse, complete collapse of a structure or loss of a floor</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

In this study, it is assumed that there is, in fact, a direct correlation between D0 and undamaged, D1 and slight damage, and D2 and moderate damage. Considering the uncertainties associated with the damage prediction, and the fact that a range of building behaviour is encompassed within each damage state already, it was judged appropriate to leave the two scales as shown in Table 5-3 and assume correlation between them. The significance of this assumption with respect to the results is discussed later.

The Ground Failure Index (Table 5-4) was also developed by Bray and Stewart (2000), in order to classify the distribution and severity of the foundation damage. A common measure of ground failure-induced displacements is invaluable for calibration efforts such as this. If the only damage definitions applied were those presented in Table 5-3, a building that had experienced tilt or settlement with no structural or cosmetic cracking would strictly be classified as undamaged. This shortcoming applies to all the damage scales used in loss modelling.
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Losses due to ground failure in Adapazari, Turkey

Table 5-4: Ground Failure Index (after Bray & Stewart, 2000)

<table>
<thead>
<tr>
<th>Index</th>
<th>Description</th>
<th>Interpretation</th>
</tr>
</thead>
<tbody>
<tr>
<td>GF0</td>
<td>No observable ground failure</td>
<td>No settlement, tilt, lateral displacement, or boils</td>
</tr>
<tr>
<td>GF1</td>
<td>Minor ground failure</td>
<td>Settlement, $\Delta &lt; 10$ cm; tilt of $\geq$ 3-storey buildings $\leq 1^\circ$; no lateral displacements</td>
</tr>
<tr>
<td>GF2</td>
<td>Moderate ground failure</td>
<td>$10$ cm $\leq \Delta &lt; 25$ cm; $1^\circ &lt; \text{tilt} &lt; 3^\circ$; small lateral displacements ($&lt; 10$ cm)</td>
</tr>
<tr>
<td>GF3</td>
<td>Significant ground failure</td>
<td>$\Delta &gt; 25$ cm; tilt $&gt; 3^\circ$, lateral displacements $&gt; 25$ cm</td>
</tr>
</tbody>
</table>

The damage survey data collected by Bray and Stewart (2000) included reinforced concrete (RC) frame buildings, timber frame with brick infill, timber and unreinforced masonry, as well as special buildings such as mosques. Only RC frame buildings are considered in this study; the damage distribution for the total number of RC frame buildings for which complete data were available is shown in Figure 5-15. These are divided into low-rise (1 to 3 storeys) and mid-rise (4 to 7 storeys): there were no high-rise buildings (over 7 storeys) in the survey zone.

Figure 5-15: Distribution of building damage to low-rise and mid-rise reinforced concrete frame buildings in central Adapazari. Damage scales as shown in Table 5-3.

RC frames are the focus of this study for the following reasons:

- To preserve clarity in presentation of the data and results.
- The available structural models, both from the TCIP methodology and the DBELA approach presented in Chapter 8, consider RC frames, eliminating the need for new structural analysis, which is not the focus of this thesis.
- RC frame apartment buildings, of typically 4 – 7 storeys, known as beskat in Turkish (literally five-storied) are one of the most common building classes in the affected region, therefore this selection permits comparison to other studies and other damage datasets.
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Figure 5-15 shows that buildings are approximately equally distributed between the two height classes, with more low-rise buildings in the lower damage states, but a greater number (81) of mid-rise buildings suffering either extensive or complete damage compared to low-rise (42).

In addition to the damage survey data of Bray and Stewart (2000), additional data have been presented by Bakir et al. (2002), collated by the Adapazari Municipal Authority, where damage is either classified as repairable (equivalent to light to moderate damage) or irreparable (equivalent to heavy damage or collapse). Although it contains less detailed information than the survey by Bray and Stewart (2000), this database includes almost 24,000 buildings and covers most of the city rather than just the central area, making it an additional useful resource. The Japanese Geotechnical Society (JGS) also carried out a detailed damage survey of 197 buildings in the zones where liquefaction was observed or where the damage was most intense (Yoshida et al., 2001). They used the EMS-98 damage scale (Grünthal, 1998) to classify building damage. EEFIT (2003) also present a survey of damaged buildings in central Adapazari, noting the building height, the damage level and the cause of damage.

5.3.4 Foundation vs. structural damage

An observation that arises in many of the damage survey reports, which has also been made in other documented examples of extensive liquefaction in urban areas, is that where buildings are displaced due to foundation movements, the superstructure damage tends to be relatively light, as for the building shown in Figure 5-1. Since the interaction of ground failure and ground shaking is of great importance for the prediction of losses in regions of ground failure, this issue is considered further in the light of the data from Adapazari.

The Turkish government data indicates that over 5000 buildings in Adapazari suffered either extensive or complete damage, whereas ‘hundreds’ settled, tilted or slid excessively (Bray & Stewart, 2000). This suggests that whilst receiving the most attention, it was not liquefaction that was the main cause of damage to buildings in Adapazari. Within the Adapazari survey zone, 25% of the buildings suffered heavy damage to collapse (D3 to D5) and only 14% suffered moderate to significant ground failure (GF2 to GF3, see Figure 5-16). Less than 1% of the surveyed buildings had clearly suffered both severe structural damage and severe ground failure. On the other hand, for various reasons, no ground failure index was assigned to approximately 26% of the total number of surveyed buildings. As can be seen in Figure 5-16, more than half of the buildings with no reported GFI had structural damage D4 or D5, i.e. heavy damage to collapse. This is significant, since one of the reasons for not being able to assign a ground failure rating was that heavy structural damage made it impossible to positively identify...
the occurrence of ground failure at the site. If there was some degree of ground failure beneath these buildings, the distribution shown in Figure 5-16 would alter considerably, and contrary to the above observation, there would be a large proportion of buildings with both severe structural damage and severe ground failure.

![Figure 5-16: Relative distribution of structural damage and ground failure damage for 719 buildings (Bray & Stewart, 2000, survey data), all building types and heights.](image)

Although it seems from damage surveys as well as photographs of damage in central Adapazari that there is a tendency for buildings on liquefied soil to be undamaged above foundation level, this observation could be misleading, due to the large number of completely collapsed buildings for which it was not possible to determine the extent of ground failure prior to collapse. On the other hand, there are more coincidences of low structural damage with high ground failure than high structural damage with high ground failure. Sancio et al. (2004) present a more detailed assessment of the interdependence of ground failure and structural damage in Adapazari.

Particular types of damage observed in Adapazari include lower-storey collapse, roof or top-storey collapse, damage due to torsion, damage of extension storeys or spans, failure of beam-column joints, and damage resulting from soil liquefaction or settlement (AIJ, 2001; EEFIT, 2003). This list clearly shows that liquefaction-related damage was not the only significant feature in Adapazari. Widespread liquefaction damage was, however, a feature unique to Adapazari amongst all the cities that suffered catastrophic damage in the Kocaeli earthquake.
Even without liquefaction, the strong shaking and soil amplification may have been sufficient to make Adapazari the worst affected city. This is an important feature of liquefiable regions; these soils may lead to significant damage through amplification even when liquefaction is not actually triggered. In this respect, the crude approach used in intensity based loss estimations such as ATC-13 (ATC, 1985) of simply increasing the intensity by one unit in regions of poor soil does have a high empirical justification, even if no theoretical backing.

5.3.5 Comparison of Adapazari to other survey data

Figure 5-17 compares the data for RC frame buildings with those collated by the Architectural Institute of Japan (AIJ, 2001) and used in the aforementioned study by Spence et al. (2003). These three zones, two in Gölcük and one in İzmit (see Figure 5-10 for locations) are comparable to Adapazari to some extent, although ‘Gölcük Hillside’ is on stiff soil, and the others are all on soft soil. All sites are in the near-fault region, with Gölcük being the closest to the fault, but directivity effects would have varied between the sites.

The right-hand column in Figure 5-17 shows the Mean Damage Ratio (MDR) a composite damage indicator that relates to the overall cost of the damage, assuming damage ratios of 2%
for slight damage, 10% for moderate damage, 50% for extensive and 100% for complete damage (after FEMA, 2003). These same loss ratios were used in previous studies for Turkey by Bommer et al. (2002a) and Spence et al. (2003) although it is important to note that they were developed for the United States, and have not been calibrated to Turkish economic conditions.

The hillside zone in Gölcük experienced the lowest level of overall damage, due mainly to the stiffer soils. In Adapazari the distribution was similar to the Gölcük coastal zone except that a larger proportion of undamaged buildings was recorded. This may be related to the occurrence of liquefaction and the fact that buildings with foundation failure but no structural damage were recorded as undamaged in the survey. Alternatively it could be a function of the different damage scales used for the different surveys and the subjective element of the rapid damage survey data. This latter explanation is supported by the observation that differences are less at the higher end of the scale; there is less ambiguity in assigning a building as ‘collapsed’. The mean damage ratio (MDR) is a relatively insensitive parameter to the different damage distributions, and it can be seen in Figure 5-17 that the MDR is quite similar for the three soft soil sites of Gölcük Coast, Izmit and Adapazari.

![Figure 5-17: Losses due to ground failure in Adapazari, Turkey](image-url)

**Figure 5-18:** Data from Figure 5-17 re-plotted with a modified damage scale.
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Losses due to ground failure in Adapazari, Turkey

The influence of subjectivity and damage scales is illustrated to some extent in Figure 5-18, where the data are re-plotted, with *undamaged* and *slightly* damaged buildings grouped together (the MDR is calculated assuming an average loss of 1% for this category). This eliminates some of the disparities between the different damage scales shown in Table 5-3, and also some of the subjectivity associated with the lower end of the scales. In Figure 5-18a, for low-rise RC buildings, there is now a relatively small difference between the three soft soil sites. In Figure 5-18b, for high-rise buildings, there remains a clear difference in the response of these buildings in each survey zone. This could be related to differences in building stock, soil response, near-fault and directivity effects, or a combination of all of these.

The shapes of the damage distributions also warrant further consideration. As discussed in Section 4.7.2, HAZUS-based fragility curves always produce the same shape of damage distribution, whereas it can be seen in Figure 5-18b that there are two different shapes of damage distribution, those with a single peak ('modal'), such as Gölcük Hillside and Izmit, and those with two peaks ('bi-modal') in the lower damage states and complete damage, with relatively few buildings falling between the two.

A hypothesis for the observed damage distribution having this 'bi-modal' shape could be is that it is actually made up of two superimposed independent distributions superimposed. For example, the damage due to ground shaking could be a modal distribution with its peak at *none* and decreasing thereafter, and the damage due to ground failure could have caused a number of buildings to collapse, and therefore by combining the two, a double peak would be obtained. However, because the damage ratings do not incorporate tilt or settlement, the most common form of failure due to ground deformations, this is unlikely.

### 5.4 Estimation of earthquake damage due to ground shaking

The procedure for estimating the damage to RC frame buildings in Adapazari as a result of ground shaking was based upon the capacity spectrum approach (see Section 4.7). The capacity curves for the TCIP project were from HAZUS, adjusted to Turkish conditions using both expert judgement and empirical calibration. The vulnerability parameters adopted are described by Bommer *et al.* (2002a), Spence *et al.* (2002) and Spence *et al.* (2003).

#### 5.4.1 Building classification

The TCIP building classification considered two levels of quality for Turkish RC frame buildings, *poor* and *good*. *Poor* buildings included those with weak ground floors, poor construction quality, or built before 1975. In the absence of sufficiently detailed information to
categorise the Adapazari survey data within these two groups, the predictions were carried out for both building classes. However, based upon the published descriptions of Adapazari building stock, and observations of its behaviour in 1999, where many soft-storey failures were observed, most RC frame buildings in central Adapazari were probably poor. This assumption agrees with the distribution assumed for urban locations in the TCIP study (Bommer et al., 2002a), where approximately 98% of RC buildings were categorised as poor.

The contribution of the unreinforced masonry infill walls commonly found in beşkat buildings to the lateral capacity of RC frame buildings has been discussed in Section 4.7.1. On the basis of the findings of Spence et al. (2003), their shear wall model, which included the contribution of the infill walls, has been used herein.

5.4.2 Demand curves

The demand curve is determined from the 5% damped elastic response spectrum for 5% damping, at PGA, 0.3s and 1.0s periods, in accordance with the HAZUS methodology. The PGA is also defined, as input for the ground failure component. The two approaches for the estimation of surface accelerations are described in Section 5.1.2. The design acceleration spectrum was constructed as follows (after HAZUS, FEMA, 2003):

\[
\text{at } T = 0 \quad S_A = PGA \\
S_D = 0 \quad (5-1)
\]

\[
0 < T \leq T_{AV} \quad S_A = S_A(T = 0.3s) \quad (5-2)
\]

\[
S_D = S_A \left( \frac{T}{2\pi} \right)^2 \quad (5-3)
\]

where \( T_{AV} = \frac{F_{dL}}{F_{AS}} \cdot \frac{S_{dL}}{S_{AS}} \) \quad (5-4)

\[
T_{AV} < T \leq T_{VD} \quad S_A \propto \frac{1}{T} = \frac{S_A(T = 1.0s)}{T} \quad (5-5)
\]

where \( T_{VD} = \frac{1}{f_c} = 10^{-5/2} \) \quad (5-6)

\[
T \geq T_{VD} \quad S_D = S_A(T = T_{VD}) \left( \frac{T_{VD}}{2\pi} \right)^2 = \text{constant} \quad (5-7)
\]

\[
S_A \propto \frac{1}{T^2} = S_D \left( \frac{4\pi}{T^2} \right) = S_A(T = T_{VD}) \cdot \frac{T_{VD}^2}{T^2} \quad (5-8)
\]
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Where: $T$ = response period (s)

$T_{AV}$ = period where constant acceleration changes to constant velocity

$T_{VD}$ = period where constant velocity changes to constant displacement

$S_a$ = spectral acceleration

$S_d$ = spectral displacement

$PGA$ = peak ground acceleration

$F_{AS}$ = soil amplification factor for short period ($T=0.3s$) acceleration

$F_{AL}$ = soil amplification factor for long period ($T=1.0s$) acceleration

$S_{AS}$ = short period acceleration, $S_a$ at $T=0.3s$

$S_{AL}$ = long period acceleration, $S_a$ at $T=1.0s$

$f_c$ = corner frequency

$M$ = moment magnitude of earthquake

The resulting spectrum is plotted in terms of acceleration against displacement (see Section 4.2.2), as shown in Figure 5-19. The constant displacement portion of the spectrum does not begin, in this case, until a period ($T_{VD}$) of approximately 15 seconds ($\approx 3$ m spectral displacement) and is therefore not relevant to the Adapazari building stock. The second approach for the estimation of surface accelerations described in Section 5.1.2 was preferred in this case, because of the improved representation of non-linear soil behaviour, using the NEHRP factors, which becomes important at longer periods.

![Figure 5-19: Demand curves for central Adapazari, at 7 km from fault rupture, for Mw=7.4, obtained by interpolating NEHRP factors for Site Classes D and E.](image)

Figure 5-19 compares the best estimate surface response spectra from this study to the surface response spectra computed by Sancio (2003) using the site response analysis program.
SHAKE91 (Idriss & Sun, 1991) plus soil profiles from their geotechnical investigations. Uncertainties in the former relate to the selection of the most appropriate predictive relationship, and how many standard deviations to incorporate as well as to the simplification of soil response. In the latter approach the uncertainties relate to the selection and scaling of input motions, soil variability, selection of appropriate dynamic soil parameters, and modelling assumptions (in this case equivalent linear). There is reasonable agreement between the two independently calculated spectra, with the exception that the smoothed spectrum does not show the amplification of up to three times the input motion between 2.0 and 3.0 seconds that is predicted using SHAKE (related to the natural frequency of the soil column used in these analyses). This difference is to be expected, since the smoothed spectrum is based upon only two amplification factors, at 0.3 and 1.0s.

The reason for retaining the simplified procedure here is for consistency with the approach used in loss estimation methodologies. The uncertainty associated with the specification of the demand curve in a loss model is unavoidable, but the correlation between this study and the independent results obtained by Sancio (2003) suggest a relatively low (but still significant) uncertainty in this case.

5.4.3 Shaking-induced losses

Using the approach described by Spence et al. (2003), including the building vulnerability model allowing for the lateral resistance contribution of infill walls for mid-rise RC frame buildings, damage distributions have been estimated for central Adapazari. The calculations were performed using a spreadsheet-based program developed at Cambridge Architectural Research Ltd. The variable input parameters and calculation method are briefly described in Appendix C.

The probability of exceedance of each damage state, from none to complete was calculated, and from this, the probability of a building being in a certain damage state. This probability is then equated to the percentage of buildings within each damage state. The MDR, as defined in Section 5.3.3, was also calculated in order to compare the relative overall losses for each scenario. The results of the comparison are shown in Figure 5-20 for different assumed vulnerability parameters.

There is a clear difference between the predicted and the observed numbers of undamaged buildings in both cases. Combining the lower damage states of none and slight as illustrated in Figure 5-18 for the observed damage would not reduce this difference, therefore it must be due
to more than just the uncertainties associated with the damage scales. The large number of buildings categorised as undamaged in Adapazari includes all those that have suffered some degree of ground failure but no associated structural damage; however, this is also true for the predicted losses, which are based upon the damage descriptions given on the right-hand side of Table 5-3. The shear wall vulnerability model for mid-rise RC buildings in Figure 5-20b shows the best agreement with the observations, whereby the number of undamaged buildings is under-predicted by 35%, the number of moderately damaged buildings is over-predicted by 40% and the number of completely damaged buildings is under-estimated by 15%. The difference between the predicted and observed MDR for this model is less than 2%, which is surprisingly good agreement. The good building model shows better agreement with the moderate and complete damage states, but worse overall agreement, as represented by the MDR.

Figure 5-20: Predicted versus observed damage in central Adapazari. Ground shaking only, RC frame buildings. Site class D/E, median demand values. a) low-rise, b) mid-rise.
Although the predicted MDR for the ‘shear wall’ model is close to the observations, it has been arrived at through an incorrect distribution of damage states, which warrants further attention. Possible explanations for the over-prediction of moderately and extensively damaged buildings and under-prediction of both undamaged and completely damaged buildings include:

1. The damage distribution in Adapazari being altered from the expected behaviour due to the presence of liquefied and liquefiable soils.

2. A disparity between the predicted damage states used in the model, and the definitions used in the field survey.

3. The fragility curves for this building type being incorrect, in terms of their shape or the relationship between drift levels and median damage state.

With respect to the first point, a number of investigators (e.g. Gülkan et al., 2003) have observed that structural damage was less common in areas of liquefaction. This phenomenon is sometimes referred to as a ‘base-isolation’ effect whereby the overall damage due to ground shaking is reduced because once liquefaction is triggered, it limits the intensity of ground shaking (Section 3.6.2). The greater number of undamaged and slightly damaged buildings in the field compared to the predictions could be related to such a feature. On the other hand, there are some important issues that do not support the base-isolation explanation. Detailed consideration of the performance of buildings on liquefied ground suggests some degree of interdependence between ground failure and structural damage due to ground shaking (Sancio et al., 2004). Furthermore, an important feature of the building stock in Adapazari, noted during the field survey and subsequent investigations, was the use of unusually robust reinforced mat foundations with intersecting grade beams (Sancio et al., 2004). Had the foundations been more commensurate with the general standard of construction in Adapazari (i.e. poor), it is likely that the structural damage observed in the liquefied zones would have been greater, since there would have been less rigid body rotation, and more structural distortion in response to the ground deformations.

Furthermore, liquefaction-induced base-isolation does not explain the observed high proportion of completely damaged buildings, which is more probably due to incorrect vulnerability parameters. The very large variations between results for the different vulnerability models used support this explanation. For example, the model for poor buildings significantly over-predicts the complete damage, so under-prediction is not a consistent feature. The shear-wall model, although improving some aspects of the vulnerability model, does not appear to correctly capture the brittle behaviour of the infill walls (Spence et al., 2003). Other
contributing factors could be the particular vulnerability of the mid-rise RC frame buildings in Turkey due to their poor construction, often with added-on storeys, the fact that they frequently have open ground floors, thus creating a soft storey, and the coincidence of the buildings' natural periods with the dominant period of the ground shaking. The combination of these and possibly other factors may well have contributed to the skewed distribution of observed damage. The difference between the smoothed and detailed surface response spectra in Figure 5-19 is a source of additional uncertainty.

Differences relating to the damage scales (point 2 above) used undoubtedly contribute to the overall uncertainty of the results, particularly for slight, moderate and extensive damage, which would be the most difficult to evaluate visually. However, the variance between the two scales shown in Table 5-3 means that the survey data would, if anything, be underestimating the number of undamaged buildings, so adjusting this would not lead to any better agreement with the predictions.

As has been explained previously, both the observed and predicted damage shown in Figure 5-20 refer only to the traditional damage state descriptions for ground shaking-induced damage such as cracking and failure of structural members. If a more appropriate damage scale were used, which rated settled or tilted buildings as well as cracked or collapsed buildings then the observed distribution in Figure 5-20 would change, and a number of the ‘undamaged’ buildings would be redistributed into the higher damage levels. However, since the predicted damage has been calculated using the capacity spectrum approach, it is correct at this stage to consider only the traditional drift-related damage descriptors.

With respect to the third point above, the shape of the damage distributions for the survey data was briefly discussed in Section 5.3.3. Without altering the shape of the HAZUS fragility curves, for example by making them lie closer together, it would be impossible to obtain agreement between the predicted and observed damage states. This issue has been explored further in a study by Booth, Bird & Spence (2004), where the bi-modal shapes of the distributions were replicated, using Monte Carlo simulations of both capacity and demand, with the capacity curve modified to represent brittle failure (Figure 4-14). For more ductile buildings, the modal shape was produced. This study therefore suggested that a very important factor in the distribution of damage in the Kocaeli earthquake may have been the brittle behaviour of the RC frame buildings, caused either by abrupt failure of the unreinforced masonry walls, or soft-storey collapses. The displacement based earthquake loss assessment methodology (DBELA) developed by Pinho and co-workers (Section 4.8), by approaching the building response to earthquakes in a more mechanically sound manner than the pushover
curves, is found to be better suited to the modelling of the behaviour of poorly confined buildings. Particular features that would cause the bi-modal distribution in some areas but not in others (such as Izmit, Figure 5-21), that would serve as the basis for modifying the fragility curves, are not fully defined, and are outside the scope of this thesis.

The conclusions of this first stage of the study are that:

- The observed damage distribution in Adapazari was significantly more complex and involved a greater number of variables than the estimates obtained using the HAZUS based approach for ground-shaking only.

- The occurrence of liquefaction in the Adapazari survey area has modified the building response, and to ignore liquefaction in such a small zone will lead to incorrect prediction of the damage distribution, although there are other uncertainties related to the ground-shaking component that will also contribute to this.

![Graph showing observed damage distributions](image)

**Figure 5-21:** Observed damage distributions for individual survey zones on soft soils between 1 and 7 km from the fault rupture, compared to the average damage distribution for all sites. Mid-rise RC frame buildings.

### 5.5 Estimation of earthquake damage due to liquefaction

Chapter 4 explained the various published methodologies for the incorporation of ground failure into a model. In summary, the options available to loss modellers are to:

i. Ignore liquefaction completely, assuming that ground shaking will be the predominant cause of damage.
ii. Use a simplified incremental approach to represent the impact of liquefaction, such as that adopted in the TCIP study (Bommer et al., 2002a), whereby site class E was assigned to all zones susceptible to liquefaction, thus subsuming the effects of liquefaction into an artificial incremental increase in ground shaking in most areas. Other methods such as ATC-13 apply increments to the predicted damage rather than the demand.

iii. Use the HAZUS methodology, possibly making adjustments to reflect local conditions.

iv. Use in situ geotechnical data and detailed inventory data to carry out full analyses of the probability of liquefaction, the expected permanent ground deformation and the resulting damage to buildings.

The results presented in the previous section of this chapter used the first option, and it was concluded that the occurrence of liquefaction was a possible explanation for the poor performance of the model. The second and third options are now considered to assess to what extent they improve the predictions for Adapazari.

The method presented by Kiremidjian (1994), described in Section 4.9 cannot be used in this case, since the damage in Adapazari was almost universally due to ground settlement not lateral spreading.

5.5.1 Simplified ‘incremental’ methodology

The approach for incorporating ground failure used in the TCIP methodology is described in Section 4.9. The soil deposits in Adapazari are identified as being potentially liquefiable, being recent alluvial deposits, with a high water table. The TCIP model would therefore have assigned class E to the central area.
**Figure 5-22:** Predicted damage in central Adapazari using the shear wall model for site classes D/E and E (FEMA, 1997) compared to observed damage for mid-rise RC frames.

As shown in Figure 5-22, changing the demand from site class D/E to E, makes only a small difference (the MDR increases by 8%), and does not produce a closer estimate of the actual damage distribution. The differences are essentially the same as those discussed in the preceding section; hence the same shortcomings prevent any conclusion being reached.

A further observation related to the TCIP approach is that the NEHRP factors for site class E reflect the non-linearity of loose or weak soils, such that at the highest levels of shaking, the shaking, and therefore the predicted damage, is de-amplified rather than amplified. The extent and depth of liquefaction, and therefore of liquefaction-induced damage, will however continue to increase as the strength of shaking increases.

By inspection of the differences between the observed and predicted damage distributions, it can be concluded that any simple approach that seeks to adjust the demand or the damage ratios to represent the damage due to liquefaction cannot produce satisfactory results unless the ground-shaking component of the model is unable to do so.

The ATC-13 methodology, shown in Equations 4-5 and 4-6 and Table 4-4, adjusts only the mean damage factor (MDF). The intensity in Adapazari was between EMS VIII and IX (EEFIT, 2003), so, according to the ATC methodology, the probability of ground failure in Adapazari, assuming Holocene Alluvium soils, would be 20 to 30%, resulting in an MDF due to ground failure of 1.25 (0.25 x 5) times that due to ground shaking, the final value being the sum of the two, or 2.25 times the ground-shaking induced MDF. For the data presented in this chapter, it is the damage distribution that is of concern, which the ATC-13 approach would not improve by only considering the MDF.
5.5.2 HAZUS methodology

The default methodology used in HAZUS (rather than the expert generated approach) has been presented in Section 4.10. For this case study, the steps shown in Figure 4-16 have been applied without modification as summarised in Figure 5-23 and described further below.

*Step 1 Liquefaction susceptibility*

The deposits underlying central Adapazari are predominantly recent river channel and flood plain deposits. In fact the name translates as ‘Island Market’ and the city used to lie on an island between the Sakarya and Cark rivers. According to Youd and Perkins (1978) (Table 2-5), the susceptibility of cohesionless recent river channel and flood plain deposits is *very high* and *high* respectively. In fact the deposits in Adapazari had high fines contents, and their classification as cohesionless is debatable. However, within the HAZUS framework, such detailed information would probably not be available. Based upon knowledge of the actual ground conditions, and in order to illustrate the maximum losses that HAZUS would predict, the soil has been categorised as *very high*.

![Figure 5-23: Application of the HAZUS ground failure methodology to central Adapazari](image-url)
Chapter 5  
Losses due to ground failure in Adapazari, Turkey

Step 2  Proportion of map unit, $P_{ML}$

For deposits with a very high susceptibility, the proportion of the unit deemed likely to liquefy is 0.25. Observations in Adapazari supported the assumption that liquefaction is not uniform. In the survey data of Bray and Stewart (2000) it is noted that approximately 40% of all the buildings had a ground failure index of GF0, meaning that there was no evidence of liquefaction, and for a further 30% approximately there was no observation regarding ground failure. Although very approximate, these numbers suggest that the factor of 0.25 may be in the right range, although possibly too low, since these data suggest that between 30 and 60% of the total surveyed buildings experienced some liquefaction. However, when it is considered that the survey only covered four lines in the central area, where there was particularly strong evidence of liquefaction, the 0.25 factor is judged to be a reasonable approximation. Consideration of the range and uncertainty associated with the $P_{ML}$ factor, as noted in 4.10, would be more appropriate..

Step 3  Conditional probability of liquefaction

The best estimate of peak ground acceleration for the study area is 0.4g. Other parameters input to this calculation are the magnitude ($M_w=7.4$) and the ground water level, assumed to be 1.5m below ground level (4.9 ft). From Figure 4-17 and Equation 4-8, the conditional probability of liquefaction can be obtained as follows:

$$P_{[\text{Liquefaction}]} = \frac{P_{[\text{Liquefaction}_{very\,high}]}|PGA=0.4g}{K_m \cdot K_w} \cdot 0.25$$  \hspace{1cm} (5-9)

$$P_{[\text{Liquefaction}_{very\,high}]}|PGA=0.37g = 9.09 \times 0.37 - 0.82 = 2.54 \geq 1.0$$  \hspace{1cm} (5-10)

$$\therefore P_{[\text{Liquefaction}]}|PGA=1.0$$

$$K_m = 0.0027(7.4)^3 - 0.0267(7.4)^2 - 0.2055(7.4) + 2.9188 = 1.03$$  \hspace{1cm} (5-11)

$$K_w = 0.022 \cdot 4.9 + 0.93 = 1.04$$  \hspace{1cm} (5-12)

$$\therefore P_{[\text{Liquefaction}]} = \frac{0.25}{1.03 \cdot 1.04} = 0.23$$

Step 4a Calculation of lateral spread

The threshold acceleration at which liquefaction may be triggered, PGA(t), for very high susceptibility soils is 0.09. Therefore the PGA normalised to the threshold value is 4.11. The
expected lateral spreading for normalised accelerations greater than 0.4 is 100 inches (Figure 4-18), modified for magnitude using the equation:

\[ K_\Delta = 0.0086M^3 - 0.0914M^2 + 0.4698M - 0.9835 = 0.97 \]  

(5-13)
i.e. 97 inches, or 2.46 m.

**Step 4b Calculation of settlement**

The amplitude of expected settlement, for very high susceptibility, is 12 inches, or 0.305 m (Table 4-7). This information is not available per se for the survey zone, however, the ground failure index, defined in Table 5-4, gives the range of permanent ground deformation beneath buildings, plotted in Figure 5-24. 0.305m of settlement would be equivalent to GF3, and two comments can be made with respect to this comparison. Firstly, it is clear that there was significant variability within the degree of ground failure in Adapazari, even within the small survey zone, that cannot be sufficiently represented by a single value. Therefore even if the susceptibility category has been wrongly assigned in this study, the other values in Table 4-7 would not improve upon this. Secondly, the values used in HAZUS are based upon free-field calculations, whereas the data presented in Figure 5-24 are obtained from building foundations. The angle of tilt is an important variable that cannot be estimated using free-field volumetric strain approaches, and furthermore, the free-field assumption ignores the influence of bearing pressures on the foundation settlement.

![Ground Failure Index](image)

**Figure 5-24:** Ground failure distribution for all surveyed buildings (Bray & Stewart, 2000, survey data)

**Step 5 Damage estimation**

The final estimation of damage is based upon the fragility curves shown in Figure 4-20. Although foundation information is not available for most of the surveyed buildings, it is understood that only one of the RC frame buildings was piled; therefore only shallow
foundations were considered. Figure 5-25 shows the probability of extensive or complete damage as a function of the expected permanent ground deformation; as noted previously, in HAZUS it is assumed that buildings are either undamaged, or else suffer extensive or complete damage due to ground failure.

According to the HAZUS manual (FEMA, 2001) the more damaging scenario of lateral spread or settlement is used; from Figure 5-25 it can be seen that this would be lateral spread. In this case it is known that lateral spread was a rare occurrence in Adapazari, because there were few free faces in liquefiable soils, close to buildings (R. Sancio, Pers. Comm. 2004). Additionally, the fine-grained nature of the liquefiable soils in Adapazari does not generally lead to large displacements. Therefore, although the purpose of this study is to use the HAZUS methodology without modification, in this step the probability of settlement-induced damage has been used. A more appropriate calculation than HAZUS should consider the presence and nature of free faces for the lateral spreading induced damage. Strict application of the methodology, rather than applying judgement to select the observed form of ground deformations, would have in fact predicted a higher possibility of lateral spread-induced failure than settlement.

\[
\begin{align*}
\text{Figure 5-25: HAZUS fragility curve for shallow foundations on ground failures as shown in Figure 4-18. Lines added for predicted lateral spread and settlement in central Adapazari from Step 4 described above.}
\end{align*}
\]

\[
P(E \text{ or } C) \text{ for settlement (0.56, from Figure 5-25) is then multiplied by the conditional probability of liquefaction (0.23, Step 3 above), the final value of 0.13 being the combined probability of liquefaction occurring and causing extensive or complete damage.}
\]

Where buildings are extensively or completely damaged by liquefaction, it is assumed that 20% of these are completely damaged, the other 80% being extensively damaged (FEMA, 2003). The distribution of building damage due to liquefaction only is shown in Figure 5-26. The two
building categories in Figure 5-26 are based upon the survey data only, since there is no distinction made in the HAZUS methodology for either building height or building type.

Figure 5-26: Predicted damage distribution for liquefaction-induced settlement, applying the HAZUS default methodology to the Adapazari survey data.

The final step in the HAZUS methodology is to combine the damage due to ground shaking with the damage due to ground failure, using Equation 4-14. The results are shown in Figure 5-27.

Figure 5-27: Predicted damage in central Adapazari due to both ground shaking and liquefaction. a) Low-rise RC frame buildings, poor quality. b) Mid-rise RC frames, shear walls. Site class D/E assumed for ground shaking demand.
5.6 Discussion of results

An important finding from this study is that the structural damage scale used is not appropriate when ground failure is incorporated. If all buildings that suffered large settlement or tilt were rated as extensively or completely damaged, both in the survey and in the predictions, a number of buildings in the lower categories would be redistributed into higher categories for the observed data. On the basis of this case study damage scales have emerged as a fundamentally important issue to be resolved for the development of an improved methodology for estimating losses due to liquefaction.

The larger proportion of undamaged RC frame buildings in central Adapazari compared to survey zones in similar cities such as Gölçük and Izmit, and the lower proportion of slight and moderately damaged buildings, has a number of possible explanations. These include subjectivity in the interpretation of damage scales by different field surveyors and differences between site conditions and source effects between the different regions. The shortcoming in currently available damage scales noted above may also be a contributing factor.

Three different approaches have been tested for the incorporation of liquefaction, namely: ignoring it; assigning the worst site class to liquefiable soils; or using the HAZUS default methodology. These results, summarised in Table 5-5, suggest that the additional data, analysis and time required to incorporate liquefaction-induced damage using the simplified HAZUS methodology in this particular case is not justified. The predicted damage is more sensitive to the building vulnerability model used to estimate the damage due to ground shaking than to whether liquefaction is incorporated or not. Furthermore, there is no evidence, on the basis of this case study, that the HAZUS approach is in fact modelling liquefaction-induced damage correctly. This creates a situation where although ground failure is included in a loss model, this component has such a high uncertainty, most of which is hidden to the user and not explicitly modelled, that it does not improve the validity of the results in zones susceptible to liquefaction at all.

Table 5-5: Comparison of different loss models for central Adapazari, showing the difference between the predicted and observed damage for mid-rise RC frame structures on soil class D/E.

<table>
<thead>
<tr>
<th>Model for Liquefaction</th>
<th>Vulnerability model</th>
<th>Difference to observed damage states</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ignored</td>
<td>Good</td>
<td>None: -35% Moderate: +14% Complete: -5% MDR: +28%</td>
</tr>
<tr>
<td>Ignored</td>
<td>Shear wall</td>
<td>None: -30% Moderate: +40% Complete: -15% MDR: -1%</td>
</tr>
<tr>
<td>Site class E</td>
<td>Shear wall</td>
<td>None: -32% Moderate: +35% Complete: -11% MDR: +7%</td>
</tr>
<tr>
<td>HAZUS</td>
<td>Shear wall</td>
<td>None: -30% Moderate: +34% Complete: -13% MDR: +4%</td>
</tr>
</tbody>
</table>
The effect of adding liquefaction to the damage prediction following the HAZUS approach is to move a number of buildings from the damage states *none, slight* and *moderate* into *extensive* and *complete*. In Figure 5-27b, the predicted proportion of completely damaged buildings is improved with the combined model, but generally, the effect is detrimental in terms of agreement with observations, rather than beneficial.

Although liquefaction was a significant feature in central Adapazari, it is far from clear, on the basis of this case study, that it had the effect of making the overall damage worse than if it had not occurred. If it were possible to accurately predict losses due to building damage, then it would be possible to determine exactly how much difference the occurrence of liquefaction in Adapazari made, simply by analysing one case with, and one without, the liquefaction component. However, as was shown by Spence *et al.* (2003) and Booth *et al.* (2004) and reiterated in this chapter, the results of the current displacement-based damage estimation approaches are still questionable in their reliability, and for the Kocaeli earthquake it was not possible to reproduce the observed damage distributions. Therefore, the findings presented in this chapter have to be interpreted in the light of this fundamental uncertainty. Specifically, Spence *et al.* (2003) and Booth *et al.* (2004) highlighted uncertainties associated with the simplified pushover model for structural behaviour, and its poor representation of the real response of buildings.

As with all case studies, there are some peculiarities of the Adapazari dataset that make interpretation of the results more difficult. Two significant features in this case were the uniformity of the foundations, and the fine-grained nature of the soil. The foundations, being unusually stiff, certainly dominated the building response to liquefaction, and more variety in this respect may have enabled some tentative conclusions to made as to the causes of different damage distributions. The importance of the foundation type in ground failure zones is not given significant precedence in the HAZUS (FEMA, 2003) methodology. In Adapazari, the foundations were robust, not commensurate with the poor quality of the superstructures and this factor undoubtedly influenced the discrepancies between predicted and observed damage. Unfortunately (from the point of view of this research) the foundations in Adapazari were also fairly uniform across the building stock and therefore there was no opportunity to examine different foundation responses from the data. The observed damage may well have been much closer to the predictions had the foundations been weak and susceptible to failure due to differential settlement. With respect to the fine-grained nature of the soils, although liquefaction was a widespread occurrence in Adapazari, the detailed sub-surface investigation revealed that the liquefied soils were more fine-grained than most liquefiable deposits tend to
be. This feature is not well represented by most empirical datasets, and therefore many of the methodologies employed within the HAZUS ground-failure component will be additionally uncertain when applied to liquefiable soils with high fines contents.

In considering the results of this study, it should be re-emphasised that the HAZUS methodology for liquefaction has been developed specifically for application within the United States, and most of the empirical relationships used in the methodology are derived from US data. Nonetheless, such relationships are frequently adopted in Turkey in the absence of methodologies specifically derived for Turkish conditions.
6 SITE AMPLIFICATION AND LIQUEFACTION HAZARD EVALUATION FROM IN SITU DATA

Although the HAZUS (FEMA, 2003) default methodology simplifies the input data as far as possible to a single map of liquefaction susceptibility, there is still considerable work required to produce this map. By estimating building damage due to ground failure without any in situ geotechnical data as input, particular features of local soils, such as the fine-grained nature of some Adapazari soils, are ignored. Furthermore, by assuming constant values for the penetration resistance of different mapped soil categories, the depth and thickness of the liquefiable soils, and single relationships for estimating the permanent ground deformations, the inherent scatter in each of these quantities is excluded. An alternative approach, presented in this chapter, is to use as much site-specific geotechnical data as feasible within the confines of a regional model. This greatly increases the time and effort required, but avoids the need for major simplifications that may not be appropriate to the study area.

Geological and geotechnical data from the Hong Kong region are used. The primary purpose of the chapter is to illustrate a methodology that uses a good geotechnical database for the study area. Of particular relevance to the topic of this thesis is the potential for the coastal reclamation fill and natural coastal deposits to liquefy. Due to the small land area and the mountainous environment, there has been extensive coastal reclamation in Hong Kong since the mid 19th century in order to maximise the developable land area.

This chapter presents a brief summary of the seismicity in Hong Kong, and describes an assessment of site amplification effects, undertaken by the author during a secondment to Arup Hong Kong as part of this PhD programme. Subsequent sections (6.3 to 6.6) describe the assessment of liquefaction hazard as input to a regional loss estimation. The findings and interpretations presented in these sections are those of the author, not of Arup. The objective of the study is to consider the liquefaction component of earthquake loss estimations in the light of available data, not to present a definitive assessment for the region, and it should not be taken to define seismic risk in Hong Kong.

6.1 Background

There is a growing awareness that regions of low to moderate earthquake hazard may face large potential losses. For example, the city of New York has experienced only a few earthquakes in its history, of moderate magnitude (M<5.5), and as such can be considered a low to moderate
hazard area, which is reflected in US seismic design codes. However, with assets valued at approximately US$1 billion ($1 \times 10^{12}$) (Nordensen et al., 2000), New York faces extremely high potential losses in the event of a damaging earthquake.

The Hong Kong Special Administrative Region of China (Hong Kong), shown in Figure 6-1, is in a region of low to moderate seismicity and has a potentially very high exposure due to its large population density, its exceptional concentration of high rise buildings, and its important economic status. Hong Kong is a 'mega-city', with approximately 6.8 million inhabitants (CIA, 2004) over a total area of 1092 km$^2$ of which 1042 km$^2$ is land (comparatively, Greater London has a population of 7.2 million, from 1998 figures, and an area of 1580 km$^2$). In fact it is one of the most densely populated areas in the world (Lee et al. 2000). Furthermore, it holds an important position in the world’s financial markets, as well as having a major container port and manufacturing facilities.

Hong Kong is situated in a stable continental intraplate region, meaning it is far from any tectonic plate boundaries and there is little or no crustal deformation taking place. The nearest plate boundary is approximately 700 km distant, underlying Taiwan, and there is no evidence of fault activity in the immediate vicinity of Hong Kong over the past 80 000 years (Free et al., 2004). There are no historical records of earthquakes greater than magnitude 5 affecting Hong Kong (Lee et al., 2000), although within 500 km there have been three earthquakes of magnitude greater than 7, in 1604, 1605 and 1918.
The cause of earthquakes in regions distant from plate boundaries is less clearly defined than in regions of active tectonic deformation. Nonetheless, damaging earthquakes do occur in intraplate regions; it is thought that these are generally due to the relief of stresses caused by distant plate interactions, with the stress relief likely to occur in zones of weakness such as pre-existing faults (Atkinson, 2004). Two significant features of intraplate earthquakes are that the seismicity is likely to be diffused over wide areas, rather than delineated, and that there is rarely surface fault rupture associated with these earthquakes.

A probabilistic seismic hazard assessment (PSHA) for Hong Kong has recently been published by Free et al. (2004): the results of this evaluation are used as the basis for the analyses presented in this chapter. For regions of low seismicity, PSHA is more commonly used than DSHA due to the difficulties associated with defining characteristic earthquakes in a deterministic manner where the seismicity is diffuse and not easily identifiable with particular faults. Epistemic uncertainties, associated with incomplete knowledge, are significant for the Hong Kong area; its low historical seismicity means there is little information regarding the location, size and frequency of earthquake occurrence, therefore the logic tree approach used by Free et al. is a suitable tool.

The results of the PSHA are presented in terms of bedrock acceleration, where bedrock was defined as NEHRP site class A (FEMA, 1997), or ‘Hard Rock’, with a shear wave velocity of 1500 m/s, in accordance with measured data from Hong Kong. Three probabilities of exceedance were considered, 50% in 50 years, 10% in 50 years and 2% in 50 years, where:

\[ q = 1 - e^{-LN} \]  
\[ T_r = \frac{1}{N} \]  

and:  
q = probability of exceedance  
L = design life (years)  
N = annual frequency of occurrence  
Tr = recurrence interval (years)

Thus for the aforementioned probabilities of exceedance in 50 years, the recurrence intervals are 72, 475 and 2475 years respectively. The results of the PSHA are presented in terms of uniform hazard response spectra (UHRS) as shown in Figure 6-2.
The disadvantage of PSHA results in not yielding a single, scenario earthquake for design have been discussed in Section 2.6.4. In this case, two magnitude-distance-standard deviation combinations were obtained through disaggregation. For the 2475 year recurrence interval, these scenario earthquakes comprised a magnitude 6.3 earthquake at 50 km (median +1.2 σ), and a magnitude 7.5 at 300 km (median +1.15 σ) for the short (0.2s) and long-period (2.0s) portions of the UHRS respectively.

6.2 Site classification, zoning and soil amplification

The subsoil conditions have been categorized across the Hong Kong region according to the NEHRP site categories (Pappin et al., 2004).

6.2.1 Geology

The following summary description of the geology of Hong Kong is provided by Pappin et al. (2004):

The geology of Hong Kong is described by Sewell et al. (2000) and Fyfe et al. (2000) and is summarized on (Figure 6-3). More than three-quarters of the land area of Hong Kong is underlain by igneous rocks predominantly volcanic tuffs and granites of Late Jurassic to Early Cretaceous age (140 to 120 Ma). Older, Late Paleozoic age (420 to 240 Ma) sedimentary rocks and younger Late Mesozoic to Tertiary age (140 to 2 Ma) sedimentary rocks underlie the majority of the remaining land area. Superficial deposits comprising Quaternary age (less than 2Ma) alluvium and other unconsolidated deposits are also present throughout the territory. The Quaternary age deposits in Hong Kong comprise alluvium and colluvium materials deposited throughout the Middle and Upper Pleistocene and Holocene periods, covering a period of approximately 200,000 years. The sediments generally overlie the bedrocks, which have weathered mantle of highly variable thickness. These weathered profiles are typically a few metres to several tens of metres but can exceed 200m locally (Fyfe et al., 2000). Large reclamation areas are also present in Hong Kong (see Figure 6-3), with the history of
reclamation extending back to before 1887. These deposits comprise sand fill, mixed rock and soil fill as well as construction and demolition debris and are typically quite variable both laterally and vertically.

Figure 6-3: Simplified geological map of Hong Kong (Fyfe et al., 2000, taken from Pappin et al. 2004)

The history of the development of reclamation in Hong Kong is shown in Appendix D.

6.2.2 Geotechnical investigation data

The ground investigation data employed in this chapter is selected from a database of over 1200 boreholes collated by Arup Hong Kong (Pappin et al., 2004). As would be expected, the distribution of the boreholes coincides with the built-up areas. Data comprised soil profile descriptions and standard penetration test (SPT) results and in a number of cases shear wave velocity profiles, measured using downhole, crosshole, seismic cone or spectral analysis of surface waves (SASW) techniques.

6.2.3 Regional site classification

The NEHRP categories (FEMA, 1997) are presented in Table 2-2 of this thesis. Although this classification system was developed for use in the United States, it is frequently adopted for studies outside the US. The main requirements of a classification system are to provide a logical set of criteria from which to zone study areas, and to group similar soil profiles, such
that they can be represented by single amplification factors. The approach adopted was to use the NEHRP classification system, and then incorporate any uncertainty and variability related to the site amplification, including the deep geology, into new site amplification factors for each site class. This is considered by the author to be appropriate even though it could be argued that the NEHRP system has no relevance to conditions in Hong Kong.

Figure 6-4: Site classification zoning map for Hong Kong (Pappin et al., 2004)

Site class boundaries were found to broadly coincide with the geological boundaries across the territory (Pappin et al., 2004). For example, the boundary between site class D and site class C broadly agrees with the boundary between Quaternary and Pre-Quaternary materials. The boundary between site classes C and B was found to be related to the presence of a layer of residual soil over the bedrock. This layer was generally present on shallower slopes, and absent where the gradient was steeper. Site class E, as would be expected, coincided with areas of thick reclamation fill or alluvial or marine deposits.

6.2.4 Soil amplification factors

The NEHRP amplification factors are based primarily on Californian data (Dobry et al., 2000). Table 6-1 compares the seismicity, in terms of spectral accelerations, used in the NEHRP guidelines with that of Hong Kong presented by Free et al. (2004). The NEHRP guidance presents a single amplification factor for all accelerations below the minimum value.
Table 6-1: Comparison of Hong Kong short- and long-period bedrock spectral accelerations, at 0.2 s and 1.0s periods respectively, with the ranges used for the NEHRP amplification factors (Tables 2-3 and 2-4).

<table>
<thead>
<tr>
<th>Probability of Exceedance in 50 years</th>
<th>Spectral Accelerations (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$S_{AS}$ Hong Kong</td>
</tr>
<tr>
<td>50%</td>
<td>0.08</td>
</tr>
<tr>
<td>10%</td>
<td>0.23</td>
</tr>
<tr>
<td>2%</td>
<td>0.48</td>
</tr>
</tbody>
</table>

In order to evaluate the regional soil amplification in Hong Kong, a number of one-dimensional site response analyses were undertaken for a range of typical soil profiles and input parameters. The results were used to determine ground-motion amplitude- and period-dependent factors for each site class. These factors have the advantage of being specific to Hong Kong in terms of the local soil conditions and the amplitude of the reference input motion. Furthermore, by developing new regional site amplification factors, the uncertainty associated with them can be evaluated.

### 6.2.5 Site response analysis - method

The site response effects were analysed using the following steps:

1. A suite of representative soil profiles was compiled to represent the range of ground conditions encountered in Hong Kong (Table 6-2). These are defined in terms of the soil type and the small strain shear modulus ($G_0$) versus depth. For each soil type the shear modulus degradation curve and the density are defined. In each case the profile extended to bedrock, defined as moderately decomposed or Grade III weathered rock.

2. Disaggregated bedrock response spectra were used to define appropriate earthquake strong-motion records for input as reference motions. Input motions were obtained by selecting one time history for each scenario with the best possible agreement in terms of magnitude and distance, spectral shape and amplitude, and then modifying these records using the program SYNTH in order to match them to the target spectra. SYNTH (courtesy of Arup Geotechnics) operates by iteratively adjusting an input time history to match a target spectrum in the frequency domain. By employing reasonably stringent selection criteria for the input motions, the amount of adjustment required can be minimised.

3. One-dimensional site response analyses were carried out using the program SIREN (Section 2.7.4). A sample input and output file for SIREN is presented in Appendix E.
Surface time histories were computed in SIREN and converted to 5% damped elastic response spectra.

4. Spectral ratios were determined for each soil profile, from the ratio of the surface to the input bedrock spectra.

**Table 6-2:** Hong Kong profiles selected for site response analysis and liquefaction assessment.

<table>
<thead>
<tr>
<th>Profile Name</th>
<th>Depth to bedrock (m)</th>
<th>Location</th>
<th>NEHRP Site Class</th>
<th>Liquefaction assessment</th>
</tr>
</thead>
<tbody>
<tr>
<td>C1</td>
<td>18</td>
<td>Mid-levels</td>
<td>C</td>
<td>x</td>
</tr>
<tr>
<td>C2</td>
<td>15</td>
<td>Hong Kong Island – Victoria Peak</td>
<td>C</td>
<td>x</td>
</tr>
<tr>
<td>D1</td>
<td>55</td>
<td>Hong Kong Island, Central District</td>
<td>D</td>
<td>✓</td>
</tr>
<tr>
<td>D2</td>
<td>26</td>
<td>New Territories, Yuen Long</td>
<td>D</td>
<td>✓</td>
</tr>
<tr>
<td>D3</td>
<td>44.5</td>
<td>Hong Kong Island, Central</td>
<td>D</td>
<td>✓</td>
</tr>
<tr>
<td>D4</td>
<td>45</td>
<td>HK Island Victoria Peak</td>
<td>D</td>
<td>x</td>
</tr>
<tr>
<td>D5</td>
<td>46</td>
<td>HK Island Victoria Peak</td>
<td>D</td>
<td>x</td>
</tr>
<tr>
<td>D6</td>
<td>84</td>
<td>HK Island Mid-levels</td>
<td>D</td>
<td>✓</td>
</tr>
<tr>
<td>D7</td>
<td>24</td>
<td>HK Island Mid-levels</td>
<td>D</td>
<td>x</td>
</tr>
<tr>
<td>D8</td>
<td>84</td>
<td>HK Island Mid-levels</td>
<td>D</td>
<td>x</td>
</tr>
<tr>
<td>D9</td>
<td>60</td>
<td>HK Island Central</td>
<td>D</td>
<td>✓</td>
</tr>
<tr>
<td>D10</td>
<td>52</td>
<td>Kowloon Bay</td>
<td>D</td>
<td>✓</td>
</tr>
<tr>
<td>D11</td>
<td>130</td>
<td>Tin Shui Wai</td>
<td>D</td>
<td>✓</td>
</tr>
<tr>
<td>DE1</td>
<td>78</td>
<td>HK Island Mid-Level</td>
<td>D/E</td>
<td>x</td>
</tr>
<tr>
<td>DE2</td>
<td>75</td>
<td>Hong Kong Island Central</td>
<td>D/E</td>
<td>✓</td>
</tr>
<tr>
<td>DE3</td>
<td>55</td>
<td>Hong Kong Station, Central</td>
<td>D/E</td>
<td>✓</td>
</tr>
<tr>
<td>DE4</td>
<td>42</td>
<td>Tsing Yi</td>
<td>D/E</td>
<td>x</td>
</tr>
<tr>
<td>E1</td>
<td>45</td>
<td>Kowloon Bay</td>
<td>E</td>
<td>✓</td>
</tr>
<tr>
<td>E2</td>
<td>30</td>
<td>Kowloon</td>
<td>E</td>
<td>✓</td>
</tr>
<tr>
<td>E3</td>
<td>40</td>
<td>Kowloon</td>
<td>E</td>
<td>✓</td>
</tr>
<tr>
<td>E4</td>
<td>95.5</td>
<td>Ma On Shan</td>
<td>E</td>
<td>✓</td>
</tr>
<tr>
<td>E5</td>
<td>55</td>
<td>Tseung Kwan O</td>
<td>E</td>
<td>✓</td>
</tr>
<tr>
<td>E6</td>
<td>40</td>
<td>Yuen Long</td>
<td>E</td>
<td>✓</td>
</tr>
</tbody>
</table>

Selected shear modulus degradation curves are presented in Table 6-3. The uncertainty associated with the selection of a degradation curve for Hong Kong soils, in the absence of extensive experimental data available for Hong Kong, was compensated for by including sensitivity checks in the suite of analyses.
### Table 6-3: Summary of published and unpublished site response studies for Hong Kong, and the G/G₀ curves adopted in these studies

<table>
<thead>
<tr>
<th>Reference</th>
<th>Sand Fill</th>
<th>Marine Deposits</th>
<th>Alluvial Clays</th>
<th>Alluvial Sands</th>
<th>Colluvium</th>
<th>CDG</th>
</tr>
</thead>
<tbody>
<tr>
<td>Chandler &amp; Su (2000)</td>
<td>(1)</td>
<td>-</td>
<td>-</td>
<td>(1)</td>
<td>-</td>
<td>(1)</td>
</tr>
<tr>
<td>Wong et al. (2001)</td>
<td>(2)</td>
<td>i. new test results, ii. (4), iii. (5) PI=5-10%</td>
<td>i. new test results, ii. (5) PI=5-10%, iii. (5) PI=10-20%</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>GEO (1997)</td>
<td>(1)</td>
<td>(5) PI=50%</td>
<td>-</td>
<td>(1)</td>
<td>(1)</td>
<td>(5) C1 or C2</td>
</tr>
<tr>
<td>Arup previous studies</td>
<td>(1)</td>
<td>(3) PI=35%</td>
<td>(3) PI=35%</td>
<td>(1)</td>
<td>(1)</td>
<td>(1) or SBPM results</td>
</tr>
<tr>
<td>Lee et al. (1998)</td>
<td>(3) PI=0</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
</tr>
<tr>
<td>This study</td>
<td>(1)</td>
<td>(3) PI=35%</td>
<td>(3) PI=35%</td>
<td>(1)</td>
<td>(2) lower bound</td>
<td>New data</td>
</tr>
</tbody>
</table>

where (1) = Seed & Idriss (1970) curve for sands
(2) = Seed et al. (1986) curves for sands and gravels
(3) = Vucetic & Dobry (1991) curves for clays of different plasticity index (PI)
(4) = Stokoe & Lodde (1978) curves for clays
(5) = Sun et al. (1988) curves for clays of different PI
SBPM = self-boring pressuremeter

#### 6.2.6 Site amplification factors

Figure 6-5 compares the average computed site amplification factors to those presented in the NEHRP Guidelines (FEMA, 1997). At short periods, the computed site amplification factors for site class C exceed the NEHRP factors significantly. At long periods, the NEHRP factors are larger than the computed factors, with the exception of site class D which shows reasonably good agreement. In general, the agreement between the NEHRP factors and those computed for Hong Kong is poor, justifying the decision to derive new factors. The uncertainty associated with the use of the NEHRP factors, which were found to overestimate amplification by a factor of more than two in some cases, could overshadow many of the other uncertainties, including the aleatory uncertainty associated with the seismic hazard estimate.
Chapter 6 Site amplification and liquefaction hazard

- Site Class C - calculated
- Site Class D - calculated
- Site Class E - calculated

**Figure 6-5:** Comparison of computed average site amplification factors for site classes C, D and E for Hong Kong soil profiles and seismic hazard levels, compared with the NEHRP factors (a) short-period, $S_{as}$ (see Table 2-3), (b) long-period, $S_{al}$ (see Table 2-4)

The approach presented here allows the uncertainty to be better defined than when NEHRP or similar factors are adopted. There was a significant scatter in the computed spectral ratios associated with the large number of representative soil profiles selected for each site class and the sensitivity checks included to allow for uncertainty in some of the dynamic soil parameters assumed in the site response analyses. In a regional study, uncertainties associated with the need to group similar soil profiles into a limited number of classes, when, in fact, each profile will respond differently to strong ground shaking, are inevitable. For Hong Kong, this variability was incorporated into the seismic hazard estimation, through the derivation of the mean and one and two standard deviations above and below the mean for the site amplification factors (Pappin *et al.*, 2004).
6.3 Assessment of liquefaction potential

Regional assessments of seismic hazard and risk require the definition of zones of equal relative susceptibility to liquefaction, such that for each zone, the expected ground deformation under a given strength of shaking can be estimated. The availability of a reasonably large body of geotechnical data for Hong Kong offers an opportunity to adopt an 'expert-generated ground failure estimation' approach to determine the probability of liquefaction (FEMA, 2003). This section presents an assessment of the liquefaction potential for Hong Kong, based upon the soil profiles indicated in Table 6-2.

Susceptibility to liquefaction can be determined from historical precedence or empirical criteria, as described in Section 2.9.2. There is no historic evidence of liquefaction in Hong Kong; however, this could be due to the lack of significant earthquake shaking in recent times, and does not preclude the occurrence of liquefaction in future earthquakes, particularly considering the extensive development and reclamation carried out in recent years.

Geological and geotechnical information for Hong Kong have been used to identify which soil layers are potentially susceptible. Commonly encountered near-surface soils are shown in Table 6-4, along with their considered susceptibility to liquefaction, with explanatory notes below. The ratings presented in Table 2-6 (from Iwasaki, 1982) were referred to, in combination with local knowledge and judgement. Careful consideration of liquefiable soils at this stage is a valuable component of a loss estimation, since it can eliminate a large number of sites and therefore allow greater focus on the appropriate zones.

Figure 6-6: Computed spectral ratios for selected site class D soil profiles, showing mean and standard deviations (σ) (Pappin et al., 2004). The input motion in this case was representative of the Mw 6.3 at 50 km, scenario, 1.2 σ above the median value to match the 2% in 50 year ground motion spectrum.
Table 6-4: Liquefaction susceptible soil deposits in Hong Kong

<table>
<thead>
<tr>
<th>Deposit</th>
<th>Variability</th>
<th>Distribution around Hong Kong</th>
<th>Locations</th>
<th>Susceptibility rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reclamation fill¹</td>
<td>High</td>
<td>Moderate</td>
<td>Developed coastal zones</td>
<td>Likely</td>
</tr>
<tr>
<td>Alluvium²</td>
<td>High</td>
<td>Widespread</td>
<td>Coastal and low lying areas</td>
<td>Possible</td>
</tr>
<tr>
<td>Marine Deposits²</td>
<td>High</td>
<td>Limited</td>
<td>Reclaimed coastal zones</td>
<td>Possible</td>
</tr>
<tr>
<td>Colluvium³</td>
<td>Very High</td>
<td>Moderate</td>
<td>Hillsides and valleys</td>
<td>Unlikely</td>
</tr>
<tr>
<td>Decomposed rock</td>
<td>Low</td>
<td>Widespread</td>
<td>Everywhere</td>
<td>Unlikely</td>
</tr>
</tbody>
</table>

¹ The reclaimed fill material in Hong Kong is very variable in terms of composition and relative density, from uniform clean sand with a fines content of less than 10%, to public fill containing construction debris and decomposed rock. It is the clean sandy fill that is considered in this liquefaction assessment.

² The alluvial and marine deposits vary from sands through to clays, and only the sands and silts are considered susceptible for these purposes. Strength loss in plastic soils with high clay contents is not considered in this study.

³ Colluvium is generally very variable and even if small pockets may be susceptible to liquefaction, it is considered unlikely that this would be widespread, or significant in terms of the overall risk in Hong Kong.

15 soil profiles for which liquefaction assessments are carried out, based upon the susceptibility rating in Table 6-4, are illustrated in Figure 6-7 and Figure 6-8. All of the representative sites are located in areas of coastal reclamation. This was not a deliberate criterion, but it is nonetheless considered a reasonable premise that the liquefaction hazard in Hong Kong will be concentrated in such zones.

6.4 Probability of liquefaction

As noted in Section 2.9.3, a probabilistic approach to determining the likelihood of liquefaction is preferable to a deterministic one in the context of risk assessment. This is because ascertaining the factor of safety does not provide sufficient information to quantify the risk.
Figure 6-7: Illustrations of selected profiles for liquefaction assessment

Legend:
CD – completely decomposed
HD – highly decomposed
MD – moderately decomposed
G – Granite
V – Volcanics
S – Siltstone
Figure 6-8: Illustrations of selected profiles for liquefaction assessment
6.4.1 Assessment methodology

In this case study, the probabilistic relationship of Seed et al. (2003), shown in Equation 6-3 has been used. A significant advantage of this approach over previous studies is the more rigorous treatment of uncertainties relating to measurement errors, model imperfection and inherent variability in the estimation of the cyclic shear stress ratio (CSR) and measured penetration resistance. In the derivation of Equation 6-3 each variable was considered as a distributed parameter rather than a point value.

\[
P(L) = \Phi \left[ \frac{N_{1,60} \cdot (1 + 0.004 \cdot FC) - 13.32 \cdot \ln(\text{CSR}) - 29.53 \cdot \ln(M_w) - 3.70 \cdot \ln(\sigma_v') + 0.05 \cdot FC + 44.97}{2.7} \right] \tag{6-3}
\]

where:

- \( P(L) \) = Probability of liquefaction
- \( \Phi \) = Standard cumulative normal distribution
- \( N_{1,60} \) = SPT N corrected for overburden effects and for energy as follows:
  \[
  N_1 = \left( \frac{1}{\sigma_v'} \right)^{0.5} N \quad \text{(where } \sigma_v' \text{ is in atmospheres, 1 atm = 101.325 kPa)} \tag{6-4}
  \]
  \[
  N_{1,60} = N_1 \cdot C_R \cdot C_S \cdot C_B \cdot C_E \tag{6-5}
  \]
- \( C_R \) = Correction for rod lengths shorter than 10m
- \( C_S \) = Correction for non-standard SPT samplers
- \( C_B \) = Correction for borehole diameters greater than 115mm
- \( C_E \) = Correction for energy efficiency of SPT hammer
- \( FC \) = Percent fines content (percent finer than 0.074mm), expressed as an integer
- \( M_w \) = Moment magnitude
- \( \sigma_v' \) = Effective vertical stress (in psf, 1 kPa = 20.884 psf)
- \( \text{CSR} \) = ‘Equivalent uniform cyclic shear stress ratio’, evaluated by means of site-specific seismic site response analyses, described further below. The ‘equivalent uniform’ CSR is assumed to be equal to 65% of the computed peak shear stress, where:

\[
\text{CSR}_{eq} = 0.65 \cdot \frac{\tau_{\max\text{ (site response)}}}{\sigma_v'} \tag{6-6}
\]
Figure 6-9a shows that the deterministic 'simplified' relationship proposed by Seed et al. (1984) appears to correlate to a probability of liquefaction of around 50% using the above procedure. This is not the case however, as the two relationships calculate the CSR differently and are not directly comparable (Seed et al., 2003). After adjusting the earlier relationship for the difference in CSR, it is shown to correlate to a probability of liquefaction of 10-30%. The difference in CSR for the two procedures relates to the use of site-response analyses in place of the 'simplified' relationship (Equation 2-5). This underlines the importance of calculating CSR in a manner consistent with the selected liquefaction assessment relationships.

6.4.2 Calculation of CSR through site response analysis

Simplified relationships for estimating the stress reduction factor, $r_d$, such as those presented by Youd et al. (2001), do not consider the scatter, influenced by earthquake magnitude, intensity of shaking and site stiffness as well as depth below ground level, and are therefore potentially unconservative. The calculation of the in situ CSR by means of site response analysis reduces this uncertainty. Seed et al. (2003) state that "... the best means of estimation of in situ CSR ... is to directly calculate CSR by means of appropriate site-specific, and event-specific seismic site response analyses".
Input parameters to the site response analysis, using SIREN, have been briefly described in Section 6.2.5 and Appendix E. The small strain shear moduli, $G_0$, shown in Figures 6-7 and 6-8 are obtained both from in situ shear wave velocity measurements (and Equation 6-7), and from empirical correlations between penetration resistance and $V_s$ or $G_0$.

\[
G_0 = \rho (V_s)^2
\]  

(6-7)

where $\rho$ is the soil density (in T/m$^3$).

In situ tests such as SPT and CPT generate large strains in the soil and therefore empirical correlations relating them to small strain soil stiffness should be expected to show significant scatter and should be treated with caution. Nonetheless they are frequently used in practice, and provided that the empirical dataset from which they were derived is sufficiently large, and relates to the soils in question, they provide a reasonable approach using widely available input data. For sands, the following correlations between SPT and $G_0$ have been presented.

\[
G_0 \text{ (MPa)} = 6.4 \ N_{60} 
\]  

(6-8)  

Stroud, 1989

\[
G_0 \text{ (MPa)} = 15.6 \ (N_{60})^{0.68} 
\]  

(6-9)  

Imai & Tonouchi, 1982

where $N_{60}$ represents $N$ normalised to 60% energy efficiency, see Section 6.4.3.

\[Figure 6-10: \text{SPT N vs. G0} \text{ (Imai & Tonouchi, 1982)}\]

SIREN computes the maximum shear stress, $\tau_{\text{max}}$, in each element, the thickness of the elements being determined by the natural frequency of the layer, which should be kept approximately constant throughout the profile. $\tau_{\text{max}}$ is then input to Equation 6-6 to calculate $\text{CSR}_{\text{max}}$ in each element.
6.4.3 Input parameters for liquefaction assessment

The variable input parameters for Equation 6-3 are CSR, \((N_{1,60})\), \(\sigma_v^*\), FC and \(M_w\). The best-estimate SPT N values are derived from a number of boreholes to obtain each representative soil profile and are consistent with the Go values used in the site response analyses. Single high or low values in a borehole will not be included in the analysis, unless they are representative of more general trends. These single values are unlikely to influence the likelihood of damaging liquefaction over a given area, which will be governed by the average behaviour. On the other hand, there is a natural variability associated with soil relative density, reflected by the scatter in measured SPT N values. For the analyses presented herein this scatter is represented by the different soil profiles analysed, as shown in Figure 6-11 there is a reasonable variation amongst these profiles, due to a combination of different thicknesses of fill material and different levels of compaction.

The data in Figure 6-11 are labelled according to their NEHRP site classifications, which are derived from the weighted average SPT N over the top 30 m of soil. However, for the purposes of liquefaction assessments, which are dependent upon individual SPT N values in any layer, it should be noted that these site classifications are not necessarily relevant.

![Figure 6-11: SPT N blowcounts versus depth of profiles analysed for liquefaction potential (Closed symbols are Site Class D, open symbols are Site Class E)](image)

The following \(N_{1,60}\) correction factors (Equation 6-4) have been assumed:

- The values for \(C_R\) are shown in Table 6-5:

---

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Table 6-5: Rod length correction factors ($C_R$) (after Youd et al., 2001)

<table>
<thead>
<tr>
<th>Rod Length (m)</th>
<th>$C_R$</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt; 3</td>
<td>0.75</td>
</tr>
<tr>
<td>3 to 4</td>
<td>0.8</td>
</tr>
<tr>
<td>4 to 6</td>
<td>0.85</td>
</tr>
<tr>
<td>6 to 10</td>
<td>0.95</td>
</tr>
<tr>
<td>&gt; 10</td>
<td>1.0</td>
</tr>
</tbody>
</table>

- $C_S = 1.0$, as standard SPT samplers are used in Hong Kong
- $C_B = 1.0$, assuming a standard 115mm borehole diameter
- $C_E = 1.0$ for standard automatic trip hammers used in Hong Kong

The fines contents of the soils are assumed to be 10% for reclamation fill, 20% for alluvial and marine sands, and 40% for alluvial silt.

### 6.4.4 Calculated probability of liquefaction, $P(L)$

Figure 6-12 presents results for the two earthquake scenarios corresponding to a 2% probability of exceedance in 50 years, defined in Section 6.1. For lesser ground motions, both the induced shear stresses and the magnitudes are lower and the probability of liquefaction was found to be zero or negligible. The ‘deterministic’ curves of Seed et al. (1984) were intended to represent 10-15% probabilities (Seed et al., 2003), therefore it is assumed that where $P(L) \leq 15\%$ throughout a profile, the liquefaction risk is negligible and can be ignored.

The two earthquake scenarios considered comprise $M_w 6.3$ at 50 km, and $M_w 7.5$ at 300 km. In past investigated earthquakes, there have been very few cases of damaging liquefaction at distances greater than 100 km from the earthquake source (Ambraseys, 1988; Youd & Perkins, 1978; Iwasaki, 1986). Ambraseys (1988) also showed that $M_w 6.3$ earthquakes have not caused liquefaction at distances of more than approximately 40 km. Nonetheless, the aforementioned studies were based on limited data from Europe, California and Japan respectively, so a more direct approach considering the strength and duration of ground shaking and the in situ soil properties was warranted in this case. Figure 6-12 shows that the liquefaction risk is lower for the moderate magnitude event at 50 km than it is for the larger magnitude distant event. This would suggest that the use of PGA as a parameter for determining the susceptibility to liquefaction may not be appropriate, since the surface PGA is typically 30 to 50% larger for the former. This observation is primarily a function of the induced stresses, since the same pattern is seen even after removing the magnitude dependence.
There is a significant scatter in the results of this assessment both in terms of probability of liquefaction and in thickness and depth of the liquefiable layer. Generally, it can be concluded that coastal reclamation sites are potentially liquefiable at the upper bound of the expected levels of shaking; this liquefaction may have probabilities as high as 80%, and may affect materials as deep as 15 m.

6.4.5 Sub-division of liquefiable sites

The calculated probability, $P(L)$, for individual profiles is shown in Figure 6-13. An important difference between regional studies and site-specific assessments is illustrated by this figure. In a site-specific assessment, the variability of the liquefaction potential across a site would be considered, and accounted for in design. A number of the profiles considered here were found to have a very low individual susceptibility to liquefaction. However, in regional assessments, it is necessary to group many sites which, based upon preliminary considerations, are judged to have similar susceptibilities to liquefaction. These groups must be assumed to behave in a reasonably uniform manner for all subsequent calculations. In reality, some buildings sited on these soil groups will not experience liquefaction, and others almost certainly will, but the overall behaviour must be represented by a ‘typical’ probability of liquefaction, somewhere between the two.

The behaviour of the 15 profiles has been considered with a view to dividing them into sub-groups with similar response to liquefaction, in order to reduce the influence of the variability
amongst profiles. To satisfy the requirements of a regional study, these groups should be straightforward to define, and not rely heavily on in situ data that are unlikely to be readily available.

Figure 6-13: Probabilities of liquefaction for each soil profile. Solid line represents the moderate magnitude mid-field event, dashed line represents a large magnitude far-field event, both have a probability of exceedance of 2% in 50 years.
Bearing in mind that the original selection of potentially liquefiable profiles was based upon the presence of reclamation fill and/or cohesionless alluvial or marine deposits, the following additional refinements were considered, presented in order of increasing data requirements:

1. $V_{S30}$ classifications, i.e. using the NEHRP site classifications that have already been defined, with an additional intermediate class ‘D/E’. There is no additional effort required since these classifications are necessary for soil zonation. The results divided in this manner are shown in Figure 6-14.

2. Thickness of reclamation (Figure 6-15). This is relatively easy to define, using the information presented in Appendix D, combined with topographic and bathymetric data.

3. The depth to the base of the naturally occurring Quaternary deposits (i.e. top of weathered rock). This classification potentially improves upon the previous one in that the presence of alluvial and marine deposits is included. However, it doesn't distinguish between clays and sands in these deposits, which would require in situ data, and is more difficult to define. The sites were grouped into three categories: between 10 and 20 m; between 20 and 30 m and over 30 m to the base of the Quaternary deposits, shown in Figure 6-15.

4. Average SPT N value in liquefiable deposits. This classification obviously has the most direct relation to the susceptibility to liquefaction. It is also the most difficult to define in terms of what thickness of deposits to consider, and whether to use average or minimum values.

Figure 6-14: Probability of liquefaction for sites categorised according to NEHRP classifications. As before, solid lines represent the moderate magnitude mid-field event, dashed lines represent the large magnitude far-field event, both have probability of exceedance of 2% in 50 years.
A particular criterion sought from more refined classifications of potentially liquefiable soil was that they should identify a rule that could eliminate the sites with zero or negligible probabilities of liquefying, namely D3, D6 and E5 (see Figure 6-13). None of the four schemes considered above were successful in isolating or even obtaining a common grouping for these three sites. By considering them individually the reason for the comparatively low risk of liquefaction can be identified. D3 has a very thick layer of reclamation fill, but this fill has been well compacted and therefore has a SPT N value greater than 10 throughout. At D6 the top of the weathered rock is relatively shallow, and furthermore this is overlain by Colluvium, which is non-
liquefiable, and has a higher stiffness than other Quaternary deposits. The ground motion therefore undergoes less amplification than at other sites, so the shear stresses in the potentially liquefiable fill are not high enough to induce liquefaction. Site E5 has a layer of very soft marine clay underlying the fill material, which de-amplifies the shaking, plus the fill material is again well compacted. These features can only be identified from site specific investigation and analysis.

In this case, none of the simple criteria considered were satisfactory in terms of sub-dividing potentially liquefiable sites in order to reduce the scatter in the results. The four criteria considered above require additional expenditure of time and effort and would increase the complexity of the calculations, with no clear benefit in terms of reducing the uncertainty.

6.4.6 Thickness of liquefied layer

An alternative representation of the results is shown in Figure 6-17. The probability of liquefaction has been divided into 3 categories; P(L)>0.15, P(L)>0.5 and P(L)>0.85, where the lowest probability is representative of a present risk of liquefaction occurrence, the middle value represents an above average risk, and the final category a significant risk. Figure 6-17 shows that for the highest probability of liquefaction occurrence, the average thickness of liquefied soil is approximately 1 m, a thickness that has little damaging potential in many cases.

![Figure 6-17: Thickness of liquefied layer versus probability of liquefaction, P(L), all sites](image)

6.5 Correlation of P(L) to ground motion parameters

The detailed calculations of the probability of liquefaction presented in the previous sections are only relevant to specific earthquake scenarios. If alternative seismic hazard scenarios for the region were to be considered, the entire procedure described in Section 6.4 would have to be
repeated. In this respect, one of the advantages of the HAZUS methodology described in Section 4.10 becomes clear. Having defined the susceptibility ratings of the soils in the study area, very little additional work is required to evaluate the losses due to liquefaction for any number of seismic hazard scenarios. In this section, a similar approach is sought, but using the detailed calculations for the region as input, such that possible simplified relationships have a rational basis, whilst removing the necessity for further site response analysis.

6.5.1 CSR ‘simplified’ vs. CSR from site response analysis

As explained in Section 6.4.2, the use of site response analysis to calculate the CSR is both consistent with the recommendations of the authors of the liquefaction assessment method (Seed et al., 2003) and eliminates the uncertainty associated with the stress reduction factor, $r_d$. Nonetheless, undertaking site response analyses generates significant additional work. The CSR from Equation 6-6 is therefore compared with the ‘simplified’ CSR (Equation 2-5). If a correlation between the two could be established, $P(L)$ could be calculated for other hazard levels on this basis.

![Correlation between simplified CSR and site specific CSR](image)

**Figure 6-18:** Correlation between simplified CSR and site specific CSR. Dashed line shows a gradient of 1, solid line shows a linear trendline, constrained to pass through the origin.

Figure 6-18 compares the two sets of values. The CSR values from site response analysis in Figure 6-18 have been calculated for the three levels of seismic hazard described in Section 6.1, and for all 15 profiles for which liquefaction assessments have been carried out. The simplified CSR has been calculated according to Equation 2-5. The surface PGA is the estimated value obtained as described in Section 6.2; although site specific PGA values were available for each profile and each scenario from the site response analysis, using these values is not consistent
with the use of the simplified relationship. The stress reduction factor, $r_d$, for input to the simplified calculation has been obtained using the relationship presented by Youd et al. (2001):

$$r_d = \frac{(1 - 0.4113z^{0.5} + 0.04052z + 0.001753z^{1.5})}{(1 - 0.4177z^{0.5} + 0.05729z - 0.006205z^{1.5} + 0.001212z^2)}$$  \hspace{1cm} (6-10)

where $z$ = depth below ground surface in m.

Improved $r_d$ relationships are presented by Seed et al. (2003), obtained from regression of 2,153 site response analyses, and recognising the non-linear dependency of $r_d$ on a number of parameters, not only depth. Their (somewhat cumbersome) equation for $r_d$ at depths less than 65 ft (20 m) is shown in Equation 6-11. However, the increased number of variables, and particularly the requirement for $V_{s,40}$, to be calculated for each site, means that these relationships are not consistent with the requirements of the simplified relationship.

$$r_d = \frac{\left[1 + \frac{-23.013 - 2.949 \cdot a_{\text{max}} + 0.999 \cdot M_w + 0.016 \cdot V_{s,40}}{16.258 + 0.201 \cdot e^{0.104(z - 0.0785 \cdot V_{s,40}) + 24.888}}\right]}{\left[1 + \frac{-23.013 - 2.949 \cdot a_{\text{max}} + 0.999 \cdot M_w + 0.016 \cdot V_{s,40}}{16.258 + 0.201 \cdot e^{0.104(z - 0.0785 \cdot V_{s,40}) + 24.888}}\right]} + \sigma_{r_d}$$  \hspace{1cm} (6-11)

where: $V_{s,40}$ is the average shear wave velocity over the top 40 feet (12.2 m) of the site

$$\sigma_{r_d} = 0.0072 \cdot z^{0.850} \text{ for } z < 40 \text{ ft and } 0.0072 \cdot 40^{0.850} \text{ for } z \geq 40 \text{ ft}$$

The two methods were not expected to produce the same CSR values; the discussion of the difference between the two curves shown in Figure 6-9 identified the divergence between the two approaches. However, Figure 6-18 also shows that the correlation between the two parameters is poor in this case, with an R-squared value of only 0.42 from the linear trendline. It seems that the scatter is less at low values (CSR ≤ 0.05), these values relate to the disaggregated scenarios from the 50% in 50 year hazard level, and do not pose any liquefaction risk. At higher CSR values, relevant to liquefaction risk assessment, the scatter increases significantly. This scatter can be explained by the variability amongst the different sites analysed, which is only represented in terms of the mean site classification amplification factor used to obtain the surface PGA in the simplified procedure, and by the use of PGA in the simplified relationship. The principal reason for using PGA in the simplified relationships is assumed to be its wide availability.
6.5.2 Improved relationships for estimating CSR

In this study, both bedrock and surface response spectra are available for each profile, which provides an opportunity to consider ground motion parameters other than PGA that may have a better correlation to the computed shear stress ratio and therefore to the calculated probabilities of liquefaction.

Through linear regressions of the shear stress ratio as a function of both a ground motion parameter and depth, it was concluded that a reasonable degree of correlation could be obtained using PGV as the ground-motion parameter. The following relationships have been derived:

Site class D
\[
\ln \left( \frac{\tau_{\text{max}}}{\sigma_r} \right) = 11.48 \cdot PGV - 0.024 \cdot z - 3.0 \quad R^2 = 0.86 \quad (6-12)
\]

Site class E
\[
\ln \left( \frac{\tau_{\text{max}}}{\sigma_r} \right) = 11.17 \cdot PGV - 0.029 \cdot z - 3.08 \quad R^2 = 0.80 \quad (6-13)
\]

where
- PGV = peak ground velocity at ground surface (m/s)
- \(z\) = depth below ground surface (m)

The strengths of these relationships are the good correlations, and the rational basis behind the selection of PGV, being related to the shear strain in the soil. The main weakness is that the use of PGV would require a suitable relationship for the prediction of surface PGV in Hong Kong. The present values have been obtained from the site response analyses; this partly explains the good correlation. Alternative relationships were considered based upon bedrock spectral accelerations, at PGA, 0.5s, 1.0s and 2.0s. The best correlation was obtained for the 0.5s spectral acceleration, where:

Site class D
\[
\ln \left( \frac{\tau_{\text{max}}}{\sigma_r} \right) = 0.73 \cdot SA(0.5s) - 0.022 \cdot z - 2.94 \quad R^2 = 0.83 \quad (6-14)
\]

Site class E
\[
\ln \left( \frac{\tau_{\text{max}}}{\sigma_r} \right) = 0.63 \cdot SA(0.5s) - 0.03 \cdot z - 2.92 \quad R^2 = 0.78 \quad (6-15)
\]

where \(SA(0.5s)\) = spectral acceleration at \(T=0.5s\) at bedrock (m/s²)

The PGV relationships have a slightly improved correlation overall, but the difference is not significant. Using bedrock accelerations avoids double-counting of the uncertainty associated with the soil amplification. However, since only six earthquake scenarios were defined, the
above relationships are based upon insufficient data to be considered reliable, and are presented herein as an illustration of a possible approach for future regional risk models.

6.5.3 Simplified estimation of \( P(L) \) for case study

Relationships have been obtained for estimating the shear stress ratio as a function of site class, depth, and 0.5s spectral acceleration at bedrock (Equations 6-14 and 6-15). The other random variables that influence the calculated probability of liquefaction are \( N_{1,60} \), FC, \( M_w \) and \( \sigma'_v \) (see Equation 6-3). For any scenario earthquake, \( M_w \) is known and can be incorporated directly. Fines content and effective vertical stress are relatively well constrained, and the influence of their variability within these constraints is ignored for these purposes. This assumption is supported by a sensitivity study presented in Section 7.5.1. Therefore if CSR can be calculated from Equations 6-14 and 6-15, \( N_{1,60} \) is the only other significant variable that needs to be determined for a simplified calculation of \( P(L) \). For illustration, the average \( N_{1,60} \) value and +/- one standard deviation for the top 15m of soils of all Site Class E profiles has been calculated, shown in Figure 6-19.

**Figure 6-19:** \((N_{1})_{60}\) data for site classes D and E, solid line shows the mean, dashed lines show +/- one standard deviation above and below the mean, for site class E data.

Assuming a uniform fines content of 10%, a linear relationship for \( \sigma'_v \) with depth (for density = 19 kN/m\(^3\) and ground water level = 2 m below ground level), and \( M_w=7.5 \), \( P(L) \) can be estimated for any value of \( SA(0.5s)_{\text{bedrock}} \) from Equations 6-15 and 6-3, as shown in Figure 6-20. Figure 6-21 compares the more rigorous calculation of \( P(L) \) with this simplified approach for
site class E data. The prediction appears to give a better agreement in the upper 7m or so. This could be because there are fewer data points at greater depths both for the regression and for the SPT N value.

![Diagram](image)

**Figure 6-20**: Simplified calculation of $P(\text{Liquefaction})$ for $M_w=7.5$, $FC=10\%$, $(N_{100})_{\text{ave}}$ average and $\pm 1$ s.d. from Figure 6-19 for site class E. $P(L)$ is the maximum value in the profile.

![Diagram](image)

**Figure 6-21**: Simplified prediction of $P(L)$ (bold lines) as defined in Figure 6-20 for SA(0.5s)bedrock from the 2% in 50 year, large magnitude, far-field scenario, compared to the detailed calculation for site class E profiles, dashed lines show $\pm 1$ s.d. of the $(N_{1})_{\text{ave}}$ value.

### 6.5.4 Discussion

The principle uncertainty relating to development of simplified calculations of the probability of liquefaction is the assumed value of $(N_{1})_{\text{ave}}$, which could be either uniform or linearly increasing with depth. The actual scatter in N values is large, both as a result of natural variability at one
site, and, more significantly, as a result of grouping many sites into single site classifications. The calculated probability of liquefaction is sensitive to variability in the N value. However, in comparison to the HAZUS methodology, in this case N can be determined for each site classification, and its variability can be specifically incorporated.

The procedures developed in this section propose a means of combining the computationally intensive liquefaction assessments with the development of simplified relationships that are more compatible with the requirements of a regional loss estimation. The relationships are highly region specific; they do not eliminate the need for detailed calculations, but serve a useful purpose in reducing the overall volume of the calculations.

6.6 Comparison of HAZUS liquefaction assessment to detailed calculations

This section briefly compares the assessment of the probability of liquefaction, using the HAZUS default approach (Section 4.10) to the probability of liquefaction obtained from detailed analysis of the data presented above.

6.6.1 Susceptibility to liquefaction

Reclamation fill material has been shown to be the most susceptible soil in Hong Kong to liquefaction. From Table 2-5 it can be seen that fill material has a susceptibility rating of very high uncompacted, and low if compacted. Whilst the general assumption underlying this classification is reasonable, it appears to be an over simplification regarding the reclamation fill in Hong Kong, which is very widespread and of varying quality and age. The degree of compaction depends upon the placement method used and the proposed end use of the site, and is not always known in the absence of in situ data. The Youd and Perkins (1978) classification system does not describe the parent material of the fill, and does not have a classification for improved fill which has been shown to have a high resistance to liquefaction, for example in the Kobe earthquake in 1995 (e.g. Park et al., 1995). In short, judgement must be exercised in order to produce a map showing susceptibility to liquefaction, and clearly a misclassification such as rating fill material as compacted when it is in fact uncompacted or poorly compacted will have a major influence on the results.

6.6.2 Proportion of map unit

As discussed in Section 4.10.2, the P_m factor has a significant impact on the overall probability of liquefaction. From Figure 6-13 it can be seen that for the higher magnitude earthquake, only three of the 15 profiles considered have a zero or negligible probability of liquefaction, whereas
for the moderate magnitude event, 11 of the 15 sites are deemed unlikely to liquefy. If these profiles can be taken as representative of the variability amongst potentially liquefiable sites in the region, then the corresponding $P_{ml}$ factor would be 0.8 in the former case and 0.27 in the latter. This implies that magnitude and strength of shaking will influence the $P_{ml}$ factor, which is a reasonable premise, and one that the HAZUS approach fails to incorporate. As noted in Section 4.10.2, where the amplitude and duration of ground shaking are sufficient there is no reason why an entire zone of liquefiable ground could not liquefy.

Explicit consideration of the variability of liquefiable deposits through the use of in situ data represents a significant improvement upon the use of the $P_{ml}$ factor.

### 6.6.3 Conditional probability of liquefaction

The conditional probability of liquefaction for the two different earthquake scenarios is calculated below with reference to Equation 4-8 and Figure 4-17. The $P_{ml}$ factor is not applied in this case, for the reasons discussed above, it should not be required for comparison with the individual profiles considered in this chapter. The first scenario has $M_w = 6.3$ and $PGA = 0.2g$, the second has $M_w = 7.5$ and $PGA = 0.1g$ (estimated surface values from bedrock spectra and amplification factors for site class E). Ground water depth is assumed to be 2.0m below ground level in both cases, and the susceptibility categories of *moderate* and *very high* are considered to represent the variability of the fill material.

| Scenario | Susceptibility | $P[L|PGA]$ | $K_M$ | $K_w$ | $P[L_{sc}]$ |
|----------|----------------|------------|-------|-------|-------------|
| 1        | Moderate       | 0.33       | 1.29  | 1.07  | 0.24        |
| 2        | Moderate       | 0          | 1.02  | 1.07  | 0           |
| 1        | Very High      | 1.0        | 1.29  | 1.07  | 0.72        |
| 2        | Very High      | 0.09       | 1.02  | 1.07  | 0.08        |

The most significant observation from Table 6-6 is that the conditional probability of liquefaction for the first scenario is higher than for the second, whereas the more detailed calculations in this chapter have shown the opposite. This is because of the dependency of the HAZUS methodology on PGA; the high computed shear stresses obtained through site response analysis are a function of the entire time history, not only PGA. Figure 6-22 shows the range of values from Table 6-6 superimposed onto the predicted $P(L)$ from Figure 6-12. Unsurprisingly there is very poor agreement between the two results and this is worse for the long-period earthquake scenario.
6.7 Summary

The calculations of probability of liquefaction presented in this chapter require a significant additional investment in terms of time and resources compared to application of the HAZUS approach. The 15 profiles selected for the study were indicative of the range and variability of potentially liquefiable soils in the study area, but a much more comprehensive evaluation could have been carried out by increasing the number of profiles. The number of profiles required is also dependent on the characteristics of a study area.

For loss models where liquefaction is recognised as being potentially significant the steps presented in this chapter represent a possible procedure for determining the probability of liquefaction for selected earthquake scenarios. The uncertainty associated with each step will influence the final outcome. Even in this chapter, a number of simplifying assumptions have been made in comparison to site specific analyses. These are listed below.

1. The selection of representative profiles for analysis requires judgement to be exercised, leaving some uncertainty as to whether the full variability or even the expected average behaviour has been adequately represented. For a site specific study, all boreholes could be directly considered in the analysis.
2. The definition of soil parameters for each representative profile again requires simplifying assumptions to be made. The expected parameters rather than the upper or lower bounds have been considered, and in the derivation of dynamic soil properties such as $G_0$ and $G/G_0$ degradation, the most appropriate relationship has been selected using judgement, without extensive sensitivity studies to consider the influence of the modelling assumptions.

3. For the site response analyses, single time-history input files were developed based upon artificial adjustment of real earthquake records. The sensitivity of the outcome to this component has not been fully explored.

4. Only the median hazard level has been considered. An improved approach in terms of incorporating uncertainties would require at least an additional four calculations for each scenario, to consider one and two standard deviations above and below the median value. The optimum approach would be to combine the probability distributions of the demand and the soil response using reliability analysis techniques.

5. The scatter of the calculated probabilities of liquefaction has not been rigorously treated. The variability between profiles is in terms of both $P(L)$ and the thickness of the liquefied layer. No classification scheme was identified that would serve to reduce this scatter to any great extent.

Section 6.5 considered the derivation of simplified relationships for the estimation of $P(L)$ for alternative hazard scenarios. The variable parameters that would be required to implement this approach include magnitude, depth, NEHRP site class, bedrock spectral acceleration at $T=0.5$ s, fines content, the variation of $\sigma$ with depth, and the variation of $(N_{100})$ with depth. Despite the criticisms levelled at aspects of the HAZUS methodology in this thesis, the study in Section 6.5 illustrates the difficulties encountered when seeking to develop a model that satisfies the requirements of a regional loss estimation in terms of simplicity of input data, ease of incorporation into a spreadsheet or database type of calculation and also produces realistic results.

The key advantage of the explorations of liquefaction hazard presented in this chapter is that, because their origin lies in real data that has been selected and interpreted by the author, the uncertainties and the assumptions required are documented throughout the procedure and therefore transparent. Having recorded the source of the uncertainties, their significance can be explored, unlike in the use of simplified methodologies. Chapter 7 contains further discussion of some of these issues.
Chapter 7

Key issues

7 AN IMPROVED FRAMEWORK FOR LIQUEFACTION-INDUCED EARTHQUAKE LOSSES: KEY ISSUES

The previous chapters of this thesis have demonstrated that the incorporation of liquefaction into a regional damage assessment is potentially important and also that to do this is non-trivial. The data and analysis requirements can be significant and there is therefore much to gain from the use of simplified approaches. However, it has been shown that attempts to follow oversimplified guidelines can lead to the situation where a model loses rather than gains reliability by incorporating liquefaction, despite the additional investment required.

The issues and ideas developed in the previous chapters are summarised in this chapter within the general context of a framework for the assessment of liquefaction-induced building damage. The following 10 key areas are identified and discussed in turn:

1. Treatment and definition of uncertainties
2. Shortcomings in existing survey data and needs for future investigations.
3. Definition of seismic hazard and site effects as input to liquefaction studies.
5. Liquefaction susceptibility and probability of liquefaction.
7. Classification of buildings in terms of their response to liquefaction-induced ground deformation.
8. Building damage scales for field surveys and damage estimations.
9. Assessment of building vulnerability to liquefaction demands.
10. The interaction of ground shaking- and ground failure-induced damage to buildings.

The initial decision to be made is whether a loss estimation model should include liquefaction. This should be made on the basis of the requirements of the model, as shown in Figure 3-25, and also its scale. With respect to building damage, as a rule, the larger the study area, the more likely it is that ground shaking-induced damage to buildings will dominate the overall losses (except for rare but catastrophic cases of major tsunamis or massive landslides), and that any individual features of a particular zone will lose their significance. For example, in the 1999 Kocaeli earthquake in Turkey, damaging hazards included surface fault rupture, coastal...
subsidence, slope failures and liquefaction, but the dominant cause of damage over the entire region was undoubtedly ground shaking. An estimation of building damage for the entire Kocaeli region could therefore justifiably concentrate on ground-shaking damage to buildings. This feature is illustrated in Figure 7-1, where individual features of survey zones in the near-field area, which may be due to site conditions, near-fault effects, building stock or even survey techniques are, to a large extent, smoothed out when the average damage distribution is considered. For insurance purposes, the average behaviour may be the information of interest, but for many other purposes, the ability to predict the individual damage distributions could be very useful. Even for insurance schemes, there is a potential benefit to understanding the reasons for the different damage distributions, such that there would be the option to charge higher premiums to building owners in Degirmendere than in Hereke for example.

Figure 7-1: Individual damage distributions of surveys carried out following the 1999 Kocaeli earthquake, compared to the average distribution. (Source of data, Booth et al., 2004)

7.1 Recognising and defining uncertainties

The uncertainties in regional loss models are greater than in single site risk models, since regional studies require a number of simplifications to represent the built environment without modelling each individual building, site or facility separately. There are further uncertainties related to the vulnerability of each building type and to the repair costs calculated for estimation of earthquake losses. In addition to these modelling uncertainties are those implicit in any estimation of seismic hazard. As noted by Wong et al. (2000) “Uncertainties are, quite simply, an indispensable part of any risk model”.

Adding liquefaction to a loss model compounds these uncertainties. The seismic hazard uncertainty is carried into the assessment of the likelihood of liquefaction triggering, which has its own modelling and parametric uncertainties, and the building vulnerability definition also has to include the uncertainties related to the relationship between ground failure-induced soil movements, structural damage state and repair cost. Furthermore, only limited empirical data of
Chapter 7

Key issues

Liquefaction damage exist for the validation of any assumptions made or for calibration of theoretical models.

The case studies presented in Chapters 5 and 6 show that these uncertainties cannot be eliminated, although they can be either reduced or quantified by certain actions, such as increasing the quantity of geotechnical investigation, using more site classifications, and investigating the building stock in order to improve the foundation classification in the inventory database. Any steps to reduce the uncertainties associated with ground failure-induced damage will necessarily increase the expenditure, so the best possible understanding of their relative significance and the relative sensitivity of the estimated loss model to this component is essential.

The term 'uncertainty' can be used in many different ways, and the terminology can often be confusing. A consistent reference framework is sought in this chapter, although, as observed by Baecher & Christian (2003) regarding the use of terms like 'uncertainty', 'randomness' and 'chance': “Implicitly, we think we know what those terms have to do with, but sometimes the more we think about the sorts of uncertainties that are part of an engineering analysis, the less clear they seem to become.”

7.1.1 Aleatory and epistemic uncertainties

The following terms need to be clearly defined in order to put subsequent discussions of uncertainty within the consistent framework:

**Aleatory uncertainty** relates to randomness and natural variability. The term is used to reflect underlying physical randomness that cannot be reduced. Tossing a fair (unloaded) coin is an example of an aleatory process. The term comes from the Latin *aleator* meaning 'gambler'.

**Epistemic uncertainty** relates to our lack of knowledge. Dealing from a given deck of cards is an example of this type of uncertainty. The order of the cards is fixed and the same cards will be dealt regardless of who deals them, and when and where. Therefore the only reason for the uncertainty regarding the next card to be dealt pertains to our lack of knowledge (Baecher & Christian, 2003). In an engineering context, the significance of epistemic uncertainty is that it can be reduced, for example with appropriate tests or investigations. Selection of a modelling approach is an example of epistemic uncertainty, which can only be reduced by future research and calibration of analytical approaches. The term *epistemic* comes from the Greek word for 'knowledge'.
In practice, the separation between the two types of uncertainty is by no means clear. In geotechnical problems, it is often convenient for modelling purposes to represent certain variables, such as spatial variability of in situ strengths within a homogeneous soil layer, as random, and therefore irreducible, even though in reality the values could be established with very extensive investigation and testing programmes. Both types of uncertainty are an intrinsic part of engineering problems.

7.1.2 Probability as a measure of uncertainty

Uncertainties and the effect that they will have on predicted results may be quantified as probabilities. In a certain world, known input parameters and the correct modelling approach would produce a single answer that would tell us whether a design will fail or not. In an uncertain world where the same deterministic approach is used, the factor of safety is employed to implicitly allow for the effect of the model and parameter uncertainties. However, when the variability of soil properties, the uncertainty associated with the definition of the design ground motions, and modelling uncertainties are all allowed for, a distribution of possible outcomes will be obtained instead of a single answer. These outcomes are quantified in terms of their probability of exceedance, which in the case of liquefaction assessments is the same as a probability of failure. In an engineering context, these probabilities are defined using two fundamentally different concepts, described below.

Frequency-based probabilities are calculated from the relative frequencies of occurrence of a particular event from a number of trials or samples. This approach is used to represent randomness, or natural variability. Repeated tosses of a coin will show that the frequency of occurrence of ‘heads’ tends towards 50%; this is a frequency-based probability of occurrence. The greater the number of trials the more certainty there will be that this frequency is the true frequency. Considering the case described above, where the spatial variability of in situ strength of a soil layer is modelled as random for convenience, sufficient investigation and testing within this layer would allow the mean and variance of the strength to be estimated.

Degree-of-belief probability is used for events, outcomes or decisions that are uncertain because they are simply unknown, such as when there are no repeatable experiments that could be carried out to determine their frequency of occurrence. The decision as to which modelling

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4 Jacob Bernoulli famously commented that “even the stupidest man” should intuitively grasp this concept (Gigerenzer et al. 1989).
procedure to use for a geotechnical problem cannot be established through repeated trials to
determine the relative accuracy of the available models. Only one trial is carried out, and that is
whether the design succeeds or fails during its design life. In these cases, the uncertainty
associated with the selection of analysis procedures is rated in terms of the relative likelihood of
each model being correct, based on engineering judgement, experience and intuition. Degree-
of-belief probabilities need somehow to be transformed from a personal judgement to a
numerical value, as illustrated in a simplistic way in Table 7-1.

**Table 7-1:** Suggested ‘degree-of-belief’ probabilities (after Vick, 2001)

<table>
<thead>
<tr>
<th>Descriptor</th>
<th>Probability</th>
</tr>
</thead>
<tbody>
<tr>
<td>Virtually certain</td>
<td>0.99</td>
</tr>
<tr>
<td>Very likely</td>
<td>0.9</td>
</tr>
<tr>
<td>Equally likely, no preference for either outcome</td>
<td>0.5</td>
</tr>
<tr>
<td>Very unlikely</td>
<td>0.1</td>
</tr>
<tr>
<td>Virtually impossible</td>
<td>0.01</td>
</tr>
</tbody>
</table>

An interesting paradox related to the two different classifications of probability is that complete
ignorance could lead to a subjectively defined probability of 0.5 (where in the absence of any
information, there is no reason to prefer one outcome over another) and near perfect knowledge
based on extensive data can also produce a probability of rating of 0.5 (Hertford, 2001). The
fact that these two cases have the same ‘probability’ warrants careful consideration.

The uncertainties related to liquefaction-induced losses arise from a number of different
sources, which do not fall exclusively into one or other of the above definitions. In some cases
there is even some overlap between the two. Best (2001) comments that “No statistic is perfect,
but some are less imperfect than others. Good or bad, every statistic reflects its creators’
choices.” For practical purposes, both types of probability are invaluable in risk assessments, as
observed by Vick (2001) “…Relative frequency approaches apply best to uncertainties in data,
degree-of-belief to uncertainties in knowledge, and the geotechnical field is uncommonly rich in
both varieties.”

Some of the uncertainties discussed in this chapter relate to the choice of empirical versus
theoretical modelling approaches. This is particularly true with respect to the large-strain
behaviour of soils, such as liquefaction and liquefaction-induced deformations. The complexity
of this behaviour lends itself to the use of empirically derived solutions. These solutions are
frequently preferable for loss estimations because they tend to be based upon easily obtainable
parameters and because complex numerical approaches are unsuitable. Nonetheless the associated uncertainty can be significant.

7.1.3 Impact of uncertainty on estimated earthquake losses

The uncertainty is what leads to the ‘distributedness’ of the estimated loss, rather than the single point value it would have if there were no uncertainties. This distribution is created by the aleatory variabilities associated with the hazard definition, and its spread is further increased by the epistemic uncertainties. For example, having to admit the possibility of more than one soil type beneath a group of buildings in a georef will lead to a wider possible range of occurrences. Figure 7-2 shows how the influences of variability in intensity of ground shaking and in the expected damage state combine to produce a distributed damage ratio.

![Illustration of distribution of variables in a typical damage function (Grossi, 2004)](image)

Figure 7-2: Illustration of distribution of variables in a typical damage function (Grossi, 2004)

Therefore, for any risk model, meaningful results can only be obtained through consideration of not only the mean losses but also the distribution about the mean. This information is essential for the determination of the impact of insurance conditions such as deductibles and limits, for example (Kumar et al., 2004). If modelling uncertainty means that estimations of permanent ground deformation due to liquefaction occurrence tend to be conservative, this conservatism will be passed to the estimated damage due to liquefaction.

7.2 Survey data from regions damaged by liquefaction

There is no better way to design and plan for future earthquakes than by evaluating the damage caused by past earthquakes (Dowrick, 1987)

Since the two 1964 earthquakes in Niigata, Japan, and Alaska there has been a continuing cycle of field investigations and developments in theory and analysis of liquefaction engineering,
illustrated in Figure 7-3. However, as demonstrated in Chapter 3, there are far fewer case histories with extensive data on liquefaction-induced damage than shaking-induced damage and many existing methods are largely calibrated to a few events. Furthermore, until recently, field surveys have not focused on collecting detailed damage data that can be used for investigating and calibrating regional loss estimations.

Figure 7-3: Development of solutions and theories in geotechnical earthquake engineering

Chapter 5 showed that much can be learnt from objective analysis of earthquake reconnaissance data such as that gathered in Adapazari. Nonetheless, there were shortcomings associated with these and other data from Adapazari, including the following:

- Bray and Stewart (2000) were the only surveyors who clearly separated structural damage and ground failure damage. In other cases it seems likely that surveyors made subjective judgements regarding tilted and settled buildings in order to assign a damage index. Furthermore, the data presented by Yoshida et al. (2001) focused only on buildings where there was evidence of liquefaction, leading to distorted results.

- The Ground Failure Index used for the Adapazari data is a useful and convenient tool for data in zones of liquefaction. However, by reducing observed ground failure to one of four categories, much potentially useful information is lost. Since it is not clear that the GFI relates directly to damage states and cost of repairs, it may be inadequate for determining losses in such zones.
The nature of repairs undertaken is not consistently reported, nor is information as to whether structures were repaired or demolished. In the context of earthquake-induced losses, this means that the full picture is not available.

Other issues identified in Chapter 3 as requiring more data include the foundation types of damaged buildings and the interaction of liquefaction and ground-shaking damage, which is occasionally mentioned, but rarely given detailed investigation. There is an enormous potential to develop the field of earthquake losses due to liquefaction further given an appropriate dataset. To this end, a detailed checklist has been developed that includes all the relevant data that are considered important in order to develop empirical correlations, presented in Appendix F. The research begun in this thesis could be taken much further on the basis of a focused field investigation following a damaging earthquake where liquefaction effects were observed.

### 7.3 Seismic hazard and site effects

Uncertainties relating to the definition of the seismic hazard and the site amplification will propagate throughout the subsequent calculations of earthquake losses. Assessments of liquefaction triggering and resulting ground deformations are dependent upon the strength of shaking. Therefore any uncertainties related to liquefaction-induced losses need to be considered in the context of these underlying uncertainties related to the demand. In keeping with the 'Principle of Consistent Crudeness' (Elms, 1985), there may be little to be gained from the reduction of the former, if the latter cannot be reduced.

#### 7.3.1 Seismic hazard

In Chapter 5, the ideal scenario for the purposes of the comparison of predicted and measured data would be to have a deterministic measure of the demand, with no uncertainty, thus allowing other components of the model to be directly investigated. However, the absence of a strong-motion record in the immediate vicinity of the survey zone meant that this was not possible. The close agreement between two independent studies of the surface response spectra shown in Figure 5-19 suggests that the estimations presented are reasonable, although this is not definitive. Crowley et al. (2004b) showed that the choice of ground motion prediction equations for damage calculations can have a very significant effect on the results.

The influence of the scatter in the ground-motion prediction relationships is also significant. If the median plus one standard deviation from the Gülkan and Kalkan (2002) relationships is used instead of the median, the short- and long period components of the surface response spectra
would increase by a factor of 1.7 and 2.1 respectively. Figure 7-4 shows the effect that this would have on the predicted damage distribution, where the mean damage ratio (MDR) is almost 2.5 times greater for the higher hazard level.

![Figure 7-4: Comparison of damage estimations for mid-rise RC frame buildings using the 'shear wall' vulnerability model, as described in Chapter 5, for the median and median +1σ estimated ground-motion levels for the Mw=7.4, d=7km](image)

The DBELA methodology, by treating the demand as a continuous distributed variable represented by its probability density function, deals with these uncertainties in a complete and rigorous manner. Alternatively multiple hazard levels can be considered for a given earthquake scenario, for example the median, median plus one standard deviation, median plus two standard deviations etc., with final answer being the weighted sum of each calculation. In HAZUS, the variability in the demand is represented in a more approximate manner through the spread of the fragility curves, but this is only true for ground shaking-induced damage. For liquefaction-induced damage, the spread in the fragility functions is related only to the expected variability of building response (see Section 4.10.5).

Related to the issue of the definition of seismic hazard, it is worth noting that the preference for scenario based loss estimations over those based upon probabilistic seismic hazard assessment, discussed in Section 2.6.4, is strongly supported in the context of ground failure. Earthquake magnitude and the source-to-site distance are both important variables required for the estimation of both liquefaction potential and the resulting ground deformations. Therefore, either a scenario based approach that explicitly defines the magnitude-distance pairs, or disaggregation of PSHA results, is essential for these purposes.

### 7.3.2 Soil amplification factors

The uncertainties associated with site amplification are inevitably significant, being related to the highly variable and complex behaviour of soil under large dynamic loads. As observed by
the US Army Corps of Engineers (1999) "Geotechnical problems often involve certain complexities not found in structural problems". Added to this is the act of grouping many soil profiles, each of which will respond differently under different levels of earthquake loading, into a manageable number of classifications. There are two sources of uncertainty, one in the assigned site classification, and one in the amplification factor used. Misclassification of a site does not automatically mean that the amplification factor is also incorrect, because both are in fact distributed variables represented by single values. Table 7-2 lists some of the main uncertainties to consider with respect to site effects,

Table 7-2: Sources of uncertainty in site classification and soil amplification

<table>
<thead>
<tr>
<th>Component</th>
<th>Source of Uncertainty</th>
<th>Type of uncertainty</th>
<th>Treatment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Site classification</td>
<td>Assigning site classes</td>
<td>Epistemic</td>
<td>Can be reduced by further investigation</td>
</tr>
<tr>
<td></td>
<td>Grouping different soil profiles into single site class</td>
<td>Aleatory</td>
<td>Can be reduced by increasing number of site classes and using more data, but generally treated as aleatory (i.e. fixed).</td>
</tr>
<tr>
<td>Site amplification</td>
<td>Use of factors which may not be appropriate to region</td>
<td>Epistemic</td>
<td>Can be reduced by developing region specific site amplification factors, as presented by Pappin et al. (2004) for Hong Kong.</td>
</tr>
<tr>
<td></td>
<td>Scatter within data from which factors were derived</td>
<td>Aleatory</td>
<td>Generally ignored, but can be incorporated where region specific factors have been derived.</td>
</tr>
<tr>
<td>Site response analyses</td>
<td>Selection of time histories, dynamic soil properties etc.</td>
<td>Epistemic</td>
<td>Reasonable to ignore these uncertainties compared to those related to site classifications mentioned above.</td>
</tr>
</tbody>
</table>

7.4 Structural damage model

The case study for Adapazari presented in Chapter 5 illustrated the uncertainties related to the estimation of damage caused directly by ground shaking. Because of these uncertainties it is presently difficult to calibrate predicted damage due to ground failure. The main issues raised in the first part of Chapter 5, which considered damage caused by ground shaking only, include:

- The complexity of the observed damage distribution in Adapazari, and the greater number of variables, which are inadequately reflected by the estimates obtained using the capacity spectrum approach.
• The difference in shape between the predicted and observed damage distributions. The capacity spectrum model could be calibrated to match the observations, for example by defining new fragility curves which, by lying closer together, would produce a damage distribution concentrated in the *undamaged* and *complete* damage states. This could not be done on an arbitrary basis however, and it is suggested that these new curves would require firstly a better understanding of the reasons for the observed shapes, followed by non-linear finite element analyses to produce new fragility curves.

An important conclusion from the Adapazari case study, at least with respect to the structural damage levels, is that if resources are limited, the initial focus of a damage estimation should be on refining the vulnerability parameters and capacity calculations, in order to improve the accuracy of the predicted damage. This relates to the conclusion in Chapter 3 that ground shaking is almost invariably the primary cause of earthquake damage to buildings overall. Efforts to reduce uncertainties in the ground failure component should only be made having dealt with this more fundamental component. Crowley *et al.* (2004b) confirmed, using the DBELA methodology, that a significant benefit exists in investing time and effort to define the vulnerability characteristics of the exposed building stock.

### 7.5 Probability of liquefaction

Available geotechnical data for loss modelling can be grouped into the following three scenarios, similar to the three grades of zonation proposed by ISSMGE (1999):

**Level 1**

No in situ data are available and assessment must be made on the basis of geological and topographical maps and local knowledge, as in the case of the HAZUS application to Adapazari presented in Chapter 5.

**Level 2**

Some in situ data are available, comprising existing boreholes and soil profiles as illustrated in Chapter 6.

**Level 3**

The project resources allow site investigations to be specified to match the requirements of the project.

Figure 7-5 presents options for treating liquefaction hazard depending on the level of geotechnical data available. In cases where only Level 1 data are available, it is recommended that a subjective, qualitative approach is used to define the liquefaction hazard. Such an approach will not produce quantitative damage estimations due to liquefaction, but since these have been shown to be unreliable, it is in fact preferable to avoid giving the appearance of a quantified result. Using Level 1 data will allow regions of potential liquefaction hazard to be
identified, and from the other components of a loss model, their relationship to building stock and potential losses can be defined. Subsequent work to quantify the liquefaction hazard should constitute a move up to Level 2 data.

**Figure 7-5: Treatment of liquefaction hazard in a regional context**

### 7.5.1 Sensitivity analysis

Figure 7-6 shows the results of a sensitivity analysis of the probability of liquefaction calculated using Equation 6-3 to variations in the uncertain input variables. This ‘tornado diagram’ shows the ‘swing’ when each best-estimate parameter in turn is replaced by an upper or lower bound value. For the SPT N values and the CSR (a function of $\tau_{max}$), these values are the 5th and 95th percentile values obtained from statistical analysis of the data presented in Chapter 6. There is an inherent fallacy, sometimes dubbed ‘the Law of Small Numbers’, in the assumption that a small sample of data is representative of the actual range of in situ values. SPT N data have been shown to be lognormally distributed. If a lognormal distribution is assumed for the SPT N values used to derive Figure 7-6 (as suggested by Phoon & Kulhawy, 1999 and Kulhawy & Trautmann, 1996, amongst others) then the corresponding Coefficient of Variation (COV) is approximately 0.17. This is towards the lower end of the range of values (0.15 to 0.45)
suggested by Kulhavy and Trautmann (1996), and below the range of 0.25 to 0.5 reported by Koutsourelakis et al. (2002). Nonetheless, reclamation fill material, being placed in a uniform and semi-controlled manner, may be reasonably expected to have less variability than naturally occurring materials, so the value of 0.17 may have some justification.

![Tornado diagram](image)

**Figure 7-6**: 'Tornado diagram' showing sensitivity of probability of liquefaction from Equation 6-3 at a depth of 5 m. Vertical line shows the best estimate value (P(L)=0.33), and horizontal bars show the ‘swing’ when the upper and lower bound values of each parameter in turn are considered. Upper and lower bound parameters are shown in curly brackets.

For the other variables shown in Figure 7-6, there is insufficient data to obtain frequency-based estimates, therefore the likely upper and lower bound values are selected through judgement. With reference to Table 7-1 these values have been selected on the basis of being ‘very likely’ to be exceeded, and ‘very unlikely’ to be exceeded. However, since they are based upon the judgement of the author alone, they cannot be considered definitive. Studies have shown that individual experts are likely to have an ‘over-confidence bias’ (Baecher & Christian, 2003), i.e. that they imagine things to be more certain than they are in reality, which would suggest that the ranges considered in Figure 7-6 may be underestimated.

The implication of Figure 7-6 is that a reduction in the uncertainty relating to P(L) is dependent to a large extent upon reducing the possible ranges in SPT N values and CSR. As noted above, the range of SPT N values is likely to be underestimated, if anything; however, since the results are constrained by the maximum probability of unity, this underestimation can only have a limited influence. Interestingly the two most influential parameters are also the ones for which variability cannot easily be reduced, other than by increasing the number of site classifications.
considered (they are also the only two that have been defined using frequencies rather than judgement). The other values, which could be reduced through more detailed investigation and testing, are shown to be less important. The magnitude, being a selected input parameter, does not have a variability associated with it per se, and the range considered is for illustration only.

This basic sensitivity study suggests that if Level 3 data are available for a study, the best allocation of time and resources may be in increasing the number of site classifications for the liquefaction analysis, or even by analysis of each individual borehole, such that the results can be statistically treated.

7.5.2 Proportion of liquefiable areas likely to liquefy

As shown in Chapter 6, for Level 2 data and above, the explicit consideration of the variability of soils through individual soil profiles, removes the need for a factor to represent the proportion of liquefiable soils expected to actually liquefy. However, for Level 1 data, such a factor, as represented by $P_{ML}$ in the HAZUS methodology (Table 4-6) may be useful. Various predicted and observed areas of liquefaction are presented in Table 7-3. These values suggest that the factors incorporated into the HAZUS methodology are within an appropriate order of magnitude, but fail to allow for the probable variability in the field.

<table>
<thead>
<tr>
<th>Table 7-3: Percentage of liquefiable areas likely to liquefy</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Source</strong></td>
</tr>
<tr>
<td>-----------------------------</td>
</tr>
<tr>
<td>Chapter 6 (Hong Kong data)</td>
</tr>
<tr>
<td>HAZUS</td>
</tr>
<tr>
<td>Adapazari survey data</td>
</tr>
<tr>
<td>Bray &amp; Stewart, 2000</td>
</tr>
</tbody>
</table>

7.6 Permanent ground deformation

Defining the susceptibility and probability of liquefaction occurrence, discussed extensively both in Chapter 6 and the previous section, is only the first step in a complete estimation of liquefaction-induced ground deformation. In fact, it is also the best defined, despite the
uncertainties that have been presented. The subsequent stages, of defining the amount of permanent ground deformation and the resulting building response and hence induced-damage, are both more complex and less certain.

Figure 7-7: Modes of lateral deformation (not including flow failures) (Seed et al., 2003)

Ground deformations due to liquefaction include uniform or differential settlements in level ground, uniform or differential lateral displacements towards slopes or free faces, or combined horizontal and vertical movements (see Figure 7-7 and Figure 7-9). Lateral movements tend to represent the greater hazard to buildings (Glaser, 1994). Whether deformations are uniform or differential is significant, particularly for buildings on ‘flexible’ foundations such as isolated spread footings or poorly reinforced continuous footings. Therefore not only does the amplitude of the average ground deformation have to be obtained, but also an estimation of the likelihood and expected magnitude of differential deformations across the footprint of buildings. This will depend on both the size of the building and the variability of the potentially liquefiable soil deposits. Figure 7-8 illustrates the process required to define the various demand parameters for building vulnerability analyses.

Existing methodologies for the estimation of permanent ground deformations have been presented in Table 2-8, and as explained in Chapter 2, these rely heavily on empirical data, and, furthermore, are generally accepted to only be accurate to within a factor of 2 or 3. Selection of the calculation methodology is best made on a case-by-case basis; with recommendations intended for universal applications there is a danger that regional features, that may affect their
validity in some way, are overlooked. For example, the HAZUS approach described in Section 4.10.4, combines the Liquefaction Severity Index (LSI) relationships derived by Youd & Perkins (1987b) with the attenuation relationships of Sadigh et al. (1986) to derive a relationship for LSI in terms of PGA only. Neither of these empirical relationships is applicable outside of the western USA and the former is only valid for a specific environment of “wide active flood plains or other areas of gently-sloping late Holocene fluvial deposits” (Youd & Perkins, 1987b).

**Figure 7-8:** Information required for a complete assessment of building vulnerability to liquefaction-induced ground deformations

In addition to settlements caused by volumetric strain, as predicted by Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992), vertical foundation movements may be caused by loss of ground due to sand boiling, the vertical component of lateral deformation of a volume of soil, or ‘punching’ failure of foundations due to reduced bearing capacity (Figure 7-9).

The importance of differential ground movements has been discussed in Section 2.9.4. An example of large differential settlements is those that would occur beneath buildings located on the boundary between liquefied and non-liquefied soils. Bardet and Kapuskar (1993) observed
that the worst damage in the Marina district after the 1989 Loma Prieta earthquake was on such a boundary. The uncertainty in the estimation of differential movements on a regional basis relates not only to the selection of the most appropriate calculation methodology, but also, more importantly, to the simplifications made in terms of geotechnical data; a borehole beneath each corner of a building would allow a reasonable estimation of the variability in expected liquefaction-induced ground deformations but would not be compatible with regional loss modelling. In the absence of this degree of detailed geotechnical data, for modelling convenience, it is suggested that the variability be modelled as random (i.e. aleatory), using a statistical distribution such as normal or log-normal, with a mean and variance. Thus the distribution of differential settlements over the footprint of a building, in terms of a percentage of the absolute settlements, can be obtained. If the expected differential settlement is represented in terms of a percentage of the absolute value, then for any given building class there will be a threshold below which settlements may, in the context of a loss estimation, be treated as uniform.

Figure 7-9: Modes of liquefaction-related vertical ground deformations (Seed et al., 2003)
As noted by Seed et al. (2003): “there is a need for improved ‘simplified’ analytical methods for engineering assessment of expected liquefaction-induced deformations and displacements.” Much of the current uncertainty in this field is compensated for in design by providing foundations capable of minimising the damage to the superstructure in the event of liquefaction-induced ground deformations, such as robust piles or stiff continuous shallow footings. However, for the purposes of assessment of existing structures, particularly for regionally distributed building stock, the absence of reliable “simplified” methods is a major shortcoming. Particularly, as further observed by Seed et al. (2003), “the quest for relatively ‘simple’ and reliable methods for prediction of liquefaction-induced settlements of shallow founded structures has been one of the most important and elusive objectives of research to fill ‘holes’ in our analytical repertoire.”

In accordance with the need for simplified approaches that can be applied over large areas and to many different building types, it is assumed that the liquefaction-induced ground deformation beneath foundations is equal to the free-field movements. For small to medium-sized structures, this assumption may result in slightly conservative predictions of damage (Boscardin & Cording, 1989). The assumption in fact permeates throughout liquefaction engineering, since many of the empirical methods for estimating ground deformations are based upon data from the free-field, but are used as input for engineering design. As well as the stiffness of the foundation potentially modifying the soil deformations, there is also the issue of the additional bearing pressure due to the presence of the building compared to the free-field. In Adapazari it was observed that there were few examples of free-field liquefaction despite the numerous cases of buildings affected by ground failure, suggesting that the higher shear stresses at the edge of buildings may have caused liquefaction to be more prevalent in these areas (Sancio, 2003). However, it is possible that free-field liquefaction occurred several metres below the ground surface with no obvious surface manifestation.

The uncertainties associated with defining the permanent ground deformation relate to the selection of which calculation methodology to use, all have inherent uncertainties due to the scatter in the data from which they were derived, and, more importantly, due to their limited applicability and accuracy. Further uncertainties, as in the previous two sections, relate to the in situ soil parameters used as input to the calculation approaches, in terms of insufficiency of data and thus the difficulty in accurately defining the thickness and extent of different stratigraphic layers, the natural variability of the soils, and the grouping requirement.
7.7 Building classification

The HAZUS ground failure component classifies buildings into two categories, either shallow or deep foundations (FEMA, 2003). This contrasts with the detailed building classification schemes used to define vulnerability due to ground shaking, such as that presented in Table 4-1, that consider height, age and seismic design level as well as the structural system. In Adapazari, surveyed buildings almost universally had robust shallow mat foundations. The use of weaker foundations such as spread footings would clearly have altered the damage distribution, in that buildings which tilted or settled as rigid bodies may instead have undergone differential settlement, which has a much higher potential to cause damage. Building classifications should, as a minimum, be extended to distinguish between flexible and stiff shallow foundations.

Buildings on flexible foundations that are subjected to differential ground movements across their footprint will respond differently depending on the nature of the structural system. Structures with some degree of flexibility, for example timber frames, should be able to accommodate greater deformation at their foundation level before exhibiting signs of damage. Brittle unreinforced masonry structures would begin to exhibit cracking at relatively low strain levels. Stiff structures may not deform differentially at all, even if they are not tied at the foundation level. Therefore the type of superstructure should also be included in the building classifications. Field observations have also shown a correlation between the building height-to-width aspect ratio and the damage level (Sancio et al., 2002), suggesting that building height should be considered in terms of defining vulnerability to ground failure.

For a portfolio of buildings, knowledge of the foundations will be more uncertain than of the structural system (which in itself requires some significant assumptions), short of carrying out a building-by-building survey. Any assumptions made in order to represent the foundation type necessarily add an additional level of uncertainty to the inventory, which must be considered with the results.

7.8 Building damage scale

The lack of a consistent scheme to quantify the degree of building damage due to ground failure hazard has been raised in Chapters 3, 4 and 5 of this thesis. The important issue is that according to most structural damage definitions, such as those presented in Table 4-2, a building that has experienced tilt or settlement with no structural or cosmetic cracking would be classed as undamaged.
The Adapazari field survey, used for the case study in Chapter 5, was carried out as objectively as possible and structural damage was recorded separately from the foundation damage. Some interesting observations have been made by Bray and Stewart (2000) and Sancio et al. (2002) through comparing the distribution of the two damage modes along the same survey lines. This could not have been done if a single unified scale had been used to record the damage. The Ground Failure Index used by Bray and Stewart (2000) therefore represents useful progress in this field. However, the ranges used in that scheme (Table 5-4) are not related to the ranges used to define structural damage and hence to the costs, therefore it does not fully satisfy the requirements of a regional damage estimation. In terms of presentation of results of damage estimations, particularly in terms of costs, it is preferable to align damage due to structural deformations caused by ground shaking and damage due to foundation movements resulting from ground deformations. Such a unified scale would greatly benefit the development and validation of loss estimation methodologies. This important issue is discussed further in Chapter 8 where a proposed unified damage scale has been developed.

Also related to the issue of damage scales is the use of the mean damage ratio (MDR) as a parameter to represent the overall losses. In Chapter 5, in accordance with the HAZUS methodology and the values adopted by Bommer et al. (2002), the MDR was derived as shown in Table 7-4. One of the interesting findings in Chapter 5 was the close agreement between the predicted and observed MDR, even though these were derived from very different damage distributions. This suggests that the MDR may be a less sensitive parameter than the individual damage states. However, whilst it is a very convenient parameter, use of the MDR alone can only be considered valid if it can be shown that it has been obtained from realistic results. Chapter 5 showed that this was not the case for Adapazari.

<table>
<thead>
<tr>
<th>Damage State</th>
<th>Slight</th>
<th>Moderate</th>
<th>Extensive</th>
<th>Complete</th>
</tr>
</thead>
<tbody>
<tr>
<td>Assumed Loss (% replacement value)</td>
<td>HAZUS</td>
<td>5%</td>
<td>10%</td>
<td>50%</td>
</tr>
<tr>
<td>‘Alternative’ (Crowley et al., 2004b)</td>
<td>15%</td>
<td>30%</td>
<td>100%</td>
<td>100%</td>
</tr>
</tbody>
</table>

The relationship between damage states and repair costs will be dependent on many factors, such as local construction practice or the deductible in the case of insurance costs. MDR relationships should be derived on a regional basis, using economic data related to damage and repair, which is a very significant task. In the absence of appropriate regionally developed (and up-to-date) relationships, there is a danger associated with over-reliance on the MDR, as demonstrated in Chapter 5.
Crowley et al. (2004b) investigated the sensitivity of the MDR derivation to changes in vulnerability and demand parameters. Importantly, it was shown that the variation in the estimated MDR was highly sensitive to the choice of cost ratios used to derive it and that the variation was non-linear depending on the changes made to the demand and capacity (Figure 7-10). Therefore different MDR relationships could have led to very different results in Chapter 5, as illustrated in Figure 7-11.

**Figure 7-10:** The percentage variation in the calculated MDR with respect to variations in the possible ranges of demand and capacity (Crowley et al., 2004b). The MDR has been calculated according to Table 7-4. The 'zero line' of the y-axis represents the best estimate values in demand and capacity.

**Figure 7-11:** Influence of the alternative MDR, described in Table 7-4, on the comparison between observed and predicted damage in Adapazari, for mid-rise RC frame buildings with infill walls, calculated according to the HAZUS methodology.

### 7.9 Building vulnerability to liquefaction

The determination of the impact that liquefaction-induced deformations will have on existing structures remains a 'grey' area both in research and practice. There are many reasons for this,
Chapter 7: Key issues

not least being that the preferred design approach is to mitigate the liquefaction potential by ground improvement or site re-location. Generally, a structural engineer will determine the deformation tolerances for a structure, and the geotechnical engineer will ensure through foundation design that these criteria are satisfied. There is presently a dichotomy between repeated warnings in technical literature regarding the damaging potential of liquefaction, and our current inability to estimate that potential. This issue is not new (e.g. Haldar & Leutich, 1985; Haldar & Miller, 1983) but remains unresolved, particularly for regional studies rather than site-specific design. From the perspective of earthquake loss estimations, avoidance of ground deformation hazards is not an option and therefore a method is required to estimate the magnitude and mode of the ground deformation and the resulting structural damage. The variables involved in this estimation include the volume and depth of liquefied soil, which will be related to the characteristics of the ground motion and of the soil profile, and the foundation type. Haldar and Leutich (1985) observed that a risk-based approach to estimate damage caused by liquefaction is essential because of the number of parameters involved and their expected uncertainty.

Chapter 3 presented a number of examples of building response to liquefaction, and the behaviour in Adapazari was examined more closely in Chapter 5. Undoubtedly the occurrence of liquefaction in the Adapazari survey area influenced the building response, and to ignore it would lead to incorrect prediction of the damage distribution, although observed discrepancies between predicted and observed damage are also related to other factors as noted above.

Since the simplified models reviewed in Chapter 5 were unsatisfactory in terms of reproducing the observed damage distributions in Adapazari, Chapter 8 proposes a framework through which building damage due to ground displacements can be estimated, using simplified relationships to determine the interaction of the soil and the structure, and considering also the increase in uncertainty that comes with the introduction of large strain behaviour of geomaterials to a problem.

7.10 Combined damage due to ground shaking and ground failure

Section 3.6.2 discusses examples in the field of ground shaking and ground failure interaction. The assumption made in HAZUS (see Section 4.10) is that the two hazards are not additive, i.e. that the presence of liquefiable soils increases the probability of a building being damaged, but that this damage state is only caused by one or other of the two hazards, not by their combination. A more appropriate combination, if the hazards are assumed not to interact, is
proposed in Equation 7-1. There are two possibilities with respect to combining damage caused by ground-failure in any given georef. These are:

i. The two hazards do not interact, therefore any group of buildings has a probability \( P(X) \) of being affected by liquefaction, and a probability of \( 1 - P(X) \) of being damaged by ground shaking. The final damage distribution can therefore be estimated as follows:

\[
P(DS) = P(L) \cdot P(DS|L) + (1 - P(L)) \cdot P(DS | Shaking)
\]  

(7-1)

where \( DS = \) damage state (e.g. slight, moderate, extensive or complete) \( P(L) = \) Probability of liquefaction. This should be most onerous combination of \( T_L \) (thickness of liquefied layer) and \( P(L) \) with respect to the damage distribution, therefore a number of scenarios may need to be considered.

ii. The two hazards do interact, i.e. the final damage state of any building is a function of both the initial damage caused by ground shaking and any subsequent damage caused by liquefaction. In this case the final damage distribution is calculated by Equation 7-2:

\[
P(DS) = P(L) \cdot P(DS|Shaking plus Liquefaction) + (1 - P(L)) \cdot P(DS | Shaking only)
\]  

(7-2)

Presently it is not possible to be definitive regarding interaction between the two modes of damage, and there is conflicting evidence from the field, it remains a complex and somewhat controversial issue. However, it can be concluded that there is insufficient justification for ignoring the possibility of the two hazards interacting at a single site. Despite the theoretical and observational support to the 'base-isolation' hypothesis, represented by Equation 7-1, to rely on this mechanism is inappropriate: it has been shown that damaging ground shaking can be transmitted to a structure prior to or during the occurrence of liquefaction (e.g. Berrill & Yasuda, 2002). There is therefore a scenario, which cannot be eliminated, whereby strong ground shaking causes some initial damage to a building, and, if the building has not already collapsed, the subsequent demand imposed on it due to liquefaction causes additional damage (\( P(DS|shaking plus liquefaction) \) in Equation 7-2). One suggested approach for including combined damage would be to calibrate building vulnerability to liquefaction with field observations such that the complexities of the hazard interaction are implicitly included. An alternative proposal for calculating the combined damage, in the event that the two hazards are believed capable of interacting at a site, is discussed further in Section 8.9. The interaction of the two hazards is a topic worthy of further research.
7.11 Discussion

Where earthquake losses due to potential ground failure are considered to be an important component of a model (e.g. with reference to Figure 3-25), there are important subsequent decisions to be made regarding calculation procedures to use and expenditure on additional data gathering. The discussions and flow charts presented in this chapter provide guidance as to where available geotechnical and foundation data at the start of a project fit, and how the rest of a project will be influenced by this in terms of uncertainties. Most importantly, despite the need for simple methodologies based upon readily available data for loss estimations, there is a level of data below which there is little or no merit in attempting to evaluate ground-failure induced damages because the uncertainties are so large as to render the results inaccurate.

With respect to the shortcomings of the HAZUS methodology, it is clear that the principal issues relate not to what the method does wrong in its treatment of ground failure, in terms of the assumptions and over-simplifications that it makes, but rather what it fails to do, namely:

- Different shallow foundation systems, which will respond very differently to ground failure, are not included.

- The thickness of the liquefiable layer is not calculated. In the absence of in situ data, to do so would be virtually impossible, which reinforces the need for in situ data.

- Different building responses to uniform or differential foundation deformations, and how these will vary with superstructure and foundation type are not considered.

- Natural variability of soil properties, epistemic uncertainties related to choice of calculation methodology, and relationships between uncertain input and predicted results, are all ignored.

The HAZUS method has, however, proved very useful in two particular areas; raising awareness that ground failure should be a component of all loss estimations unless there is specific information saying otherwise, and identifying each of the main steps required to estimate building damage due to ground failure. Nonetheless, in the light of the points listed above, in Chapter 8, an alternative procedure for estimating building damage due to liquefaction is proposed, with the specific objective of overcoming the weaknesses in the HAZUS approach. This procedure is significantly more data intensive, but nonetheless amenable to incorporation into future loss models.
8 BUILDING VULNERABILITY TO LIQUEFACTION

The assessment of structural damage due to ground shaking requires the definition of building capacity and vulnerability versus demand, as well as damage scales to describe the state of the damaged building. As has been demonstrated in the preceding chapters of this thesis, in the case of building damaged caused by liquefaction-induced ground deformations, neither of these is available.

Therefore, to overcome these shortcomings, the two main items presented in this chapter are: a proposed unified damage scale for all modes of building damage, for use in both reconnaissance and assessment; and an analytical procedure for the assessment of structural damage in buildings subjected to differential ground movements. The former relates mainly to buildings on stiff foundations, whereas the latter is applicable to buildings on flexible foundations. The analytical solutions have the important advantage of being fully compatible with the DBELA procedure for estimation of earthquake damage caused by ground-shaking described in 4.8.

This chapter develops the issues raised in Chapter 7 with respect to the ground deformation demand and the building classification. Subsequently the modes of building response to uniform and differential foundation displacements are described, followed by proposed solutions to these modes, in the form of an empirically derived damage scale for uniform, rigid body, displacements and analytical solutions for differential displacements. For the latter, their application in regional damage estimations is described, followed by a proposal for their combination with ground-shaking induced damage, and finally a case study of their application.

8.1 Ground deformation demand

The assessment of the probability of liquefaction on a regional basis has been discussed in Section 7.5. It is noted that probabilistic procedures are preferable in this context since they allow the uncertainty in the liquefaction assessment to be included. The selection of the most appropriate methodology for a particular study must carefully consider the required input data compared to that available.

Section 7.6 describes the requirements of a building damage estimation in terms of defining permanent ground deformation, and the reliance of commonly used assessment methods on empirical data was explained in Section 2.9.4. Figure 7-8 shows the process of quantifying liquefaction-induced ground deformations beneath a building; ultimately, the required input to a damage calculation is the probability of occurrence of each mode of deformation, and the
median value, standard deviation and assigned statistical distribution of its amplitude. Again, a probabilistic approach is required to capture the uncertainties associated with defining liquefaction-induced permanent ground deformations on a regional basis.

8.2 Building classification

The first requirement of any building vulnerability assessment is a classification system for the building stock, based on the premise that buildings grouped into a single category should have essentially the same expected mode of response and failure, and comparable levels of susceptibility to the earthquake hazard under consideration. As has been noted in the preceding chapters, the mode of ground failure-induced damage is very dependent upon the type of building, and most importantly on its foundation system.

8.2.1 Foundation classification

Structural response to ground deformations depends on whether the foundations are flexible, such that columns or walls are able to move differentially, or stiff, such that ground deformation will lead to rigid body movement of the foundation and superstructure.

The term ‘flexible’ foundation is used hereinafter to describe any foundation with little or no flexural stiffness, including individual pad or spread footings beneath columns; unreinforced or poorly reinforced strip foundations beneath load-bearing walls or unreinforced or poorly reinforced mat foundations. These foundations, simplified as infinitely flexible, are assumed to deform with the ground, therefore ground deformations will be imparted directly into the structural system.

‘Stiff’ foundations, are those which can be simplified to respond in an infinitely rigid manner to ground deformations, these include reinforced mat foundations or strip or spread footings with reinforced tie beams. Solutions for piled foundations, whose response will also vary according to the presence or not of a raft, and according to the type of pile, and the material in which they are founded, are not included in this chapter, although their treatment is an important area of further work related to this thesis.

Foundation types cannot be easily identified, even from visual surveys, and a greater depth of investigation is required. Nonetheless, information on the foundations is equally as important as in situ geotechnical data for the ground failure component of a loss assessment. Some assumptions could reasonably be made, for example that poorly designed buildings are likely to have poorly designed foundations, and well-designed modern buildings are more likely to have
considered liquefaction susceptibility in the foundation design and hence to have stiff or deep foundations. However, the unreliability of such assumptions is illustrated by the data from Adapazari in Chapter 5, where the buildings were generally of poor quality, and the foundations were, in comparison, surprisingly robust (Figure 8-1).

![Figure 8-1: (a) Overturning of a building on stiff shallow foundations in downtown Adapazari, (b) close up of foundations of the same building (Bray & Stewart, 2000)](image)

Any methodology for the estimation of ground-failure induced damage is likely to be applied to relatively small study areas, where it is known that there is a prevalence of liquefiable soils. Therefore, it is reasonable to assume that in most cases there will be sufficient information on the local building stock to make appropriate judgements regarding the distribution of foundation types. For a study area the size of Adapazari, for example, it is feasible to gather such information through discussions with local engineers and builders, whereas for a larger region, such as the TCIP model for the whole of Turkey (Bommer et al., 2002), such depth of investigation is highly unlikely.

The two foundation classifications described above, representing the extreme cases of infinitely flexible or infinitely rigid shallow foundations, do not allow for the relative stiffness of the soil-structure system (e.g. ISE, 1989), and as such represent a significant simplification, in accordance with the nature of loss estimation studies. Assessment of the relative stiffness, and its likely influence on the observed foundation deformations, requires parameters beyond the data generally available in such studies. Nonetheless, it should be recognised that in reality, the relative stiffness of the building and the soil column will influence both the mode of the response of structures to ground deformation, and the mode and magnitude of the ground deformation beneath the structures.
8.2.2 Structural classification

Whilst the foundation system will dictate which mode of damage is incurred as a result of ground deformations, the structural classification will influence the ultimate damage state. Brittle structures will be less tolerant to differential deformations at foundation level than more ductile ones. The damage states of buildings on stiff foundations are less dependent upon the type of superstructure since they comprise rigid body rotations originating at foundation level (e.g. Figure 8-1).

8.3 Building response to liquefaction

The simplified modes of response to ground deformations for any given structural and foundation classification are summarised in Figure 8-2.

![Diagram](Figure 8-2: Summary of different modes of liquefaction-induced damage within one building classification.)

When seeking a solution procedure, the damage can be subdivided into two categories. The first, where there is sufficient flexibility in the foundations for the walls or columns to move independently, and thus differentially (e.g. Figure 8-3), is referred to as ‘structural damage’, because damage occurs in the structural elements. Analytical solutions can be developed in this case, because the damage is related to the deformation induced in the structure. The second category is ‘rigid body’ building response, whence the important consideration for determining the damage state is the acceptability of the system performance. For example, for uniform
settlement, issues such as the relative displacement to street level, the deformations of service connections, and the difference in level to adjacent buildings will determine whether remediation or demolition are necessary. The damage state cannot be easily classified in terms of the deformation of the structural members, therefore there is no apparent analytical solution, and an empirical solution is required.

The review of earthquake damage in Chapter 3 revealed a much larger number of occurrences of rigid body damage than of structural damage in the event of liquefaction. Hundreds of buildings were reported to have settled or tilted in the earthquakes in Turkey and Taiwan in 1999, Luzon, Philippines in 1990 and numerous others. A notable example of the second type of failure is the complete damage of the low-rise wooden structures of the Marine Laboratory at Moss Landing following the Loma Prieta earthquake in 1989 (Mejia et al., 1992). Also, Marino (1993) reported that "houses on perimeter footings were most susceptible to damage from liquefaction" in the 1989 Loma Prieta earthquake, whereby the bending and breaking induced in the foundations caused racking of house frames. Other cases were reported in the Kocaeli and Chi-Chi earthquakes (e.g. Figure 8-3). Notwithstanding the scarcity of case histories, this failure mode clearly represents a genuine hazard to buildings with flexible foundations. Marino (1997) notes that the most common earthquake induced foundation damage is separation of foundation elements, resulting from the irregularity of foundation settlement due to lateral spreading, inducing tension, bending or tilting in shallow foundations.

Figure 8-3: 'Structural damage' caused by liquefaction-induced ground deformations in the 1999 Chi-Chi earthquake. Lateral spreading and associated settlement towards the river bank (in the rear of the photo) caused structural deformations and damage in both buildings shown, which appear to be founded on spread footings. (Photograph courtesy Marshall Lew, Los Angeles Tall Buildings Design Council)
The smaller number of reports of structural damage due to differential foundation movements may be because in many cases the relative stiffness of the soil-structure system may be sufficient to cause structures to respond as rigid bodies. Alternatively, it is possible that, because the damage more closely resembles damage caused by ground shaking, in that the damage comprises cracking and yielding of structural members, it is not always immediately identified as being due to ground failure. However, this explanation seems improbable considering the buildings shown in Figure 8-3 for example. A further postulated explanation is simply that such types of damages have occurred, but have not been the focus of field-investigations and have therefore not been given particular significance in reports.

### 8.4 Rigid body response to ground deformation

This section describes and illustrates modes of response of buildings on stiff foundations (assumed infinitely rigid) or buildings undergoing uniform settlements.

#### 8.4.1 Foundation settlement

In the case of settlement of a liquefied layer with little or no differential deformation across the footprint of a building, the damage to the structure will be low; the most likely damage being cracking or failure of the ground floor slab and sand ejecta filling the ground floor for buildings on perimeter or spread footings (see Figure 8-4).

![Illustration of damage to buildings on perimeter foundations due to uniform settlement.](image)

**Figure 8-4:** (a) Non-structural damage to interior floor slab due to settlement of columns, and bulging of underlying soil, Adapazari, Turkey (Bray & Stewart, 2000) (b) Schematic illustration of damage to buildings on perimeter foundations due to uniform settlement.

For the structure shown in Figure 8-4 there appears to be no damage to the main load-carrying system, although there may be weakening due to loss of diaphragm action. If load-bearing columns settle uniformly on their foundations, non load-bearing elements such as partition walls or floating floor slabs may not move by the same amount, causing extensive non-structural
damage. Ultimately the damage state will be related to the extent of settlement and ease of repair.

Figure 8-5: (a) Uniform settlement of a building, apparently on stiff foundations, Adapazari, Turkey (b) Schematic illustration of response of buildings on stiff foundations to uniform settlement.

Figure 8-5 illustrates the response of a building on a stiff raft foundation to uniform vertical settlements. The damage state of such buildings is a function of aesthetics, serviceability and reparability. Differential settlements beneath stiff footings cause rigid body rotation, as illustrated in Figure 8-6, such as was widely observed in Adapazari.

Figure 8-6: Schematic illustration of the response of a building on stiff foundations to differential settlement at foundation level

8.4.2 Lateral spreading

The horizontal spreading of soil on gently sloping ground or towards a free face is a common feature of liquefaction. In most cases there will be a vertical component, albeit very small in some situations, to the lateral movement. This is the case both geometrically (if the lateral
movement is on even a very shallow slope, then any foundation displacing horizontally will also have to move down, as shown in Figure 8-7), and also physically (post-liquefaction volumetric strains will cause settlement, even in level ground).

![Figure 8-7: Lateral spreading of soil beneath a building on a raft foundation, causing horizontal displacement and some vertical movement. Piled buildings may be able to resist the displacements and remain fixed, thus restricting the movement of neighbouring structures.](image)

Buildings on flexible foundations may also displace uniformly; the necessary conditions for this to occur would be for the lateral spread to be large in comparison to the building footing, and for the building footprint to be entirely within the lateral spread. However, a more probable scenario for buildings on flexible foundations is that they experience some differential lateral deformations, as discussed in Section 8.5.2.

Stiff foundations are unlikely to undergo differential horizontal movements, since the forces would have to exceed the tensile capacity of the foundations. In practice it would be more likely that the entire structure would displace uniformly in the direction of soil movement by an average amount.

### 8.4.3 Repair of rigid body response to ground deformation

The necessary repairs and hence the costs of rigid body settlements and rotations will be largely governed by issues of acceptability, serviceability and ultimately safety. If damage is deemed to be only cosmetic, i.e. aesthetically altered but affecting neither the serviceability nor the stability of the structure, then the repair costs will be relatively low. For any damage above this state, whereby the settlement and/or tilt require remediation, the costs of repair are significantly increased, since this remediation will require re-levelling (e.g. by excavation beneath the foundations and jacking, or by grout injection as described by Diaz et al., 1998) and foundation repair. For very large settlement and tilt, buildings may need to be demolished and rebuilt.
For buildings that have settled uniformly (e.g. Figure 8-5), repair solutions will need to consider disruption to services, damage due to dragging down adjacent attached buildings and serviceability issues related to change of levels of doors and entrances. Additionally there may be aesthetic considerations due to level differences between a settled building and its surrounding infrastructure. In the majority of cases where a building has settled uniformly, its functionality can be restored at relatively little cost, by repairing services, modifying entrances etc. If the settlement is particularly significant it may be necessary to carry out structural remedial works in the form of jacking-up and underpinning. In the worst cases, buildings may be considered damaged beyond repair due to uniform foundation settlement. Tilted buildings, as opposed to those settling uniformly, are more likely to require repairs to rectify the tilt. As before, the effects may be aesthetic only, i.e. perceptible to the inhabitants despite not having any impact on the functionality; or they can affect the serviceability. In the worst cases the structural stability will be affected.

Issues that complicate the estimation of the likely cost of repair of buildings that have undergone rigid body deformation include:

- The foundations need to have sufficient strength to be able to survive the planned remedial action, and it may not always be easy to determine this. If the foundation system requires removal and replacement then the costs can equal or even exceed the value of the structure, particularly for low-rise residential structures (Marino, 1997).

- The issue of restoring the building and its foundations to its pre-earthquake state (referred to as ‘like kind and quality’, LKQ, in insurance terms) as opposed to improving the soil or the foundations to prevent reoccurrence of the same damage in a future event. Undertaking ground improvement will significantly increase the repair costs, particularly since geotechnical investigation may also be required.

- The selection of the most appropriate repair method for damaged foundations will involve a combination of construction solutions, construction resources and costs. Regional variability is illustrated by two apartment blocks in Figure 8-8 and Figure 8-9; the former, in Turkey, was demolished, whereas the latter, in Taiwan, was repaired. Repair versus replacement decisions need to consider the salvage value, costs of demolition, temporary relocation expenses during repair or rebuilding, the effect on surrounding buildings and administrative costs, and are therefore very difficult to determine, particularly on a regional basis for groups of buildings.
• The tolerance of the building owners to distortion. As an extreme example, it was reported that buildings in Dagupan that settled by as much as 2 m after the 1990 Luzon earthquake continued to be occupied after the event (Schiff, 1991).

There is a need for more data on the cost of repair and remediation of buildings that have settled or tilted. Only a handful of cases have been reported in the literature (e.g. Diaz et al. 1998; Ono et al., 1998; Sumi et al., 1998; Hwang et al., 2003), and these do not provide much information on the cost.

Figure 8-8: Building in Adapazari that experienced a differential settlement of approx 140cm (approx. 1 in 10, or 5 degrees tilt). This building was subsequently demolished. (Photograph courtesy EEFIT)

Figure 8-9: Apartment building in Wu-feng, Taiwan, after the Chi-Chi earthquake in 1999. a) after the earthquake, showing 9 degrees tilt due to liquefaction beneath the foundation. b) after repair, using oil jacks to re-level the building. (Photographs courtesy Professor J-H Hwang, National Central University, Taiwan)
8.5 Structural damage caused by differential movements beneath flexible foundations

This section describes the deformation of buildings on infinitely flexible foundations, subjected to differential movements at foundation level. The illustrations presented in this section, showing two-dimensional deformed shapes of RC frame buildings subjected to differential ground movements, have been obtained analytically, using the structural analysis software SeismoStruct (SeismoSoft, 2004). SeismoStruct is a finite element software package capable of modelling the large deformation behaviour of space frame structures subjected to either static or dynamic loading. In this case the loads were applied as incremental displacements to the foundation nodes. The analysis takes account of both local (beam-column effects) and global (large displacement/rotation effects) geometric non-linearities as well as material inelasticity. The spread of material inelasticity along the member length and across the section area is explicitly represented through the employment of a fibre modelling approach, used in the formulation of the inelastic beam-column frame elements employed in the analysis.

The calculations in SeismoStruct used the program’s ‘wizard’ function for two-dimensional analyses of frame structures. Parametric studies were carried out to allow for variations in building geometry and foundation displacements, and it was shown that the deformed shapes were essentially the same irrespective of these variables, providing confidence that the analytical solutions presented in Section 8.7 are appropriate for general application. Figure 8-10 shows sample outputs from SeismoStruct analyses for applied displacements at a footing node.

![Diagram](image)

**Figure 8-10:** Example computed deformed shape from SeismoStruct (SeismoSoft, 2004) for a single-bayed, RC frame, with 3m storey height and 3 or 4 m beam lengths, subject to (a) 1.0 m vertical settlement beneath one footing, and (b) 0.5 m horizontal, 0.5 m vertical displacements. These analyses assume that the fixed footing is fully restrained, and the displaced footing is unrestrained in the direction of applied displacements.
Figure 8-11: A timber and brick building believed to be on shallow spread footings in Adapazari, Turkey, 1999. Differential settlement of columns, ground floor heave, and settlement relative to the sidewalk caused the building to be demolished (Sancio, 2003).

8.5.1 Differential settlements

Differential deformations over the footprint area of a building will occur due to variability in the stratigraphy of the underlying soils and in the soil properties within each layer. Soil deposits are rarely homogeneous, and therefore in the field it is unlikely that even within a fairly small area settlements would be uniform (Ishihara & Yoshimine, 1992). Differential settlements between load-bearing columns on flexible foundations have the potential to cause significant damage to a structure, as shown in Figure 8-11.

Figure 8-12: Idealised response of a single-bayed RC frame building with flexible shallow foundations to differential vertical settlements (not to scale)

Figures 8-12 and 8-13 show the idealised response of RC frame buildings to vertical displacements imposed at foundation level. The maximum demand ($\Delta_D$) on the columns is shown to be the same for the single-bayed frame as for the multi-bayed frame, meaning that the same analytical solutions can be used for these two cases. Significantly, the deformations can be seen to take place in the columns rather than the beams, and to be concentrated in the ground...
floor columns, with the higher storeys, irrespective of storey height or number of storeys, rotating as rigid bodies. This occurs because the columns are free to rotate laterally and therefore behave as cantilevers in simple bending; the formation of plastic hinges at the base of the building is the simplest failure mechanism.

Initially, it was anticipated that the vertical deformation would be taken up by the beams, i.e. repeating the pattern at ground level in the upper floors. This deformation mode was found to occur only for cases such as that shown in Figure 8-14 where there is a restraint to keep the columns vertical. In the cases where there is no restraint, the flexural stiffness of the beams is such that they do not deform but move laterally.

Parametric analyses in SeismoStruct showed that the beams would have to be unrealistically flexible before a failure mechanism involving flexure of the beams would occur. Different analytical solutions are therefore required for multi-bayed structures experiencing the largest settlements at the end of the frame compared to those where the maximum settlements occur beneath an internal footing. In the framework of a loss estimation, it is suggested that this complication may be incorporated by subjectively assigning a proportion of the building stock to each behaviour mode, since it will clearly be impossible to predict where the maximum differential settlement is expected to occur.

The effectively rigid body response of the upper storeys of the buildings in Figures 8-12 and is potentially significant, since there may be cases where the acceptable rotation limits in terms of serviceability and stability, as presented in the next section, become the critical criteria.

![Figure 8-13: Idealised response of a multi-bayed RC frame building on flexible shallow foundations to differential vertical settlement, with approximately equal differential movements between each bay (not to scale)](image)
8.5.2 Lateral spreading

The failure mechanism for buildings subjected to differential lateral movements is shown in Figure 8-15. The deformation is once more concentrated in the ground floor columns, because the axial stiffness of the beams is much higher than the flexural stiffness of the columns.

Parametric studies in SeismoStruct showed that only a very small vertical component (approximately 1% of horizontal) is required to cause the deformation mode shown in Figure 8-15b, where the deformation is concentrated in one column, as opposed to that in Figure 8-15a, where it is evenly distributed between the two ground floor columns. The differential settlement of the right-hand column in Figure 8-15b causes a redistribution of loads, such that the load on the left-hand column is reduced, which is why only a small amount is required to produce this mode. In reality, there is likely to almost always be a vertical component, as noted in Section 8.4.2, therefore only this mode (Figure 8-15b) has been considered.

Figure 8-15: Idealised response of an RC frame building with flexible shallow foundations to lateral movements (a) without and (b) with an associated vertical component (not to scale)
Although the presence of a vertical component was found to influence the deformed shape of the structure, the damage state was found only to be governed by the amplitude of the horizontal foundation deformation: i.e. given that the vertical deformation occurs, and that it is not greater than the horizontal component, its actual amplitude is not important.

### 8.6 Building damage states

Where building response to ground failure comprises structural damage, as in the previous section, damage states can be classified using the same schemes used for structural damage caused by ground shaking (e.g. Tables 4-2 and 5-3). These damage states, representing defined ranges, are more practical than continuous damage scales for analysis and presentation of the results of damage estimations. By describing the expected damage levels, they also provide users with an understanding of the physical condition of the buildings. The absence of rigid body response, such as tilting, settlement, or sliding, from most existing damage scales has been discussed in previous chapters of this thesis. Even where there is no damage to the structural elements of such buildings, they must be considered to be damaged in terms of the definition of damage as "a change in the condition of the structure that adversely affects its future structural performance" (Kehoe, 1998).

As mentioned previously, there is no analytical approach for the estimation of damage levels of buildings that have rotated as a rigid body, since there is no structural demand (apart from possible P-Δ effects at larger rotations). Therefore the damage state of a rotated or settled building is best described using empirical solutions, to classify the damage level in terms of the functionality and reparability of the building.

The boundary between structural and non-structural damage is not easily defined in the case of rigid body response. Small movements, as noted in Section 8.4.3, are primarily an aesthetic issue; these could be considered as equivalent to building façade damage for example, which is generally classified as non-structural damage. However, building façade damage is always classified as non-structural, whereas in this case the mode of damage is the same as that which is considered structural when it is sufficiently large, therefore a single scale is more appropriate. Damage to services and connections may always be classified as non-structural, and is therefore not incorporated into this scale.

### 8.6.1 Requirements of earthquake damage scales

The main requirements of earthquake damage scales that group response to earthquakes into similar ranges are:
i. To provide a descriptive definition of the damage state, such that from the end results of a loss estimation there can be an understanding of the nature and extent of the predicted damage.

ii. To enable estimated damage levels to different building classifications to be presented in comparable terms.

iii. To assist those undertaking field damage surveys in rating individual buildings using a common scale and therefore to enable comparisons between survey zones and between earthquakes.

For consistency with the structural damage levels such as those described in Table 4-2, both rotational and settlement limits must relate to the repair cost ratio. However, determining the cost of foundation repairs is significantly more complex than for structural repairs (e.g. Marino, 1997). Variables such as the study region, its construction practices, and the attitude of the building inhabitants will all contribute to the definition of what constitutes a serviceable building and hence to the extent and cost of necessary repairs. For example, in the UK, homeowners find tilts as low as 1 in 250 unacceptable, even though such small rotations will not affect either the serviceability or the structural integrity of a low-rise building (BRE, 2003). Marino (1997) on the other hand, writing from the perspective of earthquake-induced damage, states that "not many people notice a half a percent of tilt" (i.e. 1 in 200).

8.6.2 Proposed damage scale

A new damage scale is proposed, representing rigid body settlements and rotations in addition to structural damage, for use in future damage estimations and field surveys. This is shown in Table 8-1. The scale has been derived with reference to the following sources, plotted in Figure 8-16:

- Publications that present guidance on acceptable limits for tilted building in non-earthquake situations (e.g. BRE, 2003).
- Observations of damage in the field, such as Seed & Idriss (1967) from the Niigata earthquake, Bray & Stewart (2000) from Kocaeli or Park et al. (1995) from Kobe.
- Guidance developed for evaluating damage levels (Horie et al., 2000; Watabe & Yamanobe, 1996; Youd & Perkins, 1987b).
- Published case studies, describing the post-earthquake repair procedures for settled and tilted buildings. There are only a handful of these, e.g. Diaz et al. 1998; Ono et al., 1998;
Sumi et al., 1998 and Hwang et al., 2003. Sancio (2003) records whether the buildings he studied in Adapazari were re-inhabited immediately, repaired then re-inhabited, or demolished.

A significant degree of judgement was required to derive the suggested limits from the range of values that these sources produced (see Figure 8-16 and Table 8-1). This judgement relates not only to the selection of an appropriate value within the range, but also to the interpretation of the published limits and case histories, and of what their authors' consider to constitute different damage states. Geddes (1984) observed that ground movements beneath buildings “occur as a result of many factors and influences, and attitudes to their severity are likewise conditioned by a large number of factors – technical and human, aesthetic and financial”.

Figure 8-16: Summary of data used to derive proposed unified damage scale, for rotation (upper figure) and uniform settlement (lower figure). Dotted lines show the limits proposed in Table 8-1.
In Figure 8-16 it is noteworthy that the limits proposed by BRE (2003) for acceptable tilt of low-rise buildings in the UK represent the lower bound of the values shown. The boundary between extensive and complete damage is particularly uncertain, since what is judged to merit repairing rather than demolishing and rebuilding is highly variable. The data in Figure 8-16 do not include the many buildings in Adapazari, Dagupan and elsewhere that had rotations in excess of 45 degrees, frequently only prevented from full overturning by the presence of adjacent buildings, since it is the lower bound to this damage state that is of interest. These observations were considered in defining the boundaries however.

The limits shown in Table 8-1 have a very high associated uncertainty due to the lack of data regarding repair methods and costs for settled and rotated buildings, and require further field investigation for calibration (see Section 7.2 and Appendix F). Nonetheless, the scale represents an important development in the context of earthquake losses due to ground failure since it will enable comparison of damage in different regions and different earthquakes as well as provide improved descriptors of predicted damage distributions.

Table 8-1: Limit states for rigid body settlement and rotation due to earthquake-induced ground deformations for RC frame buildings

<table>
<thead>
<tr>
<th>Damage State</th>
<th>Description of structural damage (Table 4-2)</th>
<th>Description (basis for rigid body deformation limits)</th>
<th>Settlement (Δ) only</th>
<th>Rotation only</th>
</tr>
</thead>
<tbody>
<tr>
<td>Slight</td>
<td>Linear elastic response, flexural or shear type hairline cracks, no yielding in any critical section.</td>
<td>Repairs may be necessary for aesthetic reasons</td>
<td>Δ ≤ 0.1m</td>
<td>≤ 1/100 ≤0.6°</td>
</tr>
<tr>
<td>Moderate</td>
<td>Member flexural strengths achieved, limited ductility developed, cracks widths reach 1.0mm, initiation of concrete spalling.</td>
<td>Repairable damage, serviceability and/or functionality affected</td>
<td>0.1m &lt;Δ ≤0.3m</td>
<td>1/100 to 4/100 0.6° &lt; θ ≤ 2.3°</td>
</tr>
<tr>
<td>Extensive</td>
<td>Significant repair required, wide flexural or shear cracks, buckling of longitudinal reinforcement may occur.</td>
<td>Uninhabitable, but repairable</td>
<td>0.3m &lt;Δ ≤ 1.0m</td>
<td>4/100 to 8/100 2.3° &lt; θ ≤ 4.6°</td>
</tr>
<tr>
<td>Complete</td>
<td>Repair not feasible either physically or economically, demolition after earthquake required, could be due to shear failure of vertical elements or excess displacement.</td>
<td>Demolition cheaper than repair. Structural integrity affected, possible instability</td>
<td>≥ 1.0m</td>
<td>≥8/100 θ ≥ 4.6°</td>
</tr>
</tbody>
</table>

Unlike the empirically based rigid body limits, the structural damage definitions in Table 8-1 can be related to the limit state strains such as those presented in Table 8-3.
8.7 Assessment of damage caused by differential foundation movements

Existing methods for the assessment of building damage due to ground deformations mostly relate to damage caused either by long-term consolidation settlement (e.g. Burland & Wroth, 1975) or to movements induced by excavation or tunnelling (e.g. Boscardin & Cording, 1989). These methods typically fall into one of two groups, those using empirical data from case histories to determine allowable settlement limits, or methods that apply structural engineering principles; Burland and Wroth (1975), for example, modelled building walls as simply supported beams, considering deformation modes and critical tensile strains to determine induced damage. In the context of earthquake-induced liquefaction, which may affect a significant percentage of the building stock in a region, and which may generate differential ground movements up to at least 0.5 m, these methodologies provide some useful background, but have only limited applicability.

The procedures presented in this section have the advantage of being derived from mechanically sound principles, using simple geometric and material variables. The building damage states are directly obtained from the calculations, and neither judgement nor empirical data (apart from that related to the definition of the input) are required to define the damage distribution.

The structural damage states, shown in the first two columns of Table 8-1, were fully described in Table 4-2. These damage states are similar to those employed for the damage estimation for Adapazari described in Chapter 5, with the exception that no distinction is made between none and slight damage. The reason for this is that greater than slight damage is defined by exceedance of the yield strains, as will be described further below. When a structure is below its yield point, it is not possible to accurately determine the boundary between no damage and slight damage in the calculation methodology. This is confirmed by the observations in Chapter 5, where some overlap was observed between these two states depending on the report.

The analytical procedures presented in this section consider RC frame buildings (ignoring infill walls) with flexible footings. This single building class is then subdivided into different height ranges (for ground shaking response) and into two levels of seismic design; poor and good. For RC frame buildings, the latter classification depends upon the level of confinement and the consequent failure mode due to ground shaking, where poor buildings are poorly confined, with lower limit state thresholds, and an increased likelihood of soft-storey (column sway) failure. Finally, consideration must be given to the geometric and material properties of a given group of buildings (e.g. RC frame; spread footings; mid-rise; poor seismic design), which will influence their capacity to resist earthquake damage. The variables in this method are
represented in a probabilistic framework, where the capacity parameters are assigned a probabilistic distribution that is a function of the uncertainty associated with the geometric and material properties.

8.7.1 Analytical solutions to liquefaction-induced damage: key features

An analogy between displacements applied at foundation level and lateral displacement-based assessment procedures now commonly used in earthquake resistant design and assessment (e.g. Priestley, 1997) forms the basis of these solutions. The formulations are derived within the framework of the DBELA methodology described in Section 4.8, where the original displacement capacity functions have been found to be easily adaptable to the case of applied foundation displacements. An important advantage of these new relationships is their compatibility with the ground-shaking component of DBELA, using the same geometric and material properties as variables, the same limit state definitions, and the same probabilistic framework. As described in Section 8.5, differential foundation displacements will cause deformations in either the columns or the beams. The damage state is therefore controlled by the capacity of the beams and columns to resist this deformational demand, and can be determined analytically, through the use of simplified relationships for the displacement capacity, as will be described subsequently. Three important features of this approach are:

- Only bare reinforced concrete frame buildings are considered; the contribution of the infill walls to the capacity is likely to mean that this simplification produces somewhat conservative results.

- Only regular buildings are presented. There are a number of combinations of local ground conditions deformations and building configurations that will lead to different responses. However, the cases presented are considered representative of general behaviour and therefore appropriate to the requirements of earthquake loss estimations.

- Non-linear responses are estimated using equivalent linear assumptions, which carry significant uncertainties.

The damage states are defined in terms of limit states, where the first limit state (LS1) is defined by the yield capacity, representing the boundary between slight and moderate damage. The second and third limit states (LS2 and LS3), which define the boundaries between moderate and extensive and between extensive and complete damage states respectively, are both post-yield states, defined by prescribed limiting strains in the steel and concrete (see Crowley et al.,
2004a). Furthermore, P-Δ effects are presently discounted from the analytical solutions presented herein, assuming that the damage states are controlled by member rotations.

The equations presented in the following sub-sections were developed in a collaborative effort with researchers from the ROSE school, in Pavia, Italy. They are developed from the DBELA relationships for damage caused by ground shaking, presented by Pinho et al. (2002), Glaister & Pinho (2003) and Crowley et al. (2004a). The author’s contribution relates to the modification of the existing relationships to accommodate the requirements of a ground failure component.

8.7.2 Vertical differential foundation movement

8.7.2.1 Deformational demand to columns due to vertical ground movement

Considering a single frame, as presented in Figure 8-12, the critical column is the one to the left which undergoes the smaller displacement at ground level. Given that the beam rotates rigidly, the demand on this ground floor column is defined by the horizontal displacement, ΔD, at the top of the column, which can be obtained from simple geometry, as follows:

\[ \Delta_D = \Delta_{FY} \frac{h_s}{\sqrt{l_b^2 - \Delta_{FY}^2}} \approx \frac{\Delta_{FY} h_s}{l_b} \]  

(8-1)

where: Δ_{FY} \quad Maximum differential vertical deformation of the foundation

h_s \quad Storey height

l_b \quad Beam length.

8.7.2.2 Deformational capacity of critical column

The critical column in the frame, described above, is assumed to behave as a cantilever in simple bending. The behaviour of a cantilever when subjected to a lateral deformation is illustrated in Figure 8-17. With reference to Figure 8-17(a) and (b), at any cross section in the column, the curvature, \( \phi \), can be computed as the sum of the steel and concrete strains (\( \varepsilon_s \) and \( \varepsilon_c \), respectively) at the two extremes of the section, divided by the effective depth, \( d' \).

Figure 8-17(c) shows the column member, which has storey height \( h_s \). When the section at the base of the column reaches the yield curvature, \( \phi_i \), the first limit state (yield) is reached and the curvature distribution with height can be conservatively approximated as triangular, as shown in Figure 8-17(d). By integrating the curvature distribution along the length of the deformed
member, the tangent yield rotation, $\theta_\text{ty}$, is obtained, and the yield displacement at the top of the column can be computed by the moment-area method (e.g. Gere & Timoshenko, 1997).

\[ \phi = (\varepsilon_y + \varepsilon_{y\text{.c}})/d' \]

\[ \theta_\text{ty} = \phi_y \frac{h_y}{2} = 1.07 \varepsilon_y \frac{h_y}{h_c} \]  

\[ \phi_y = \frac{2.14 \varepsilon_y}{h_c} \]

\[ \phi_{LS} = \phi_y + \phi_p \]

\[ \Delta_p \]

\[ \Delta_L \]

**Figure 8-17:** Behaviour of a cantilever subjected to lateral deformations. (Not to scale, adapted from Paulay and Priestley, 1992)

(a) typical poorly-confined reinforced concrete column section
(b) strain profile and definition of curvature
(c) prismatic reinforced concrete cantilever
(d) curvature distribution with height when the section at base reaches yield curvature
(e) curvature distribution with height at post-yield limit states
(f) components of total lateral tip deflection.

(Figure courtesy R. Pinho & H. Crowley)

The yield curvature of the column is estimated using Equation 8-2, after Priestley (1998):

\[ \phi_y = \frac{2.14 \varepsilon_y}{h_c} \]  

where $\varepsilon_y$ represents the yield strain of the reinforcing steel and $h_c$ the depth of the column section.

The tangent yield rotation, $\theta_\text{ty}$, at the top of the column (see Figure 8-17(f)), is obtained through integration of the curvature distribution at yield (Equation 8-3). This is then increased, to account for the shear deformation and the joint deformation, using empirical factors proposed by Priestley (1998), as shown in Equation 8-4.

\[ \theta_\text{ty} = \phi_y \frac{h_y}{2} = 1.07 \varepsilon_y \frac{h_y}{h_c} \]
Finally, the yield displacement capacity, $\Delta_y$, at the top of the column, is obtained from Equation 8-5.

$$\Delta_y = \theta_y \frac{2}{3} h_s = 0.96 \varepsilon_y \frac{h_s^2}{h_c} \quad (8-5)$$

The post-yield curvature of the column is the sum of the yield curvature and the plastic curvature, $\phi_p$. The limit state curvature is a function of the prescribed limit state steel and concrete strains ($\varepsilon_{CLS}$ and $\varepsilon_{CL}$, respectively), and is approximated by the sum of these two strains at the extremes of the section divided by the depth of the column section. Therefore the plastic curvature can be estimated as follows:

$$\phi_p = \phi_{LS} - \phi_y = (\varepsilon_{CLS} + \varepsilon_{CL}) \frac{1}{h_c} - \frac{2.14 \varepsilon_y}{h_c} = (\varepsilon_{CLS} + \varepsilon_{CL} - 2.14 \varepsilon_y) \frac{1}{h_c} \quad (8-6)$$

Figure 8-17(e) shows (dashed line) the approximate curvature distribution assumed in order to simplify the integration of the actual curvature profile. In this way, the plastic curvature may be multiplied by a plastic hinge length, $l_p$, assumed to be half of the section depth (Paulay & Priestley, 1992) and therefore the plastic rotational capacity, $\Theta_p$, can be approximated by Equation 8-7. The plastic hinge length, $l_p$, in Figure 8-17(e) does not represent the total extent of plasticity, but may be considered to be a representative length used for computational purposes.

$$\Theta_p = \phi_p l_p = \phi_p \cdot 0.5 h_c = (\varepsilon_{CLS} + \varepsilon_{CL} - 2.14 \varepsilon_y) 0.5 \quad (8-7)$$

The plastic displacement capacity, $\Delta_p$ (Figure 8-17(f)) is then the product of the plastic rotation and the height of the column (Equation 8-8), and the total limit state displacement capacity is the sum of the yield displacement (Equation 8-5) and the plastic displacement, as shown in Equation 8-9.

$$\Delta_p = \Theta_p h_s = (\varepsilon_{CLS} + \varepsilon_{CL} - 2.14 \varepsilon_y) 0.5 h_s \quad (8-8)$$

$$\Delta_{LS} = \Delta_y + \Delta_p = 0.96 \varepsilon_y \frac{h_s^2}{h_c} + (\varepsilon_{CLS} + \varepsilon_{CL} - 2.14 \varepsilon_y) 0.5 h_s \quad (8-9)$$

Therefore the damage estimation comprises the following stages:
1. Define the demand, using Equation 8-1.

2. Define yield capacity, Limit State 1, using Equation 8-5.

3. Define the post-yield capacity for Limit States 2 and 3 using Equation 8-9. These states are a function of the specified limiting concrete and steel strains.

4. A building's ultimate damage state due to the ground deformation will be defined by which limit state it exceeds.

8.7.3 Vertical ground deformations at the centre of a multi-bayed frame

8.7.3.1 Deformational demand to beams due to vertical ground movement

For this case, shown in Figure 8-14, the vertical displacement demand on the beam, $\Delta_{py}$, is equal to the maximum differential foundation settlement, $\Delta_{py}$.

8.7.3.2 Deformational capacity of beams

The response of the beams to the deformational demand will be to deform in double bending, as shown in Figure 8-18. The behaviour may be likened to that of two cantilevers, each of length $l/2$, placed end to end, leading to a point of contraflexure at the centre. Plastic hinges are assumed to form at either end of the beam.

![Figure 8-18: Displaced shape of beam due to vertical ground deformations at the centre of a multi-bayed frame](image)

Again, the yield curvature is defined in terms of the geometric and material member properties using the relationships presented by Priestley (1998):

$$\phi_y = \frac{1.7 \varepsilon_y}{h_b} \quad (8-10)$$
The yield chord rotation, \( \theta_y \), defined as the angle between the tangent to the axis of the beam at the yielding end and the point of contraflexure, can be found from the curvature distribution shown in Figure 8-17(d), but noting that this represents the distribution for only half the length of the beam. The moment area method can be used to calculate the displacement at the centre of the beam, as carried out previously for the column, so the chord rotation is as given in Equation 8-11:

\[
\theta_y = \frac{1}{0.5 l_b} \left( \frac{\phi_y \cdot l_b}{2} \cdot \frac{2}{2} \cdot \frac{l_b}{2} \right) = \frac{\phi_y l_b}{6} = 0.283 \varepsilon_y \frac{l_b}{h_b} \quad (8-11)
\]

The contribution of the shear and joint deformations to the total deformation is allowed for, as previously, using empirical factors proposed by Priestley (1998), thus:

\[
\theta_y = 1.35 \theta_y = 0.382 \varepsilon_y \frac{l_b}{h_b} \quad (8-12)
\]

For small rotations, the chord rotation can be multiplied by the length of the beam to give the vertical displacement capacity of the beam, \( \Delta_y \):

\[
\Delta_y = \theta_y l_b = 0.382 \varepsilon_y \frac{l_b^2}{h_b} \quad (8-13)
\]

where \( h_b \) is the height of the beam section.

The plastic curvature distribution (Figure 8-17(e)) may also be used in the case of a beam in double bending, considering again that this distribution only represents half of the beam length. Therefore the same procedure explained in Section 8.7.2.2 is followed; the limit state plastic curvature is shown in Equation 8-14 and the plastic rotation is given by the plastic curvature multiplied by the plastic hinge length (taken as half the section depth) as shown in Equation 8-15.

\[
\phi_p = \phi_{LS} - \phi_y = (\varepsilon_{cLS} + \varepsilon_{sLS}) \frac{l}{h_b} - \frac{1.7 \varepsilon_y}{h_b} = (\varepsilon_{cLS} + \varepsilon_{sLS} - 1.7 \varepsilon_y) \frac{l}{h_b} \quad (8-14)
\]

\[
\theta_p = \phi_p l_p = \phi_p \cdot 0.5 h_b = (\varepsilon_c + \varepsilon_s - 1.7 \varepsilon_y) 0.5 \quad (8-15)
\]

Finally, the plastic displacement shown in Figure 8-18 is obtained from the product of the plastic rotation and the beam length (Equation 8-16) and added to the yield displacement to give the total limit state displacement capacity (Equation 8-17).
\[ \Delta_p = \theta_p l_b = (\varepsilon_{cLS} + \varepsilon_{sLS} - 1.7 \varepsilon_y)0.5l_b \]  
(8-16)

\[ \Delta_{LS} = \Delta_y + \Delta_p = 0.382\varepsilon_y \frac{l_b^2}{h_b} + (\varepsilon_{cLS} + \varepsilon_{sLS} - 1.7 \varepsilon_y)0.5l_b \]  
(8-17)

8.7.4 Horizontal differential foundation displacements

8.7.4.1 Deformational demand due to horizontal ground movement

The deformed shape of a frame building subjected to a differential horizontal ground deformation was shown in Figure 8-15b. This mode of deformation is similar to that shown in Figure 8-12. The demand on the left-hand, critical column, \( \Delta_D \), shown in Figure 8-15b, can be approximated as the maximum differential horizontal deformation of the foundation, \( \Delta_{FH} \), i.e.:

\[ \Delta_D \approx \Delta_{FH} \]  
(8-18)

8.7.4.2 Deformational capacity of the critical column

The critical column of a frame subjected to horizontal ground movement is assumed to behave through a combination of simple and double bending, shown in Figure 8-19. Therefore a combination of the results presented in Section 8.7.2.2 for simple bending, and Section 8.7.3.2 for double bending (adapted for a column section instead of a beam), is used to derive equations for the displacement capacity of the critical column.

![Figure 8-19: Displaced shape of column due to differential displacement at foundation level, schematically shown to be a combination of simple and double bending.](image)

A column under double bending, allowing for the shear and joint deformations can be defined by modifying Equation 8-4 as follows:
The chord rotation of a cantilever, given in Equation 8-20, is obtained by dividing Equation 8-5 by the storey height:

\[
\theta_y = 0.96\varepsilon_y, \quad \frac{h_s}{h_c} \quad (8-20)
\]

A simple representation of the combination of simple and double bending is obtained by taking the average of Equations 8-19 and 8-20, shown in Equation 8-21.

\[
\theta_y = \frac{0.96 + 0.382}{2} \varepsilon_y, \quad \frac{h_s^2}{h_c} = 0.67\varepsilon_y, \quad \frac{h_s}{h_c} \quad (8-21)
\]

Therefore, by following the same procedure as before of multiplying the chord rotation by the length of the element (for small rotations), the horizontal yield displacement capacity of the column is:

\[
A_y = 0.67\varepsilon_y, \quad \frac{h_s^2}{h_c} \quad (8-22)
\]

The plastic rotation is the same regardless of whether the column experiences simple or double bending. As mentioned in Section 8.7.3.2, double bending is analogous to two members in simple bending placed end-to-end. The plastic displacement of a cantilever of height \( h_s \) is therefore the same as the plastic displacement of two combined cantilevers each with height \( h_s/2 \), such as occurs in a column of height \( h_s \) in double bending. In the post-yield limit states, the plastic rotational capacity is therefore calculated from the plastic curvature multiplied by the plastic hinge length of the section, as presented in Equations 8-6 and 8-7. The plastic displacement is then given by Equation 8-8, and the total post-yield capacity is the sum of Equation 8-8 and 8-21, given in Equation 8-23:

\[
\Delta_{LS} = 0.67\varepsilon_y, \quad \frac{h_s^2}{h_c} + (\varepsilon_c + \varepsilon_s - 2.14\varepsilon_y)0.5h_s \quad (8-23)
\]

### 8.8 Application of the analytical procedures to earthquake loss models

Application of Equations 8-1 to 8-23 allows direct evaluation of the damage state of a group of buildings, defined by their geometric and material properties, resulting from liquefaction.
induced permanent differential displacements at foundation level. The damage states are determined as a function of the yield strain of the reinforcing steel and the post-yield strains in the steel and concrete. However, such a calculation is strictly deterministic, and ignores the significant uncertainties related to both the capacity and the demand. For this reason, a probabilistic framework is preferable, as described below.

8.8.1 Probabilistic framework for the damage estimation

For the analysis of building vulnerability to ground shaking, where both the period and the capacity of each building class are significant variables with separate and independent uncertainties, a joint probability density function (JPDF) is required to fully model the variability of the capacity. However, for building vulnerability to ground deformations, the building period is not a significant variable, so a single PDF can be used to describe the uncertainty in the displacement capacity. This PDF is a function of the material properties, geometric properties, and the yield strain of steel and post-yield strain capacities of steel and concrete. In addition, the uncertainty associated with the empirical coefficients used, i.e. the yield curvature coefficient (Equation 8-2) and the joint flexibility/shear coefficients in Equations 8-5 and 8-12, and the plastic hinge length, \( l_P \), may all be represented probabilistically. Each of these variables is represented by an appropriate probability distribution, with a mean and variance, and these are combined to obtain the overall PDF of the capacity using the first order reliability method (FORM) for each group of buildings with similar properties.

Reliability analysis takes defined statistical properties of each uncertain variable in a model, i.e. the mean and variance and distribution, and propagates them through a given analytical model, in this case the model to define the limit state displacement capacity. The results are obtained in terms of the mean and standard deviation of the capacity. FORM as the name suggests, is based upon the assumption that second order terms and higher can be ignored in the expansion of a series used to represent the propagation of errors from any number of uncertain variables through a model (e.g. Baecher & Christian, 2003).

When the demand is due to permanent ground deformation, \( GD_p \), the uncertainty includes all of the variability associated with the ground motion estimation, plus the additional uncertainties associated with the triggering of liquefaction, the variability in soil parameters and stratigraphy and the uncertainty in the assessment of \( GD_p \). For the probabilistic framework presented in this section, all of these uncertainties must be combined in order to obtain a single cumulative distribution function (CDF) representing the demand.
Thus, the PDF of the limit state capacity \( f_c \) and the CDF of the displacement demand \( F_D \) are combined using the reliability formulae to obtain the probability of exceeding a given limit state \( P_{f,LS} \), as shown in Equation 8-24.

\[
P_{f,LS} = \int [1 - F_D(x)] f_c(x) dx
\]  

(8-24)

The probability of a building falling into a given damage state (Table 4-2) is then obtained as follows:

\[
P_{\text{slight}} = 1 - P_{f,LS1}
\]  

(8-25)

\[
P_{\text{moderate}} = P_{f,LS1} - P_{f,LS2}
\]  

(8-26)

\[
P_{\text{extensive}} = P_{f,LS2} - P_{f,LS3}
\]  

(8-27)

\[
P_{\text{complete}} = P_{f,LS3}
\]  

(8-28)

These probabilities are then equated to a proportion of the building stock for the purposes of obtaining a regional damage estimation.

The calculations are undertaken using a modification of the FORTRAN-based DBELA program, originally developed by Helen Crowley and others at the ROSE School, Pavia.

### 8.8.2 Capacity parameters

A sensitivity study of the epistemic uncertainties related to the input data for the ground shaking calculations using the DBELA formulation has been presented by Crowley et al. (2004b), with contributions from this author. In terms of the capacity, these uncertainties are essentially the same for the ground failure calculations presented herein. The geometric, material and limit state properties were varied firstly in the mean values, with the standard deviation held constant, and secondly with the mean constant and varying the standard deviation. The sensitivity to the assumption of normal and lognormal distributions for these variables was also analysed. Subsequently the same exercise was carried out for the empirical coefficients for joint flexibility/shear, plastic hinge length etc.

As would be expected it was found that the epistemic uncertainty in the capacity parameters, particularly the geometric parameters (mainly in the mean values rather than their standard deviations or probability distributions) has a very significant effect on the estimated damage
distribution, even in comparison with the uncertainty in the demand. This suggests that there is a significant benefit to be obtained from investing more time and effort in order to characterise these variables correctly. Furthermore, this demonstrates one of the advantages of the DBELA methodology, which allows the user to specify each of these parameters and its probable distribution. Certainly for ground failure, there are no existing procedures that allow this.

### 8.8.3 Fragility curves

From the relationships presented in Section 8.7, it is straightforward to derive fragility curves of the differential foundation displacements, $\Delta_{FV}$ and $\Delta_{FH}$, versus the probability of exceedance of the three limit states. Examples of these curves are presented in Figure 8-20.

![Fragility curves](image)

**Figure 8-20:** Illustrative vulnerability curves for poor quality RC frame buildings characteristic of European building stock, subjected to (a) multi- and single-bay differential vertical, (b) differential horizontal foundation deformations and c) multi-bay differential vertical at the centre of the building.

From Figure 8-20(a) and Equations 8-25 to 8-28, for $\Delta_{FV} = 20$ cm, a single building would have a 35% probability of experiencing slight damage, a 6% probability of moderate damage, an 8% probability of extensive damage and a 51% probability of complete damage. These quite high damage levels are commensurate with the poor quality assumed for this building class.
The curves shown in Figure 8-20 are for a specific combination of geometry, material properties and limit states. The computation of further sets of curves for any given study would be a straightforward exercise. The variability in these curves is attributable to the capacity of the buildings only; the demand is modelled deterministically. The mean values of storey height and beam length are 3.5 m and 4.0 m respectively, with coefficients of variation of 35% and 25%. The column depth is assumed to be 0.25 m with a coefficient of variation of 15%. This extremely low value represents a worst case scenario, where all the columns are oriented in the same direction and failure thus occurs in the weak axis. The building class is assumed to be poor such that the structural members are inadequately confined. The assumed limit state strains and steel yield strengths are summarised in Table 8-2. These have been taken directly from studies of Turkish building stock presented by Crowley et al. (2004b).

**Table 8-2: Assumed capacity parameters and distribution for poor quality RC frame buildings, used to calculate the fragility curves shown in Figure 8-20**

<table>
<thead>
<tr>
<th>Capacity Parameter</th>
<th>Mean value</th>
<th>Coefficient of variation</th>
<th>Probabilistic distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steel yield strength</td>
<td>275MPa</td>
<td>10%</td>
<td>Normal</td>
</tr>
<tr>
<td>Limit state 3 concrete strain, ε₃₃</td>
<td>0.75%</td>
<td>50%</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Limit state 3 steel strain, ε₃₃</td>
<td>2.25%</td>
<td>50%</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Limit state 2 concrete strain, ε₂₃</td>
<td>0.45%</td>
<td>50%</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Limit state 2 steel strain, ε₂₂</td>
<td>1.25%</td>
<td>50%</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Storey height</td>
<td>3.5 m</td>
<td>35%</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Beam length</td>
<td>4.0 m</td>
<td>25%</td>
<td>Lognormal</td>
</tr>
<tr>
<td>Column depth</td>
<td>0.25 m</td>
<td>15%</td>
<td>Lognormal</td>
</tr>
</tbody>
</table>

The increased gradient of the curves in Figure 8-20(b) as opposed to Figure 8-20(a) shows that lateral displacements have a higher damaging potential for single single-bayed frame buildings. For the multi-bayed frame, the case where the maximum settlements occur in the centre of the building not at its edge is more damaging, indicated by the steepness of the vulnerability curves in Figure 8-20(c). This is consistent with the fact that the beams are all in double bending, a much more demanding mechanism than either simple bending or combined simple and double bending for the same demand.

The curves for the three limit states are very close together in Figure 8-20(a). The physical significance of this is that when the section has yielded (i.e. failed limit state 1), the inadequate confinement that is assumed for a poor building means that it will very rapidly also fail the second and third limit states. For a good building, there would be more separation between the
curves, representing the greater post-yield capacity due to the higher limit state strains of adequately confined members.

These fragility curves are useful for consideration of the probabilistic treatment of structural response to foundation movements, but it should be noted that they represent a simplified case in that the demand is treated deterministically, i.e. it is assumed to have a single, median value, with no variability.

![Figure 8-21: Comparison of vulnerability curves for LS3 (i.e. probability of complete damage) for different building quality and different treatment of the demand.](image)

The influence of modelling the demand as a lognormally distributed variable rather than a single value is illustrated in Figure 8-21. The variance in the ground deformation demand is assumed to have a constant natural logarithmic standard deviation of 0.8; slightly greater than the variability associated with the amplitude of strong-ground motion, which tends to be of the order of 0.7. At lower levels of demand, the probability of damage is higher when the variability is taken into account, whereas at higher levels including the variability in the demand has the effect of reducing the probability of damage. This is because once the capacity is exceeded in the deterministic case, there is no uncertainty that the building has failed. Conversely, when the variability is included, at higher levels of demand there remains a small probability that the demand is lower and therefore that the building has not failed.

In the assessment of damage caused by ground shaking (Crowley et al., 2004a) the poor and good building classes may be defined by their expected failure mechanism, where poorly designed buildings are more likely to suffer soft-storey (column-sway) failures, as well as reduced limiting steel and concrete strains due to lack of confinement. For the damage due to
ground deformation only, the distinction between beam-sway and column-sway is not relevant, therefore the difference is due only to confinement levels, and the subsequent limiting strains assigned to each limit state. In the examples shown in Figure 8-21, both building classes have the same geometry. Thus, considering the similar geometry and the fact they have the same failure mechanism, the difference between the two curves shown in Figure 8-21 is not significant; nevertheless, it is confirmed that the good quality buildings are less vulnerable. If alternative assumptions were to be made about the foundation system for a well-designed building, for example that the foundations would be either deep, or stiff shallow rafts, then this difference could be significantly greater.

8.9 Interaction of Ground Failure and Ground Shaking

The complex issue of combined ground-shaking and liquefaction-induced damage to a single building is briefly considered in this section. Discussion of field observations of ‘base-isolation’ effects due to liquefaction and of apparent combined damage was presented in Section 3.6.2, and the issue was reviewed in Section 7.10. In summary, the potential scenarios at a liquefiable site (assuming some damage occurs) are:

i. Ground shaking is the only cause of damage.

ii. In zones where liquefaction occurs, the final damage state is a result of liquefaction only (Equation 7-1).

iii. The final damage state is caused by initial ground shaking, followed by liquefaction (Equation 7-2).

This section proposes an analytical approach for estimating the final damage state for the third option listed above. However, it is recognised that it is far from straightforward to determine when this will occur rather than the first or second scenarios, and that such assessments may in many cases be beyond the scope of a loss estimation model. Nonetheless, the procedure is a useful illustration of the potential influence of liquefaction following ground shaking, and also demonstrates the adaptability of the DBELA methodology.

Damage caused by ground-shaking is assumed to occur in the first part of the earthquake, followed by liquefaction-induced ground deformation either towards the end or subsequent to the earthquake. This ignores the potential for low frequency ground shaking after the onset of liquefaction (e.g. see Figure 2-15), but is considered an appropriate simplification. The final damage state is a function of the damage state reached at the end of the strong shaking, and
hence how much capacity remains to resist the liquefaction demand. A simplified behaviour model to determine the displacement capacity at the end of ground shaking is illustrated in Figure 8-22, where:

1. If the structure has not exceeded its yield capacity, i.e. it has remained elastic, then the full capacity remains to resist the liquefaction demand.

2. If either the first or second limit states have been exceeded, then it is nominally assumed that the structure is, on average, half-way between the two limits, and the remaining capacity is calculated accordingly. The capacity is calculated assuming that the unloading path has the same gradient as the initial loading path.

3. If the third limit state is exceeded then the structure is already in the complete damage state, irrespective of any further demand placed upon it by the ground failure.

\[ \Delta_{LS2} = \text{the average available displacement capacity of a structural element that has exceeded LS1 at end of ground shaking before failing LS2} \]

**Figure 8-22**: Schematic illustration of force:displacement paths for a structural element exceeding each limit state, showing the average available capacity the structural element will have left to resist liquefaction-induced deformations

The backbone curve representing the cyclic behaviour in Figure 8-22 also assumes that the same structural element will be loaded by both phases of the demand. This is likely to be the case for poor buildings exhibiting column sway behaviour under ground shaking, but possibly not for good buildings where failure may occur in the beams.

Based upon this model of 'residual' capacity at the end of cyclic loading due to ground shaking, the schematic diagram shown in Figure 8-23 can be followed to estimate the final damage state. A similar approach could be used to assess expected damage due to strong aftershocks. In order to implement this procedure, a decision must be made regarding the degree to which the
occurrence of liquefaction will modify the demand compared to the case where no liquefaction occurs. In this example, it has been hypothesised that the peak demand will still occur, but the number of cycles of strong motion would be fewer. Further, detailed research is required to establish a sound basis for modifying, or not, the ground-shaking demand at a site where it is known that liquefaction will occur. This very interesting concept is, unfortunately, beyond the scope of this thesis.

![Building Classification and Damage States](image)

Figure 8-23: Cumulative structural damage due to ground shaking followed by liquefaction.

8.10 Case study

A demonstration of the analytical approach described in Sections 0 to 8.9 is presented in this section. This requires an estimation of the corresponding levels of differential ground movements and ground shaking in the same location, preferably measured rather than predicted in order to reduce the uncertainty in the demand. The data from central Adapazari presented in Chapter 5 are used for this purpose, since both the shaking and the permanent ground deformation from the 1999 Kocaeli earthquake are approximately known. Therefore the demand can be considered without needing to include the full variability associated with the uncertainty in its estimation. The only uncertainty is the spatial variability plus the natural heterogeneity of in situ ground conditions. The observed damage in central Adapazari is not entirely analogous to the solutions presented in Section 0, due to the predominance of very stiff raft foundations.

The best estimate of the response spectra in downtown Adapazari was presented in Chapter 5, for magnitude $M_w=7.4$, closest horizontal distance to the fault of 7 km and a site classification between NEHRP D and E, with the factors obtained by interpolation. The variability across the central region is assigned a log-normal distribution (as it would be if estimated on the basis of
ground-motion prediction relationships only), with the median value as described above, and the lower bound (assumed to be the 5th percentile) represented by a distance of 8km and a site class D, and the upper bound (95th percentile) represented by a distance of 6km and a site class E. In this way the logarithmic standard deviation of the demand was estimated.

Almost no lateral spreading was observed in Adapazari, therefore only settlement induced damage is considered in this example. The ground deformation demand has been estimated from the range of settlements reported for individual buildings by Yoshida et al. (2001), assuming that these are directly analogous to the differential settlements in the ground beneath the buildings.

Differential settlements are not reported for collapsed buildings, for buildings with no reference point from which to measure the settlement, for the many buildings that did not settle, or for those that settled uniformly. Where only an angle of tilt has been reported, it is not possible to calculate the associated settlements without knowing the dimensions of the building. Although sparse, the data are sufficient to illustrate corresponding ground shaking and differential settlements. The median and standard deviation of the differential settlements have been calculated from the range of reported values, assuming a log-normal distribution. The median differential settlement was 8cm, with a lognormal standard deviation of 0.6. These values are significantly lower than the maximum settlements reported in Adapazari, which exceeded 1.0m in areas, because they are differential values across the footprint of a single building. For a 4m wide building on rigid foundations, 8cm corresponds to a rotation of 1/50, or approximately 1° (between GFl and GF2 in the Bray & Stewart ground failure index, Table 5-4), and on this basis appears a reasonable estimate of the median value.

In order to complete the estimation of liquefaction-induced damage, two further factors are required, described below:

i. The proportion of buildings which were not affected by liquefaction. Even where there is a significant likelihood of liquefaction occurring, it must be recognised that this will not occur throughout the susceptible area, due to the variability in soil properties. A proportion of 60% is assumed since in central Adapazari at least 60% of the buildings surveyed by Bray & Stewart (2000) did not suffer any ground failure. This value ignores those buildings which collapsed and therefore made it impossible to determine the occurrence of ground failure, and could therefore be higher than the true value.

ii. The proportion of buildings where liquefaction did occur where the differential deformations was zero (i.e. uniform settlements). The presence of the stiff foundations
in Adapazari would certainly have influenced this by increasing the likelihood of uniform settlements, and therefore from the available data it is not easy to determine what such a factor would be in the case of flexible foundations. A nominal factor of 10% has been assigned for illustrative purposes.

It has further been assumed that that the foundation deformations were not influenced by the type of foundation or the geometry of the building, and that building tilt was directly caused by differential settlement of the foundations rather than by bearing capacity failure.

The damage distribution has then been predicted for the following cases:

i. Ground shaking damage to RC frame buildings only, using the methodology presented by Crowley et al. (2004a).

ii. Liquefaction damage to RC frame buildings with flexible foundations, due to differential settlements as explained in Section 8.7.2.

iii. Combined ground shaking and liquefaction, with ‘base-isolation’ due to liquefaction, i.e. the two hazards are mutually exclusive, and 60% of the buildings are damaged by ground shaking only, the other 40% by liquefaction only.

iv. Combined ground shaking and liquefaction with no ‘base-isolation’ i.e. over the 40% of the area where liquefaction occurs, buildings are damaged by ground shaking then liquefaction, as suggested in Figure 8-23.

Since the demand is predetermined for the sake of this illustration, it is possible to inspect the semi-hypothetical distributions of damage due to both ground shaking (using the method described by Crowley et al., 2004a) and liquefaction in the same locality.

Figure 8-24 shows that the liquefaction alone is less damaging to the RC frame buildings than ground shaking alone, therefore if the base isolation theory was supported, then liquefaction would have an overall beneficial effect in this case. Combining the two hazards in the manner suggested in Figure 8-23 would increase the overall level of damage. Both of these scenarios are compared to ground shaking alone in Figure 8-25. Figure 8-25 assumes, in the absence of better guidance, that the strength of ground shaking before the occurrence of liquefaction is not reduced. It should be remembered that only damage caused by differential settlement to RC frame buildings with flexible foundations is considered in the liquefaction scenario, ignoring many of the complexities associated with different foundation types and damage mechanisms.
For the 10% of structures on liquefied soil where the settlement is uniform rather than differential, it is assumed that no further damage will occur after the end of ground shaking.

The results presented in Figure 8-25 are not intended for comparison with the actual observations in Adapazari, since the building classes and foundation types are not compatible, they are intended as an illustration of the approach developed in this chapter.

![Figure 8-24: Comparison of damage distributions due to ground shaking only or liquefaction-induced differential settlements only](image)

Figure 8-24: Comparison of damage distributions due to ground shaking only or liquefaction-induced differential settlements only

![Figure 8-25: Comparison of damage scenarios (i), (iii) and (iv) listed above](image)

Figure 8-25: Comparison of damage scenarios (i), (iii) and (iv) listed above

Figure 8-25 suggests that, within the limitations of this case study, the significant additional work required to predict the combined damage due to liquefaction following on from ground shaking may not be merited, given the very small difference in damage levels compared to ground shaking only.

The added complexity of the rigid body rotation of the upper storeys and the damage states that these may relate to has been disregarded in this illustration. However, it is worth noting that this could be easily incorporated into the analysis through an additional check of the overall angle of tilt of the building in relation to pre-defined rotation limits such as those in Table 8-1.
8.11 Discussion

This chapter has presented a preliminary framework for estimating the damage caused to a particular building class as a result of ground deformations resulting from earthquake-induced liquefaction. The formulae presented in Section 8.7.1 are derived from sound mechanical principles, using simple geometric and material variables. The resulting damage states are directly obtained from the calculations, and these damage states are clearly defined and do not rely on judgement or empirically based criteria. The formulae also lend themselves to the construction of fragility curves relating damage states to increasing demand.

Since only one case has been developed, the framework does not yet present a complete solution for input to a loss model. For completeness, all building types must be considered, and the formulations extended to represent damage to non-structural elements, such as partition walls and external cladding, which is related to the deflections of the connecting beams and columns (e.g. Crowley et al., 2004a). However, as only one of the multiple components of a loss estimation, this framework represents a positive step towards developing a practical approach for estimating damage due to liquefaction.

![Figure 8-26: Comparison of rigid body rotational limits with vulnerability curves for differential settlements; curves are derived for poor quality RC frame buildings, with 4m beam length, with flexible foundations. Vertical lines are for 4m wide buildings with stiff foundations.](image-url)

Separate solutions have been proposed for buildings on stiff foundations, which will respond as rigid bodies to deformations at foundation level, and those on flexible foundations, which will undergo structural deformations. As noted, there is no obvious analytical solution to the former case, and so the suggested boundaries for damage states, shown in Table 8-1 are based upon empirical criteria. Figure 8-26 compares the suggested limits from Table 8-1 for rotations with the vulnerability curves for differential settlements. The former are deterministic, in that they
ignore the variability in the beam length related to grouping many buildings into one classification. The latter include variability in the geometric and material properties, but not in the demand. An intersection of the vertical line with the vulnerability curve for that limit state at $P_f = 0.5$ would suggest nominal agreement between the two. For rigid body rotations there is a slightly lower tolerance to small movements, being controlled by aesthetic issues rather than the qualitative parameter of yield stress, but higher tolerances to larger movements, being less sensitive to increasing movements than in the case of structural deformations. Further development of the framework presented in this chapter includes the urgent need for more field data for calibration. As noted by Karl Terzaghi (1936) "No honest business man and no self-respecting scientist can be expected to put forth a new scheme or a new theory as a ‘working proposition’ unless it is sustained by at least fairly adequate evidence”. The sensitivity of the results to the uncertain variables related to both the building capacity and the permanent ground deformation demand should also be explored.

Methods for the identification of liquefaction susceptibility, and the potential for the initiation of liquefaction under a given level of shaking continue to be developed and refined, but without the capability to determine what this means for the affected infrastructure, the usefulness of such methods is inevitably limited. The framework presented in this chapter seeks to address this shortcoming, through the development of a set of relationships for systematic treatment of the capacity of buildings subjected to ground deformations in a probabilistic manner.

The variability and uncertainty relating to the demand remains a major issue for this component. In the absence of appropriate probabilistic approaches for the estimation of both average and differential horizontal and vertical ground movements, the author feels that the solutions presented herein are a step ahead of the ground deformation component. Nonetheless there is extensive ongoing research in this field, such as that described by Seed et al. (2003), and it is to be hoped that the various components can soon be re-aligned and a complete methodology can be presented. Kramer & Stewart (2004) identify the “prediction of the permanent deformations of liquefied soil masses and structures supported on or within them” as one of the future challenges of geotechnical earthquake engineering.

Finally, the role of engineering judgement throughout this framework warrants a brief discussion. There is a tendency in earthquake loss estimations to seek to standardise procedures such that they can be applied to different regions by different users in order to obtain comparable results. This feature is particularly desirable for insurance companies, and is the basis of the ground failure component of the HAZUS methodology, which has been shown to include many simplifications, including ‘hidden’ simplifications based on the developers’
judgement. In the field of geotechnical engineering, the role of judgement is particularly significant, and, when extended to the field of liquefaction engineering, a certain amount is essential to deal with uncertainties ranging from the choice of the most appropriate methodology to the selection of the input parameters. No framework could be developed that is universally applicable to all regions and all ground conditions, thus it is imperative that decisions are made on the basis of individual studies. The use of probabilistic methods therefore has a doubly important role to play, since as well as representing spatial variability they are required to describe the uncertainties related to these judgements.
9 CONCLUSIONS

The research presented in this thesis was instigated by the realisation, during the development of a loss model for Turkey for reinsurance purposes, that there was no appropriate means of incorporating ground failure into the model. 'Appropriate' in this context means that methodologies to be incorporated into loss estimations should firstly provide a pragmatic solution to the issues of spatial uncertainty and variability in ground conditions and exposed building stock, and secondly produce a realistic and meaningful estimation of the expected risks. Both the input data requirements and the degree of uncertainty associated with the input data should be commensurate with other components of the model.

The relationship between the cost and time required to incorporate ground failure in an analysis and the benefits this will bring in terms of improving the accuracy and reducing the uncertainty of the results is an important issue for those involved in risk management.

The three principal objectives of this thesis, which has focussed on liquefaction-induced ground failure, were therefore:

i. To consider ground failure-induced damage in the context of regional losses in recent earthquakes in order to provide new guidance, supported by real data, for the incorporation of ground failure into future regional risk or loss assessments.

ii. To critically review available methods for predicting the likelihood and extent of liquefaction, and the consequent damage, and to compare these methods to field data. The purpose of this is to demonstrate the assumptions and uncertainties associated with these methodologies, and thus improve their transparency and assist in appropriate decision making at the start of projects.

iii. To develop a framework for an improved methodology, using the lessons learnt from the first two objectives. This framework identifies the significant issues in each step and presents further guidance on how each step should be carried out.

9.1 Liquefaction in loss estimations: is it worth it?

It is important to contextualise the subject of this thesis by clarifying that the ground failure component of a loss estimation is, in the majority of cases, not as significant to the final results as the ground shaking component. In 50 recent earthquakes investigated in Chapter 3, the
impact of liquefaction was greatest on lifelines such as roads, bridges and buried pipelines, and port structures. With respect to building damage, as a rule, the larger the study area, the more likely it is that ground shaking-induced damage to buildings will dominate the overall losses, and that any individual features of a particular zone will lose their significance (e.g. Figure 7-1). In such cases, appropriate decision-making criteria should determine whether secondary hazards such as liquefaction, landslides and fault rupture should be incorporated. This decision is non-trivial, since it has been shown that the use of over-simplified methodologies could unrealistically increase the contribution of ground failure to the damage.

The fact that liquefaction is a secondary hazard in the context of earthquake-induced losses by no means diminishes its importance. There is a need for comprehensive methodologies capable of estimating damage caused by all expected hazards to all elements of a region’s infrastructure. In other words, it is desirable to develop methods with the scope and completeness achieved in HAZUS, but without the shortcomings that have been identified in that approach. Furthermore, for detailed studies of sub-regions within an affected region, it is increasingly likely that local features such as liquefiable soils could influence or even dominate the results, as was shown in Chapter 5 to be the case for Adapazari in the 1999 Kocaeli earthquake.

Therefore, whilst it is acknowledged that in any given project there may be particular components that could be neglected, such decisions must be made on the basis of criteria relating to the features of the study region, its size, geology and topography, and to the ultimate objectives of the study. Currently, it seems that the selection of which components to include may be made on a somewhat arbitrary basis, and often relates to perceived difficulties in incorporating certain elements.

9.2 Lessons from previous earthquakes

Qualitative mapping of ground failure occurrence and its effect on infrastructure in post-earthquake investigations is invaluable for the development and calibration of predictive methodologies. Until recently, such data have rarely been comprehensively gathered in earthquake reconnaissance. Measurements and observations of ground failure occurrence in earthquakes in the past tended to be made for individual buildings and sites, making regional and inter-earthquake comparisons difficult. The detailed damage survey data collected by Bray and Stewart (2000) for central Adapazari used in Chapter 5 is invaluable in this respect. However, the lessons that can be learnt from this database, with respect to current damage estimation methodologies, were shown to be constrained by the following features:
• The uncertainties in the estimation of damage caused by ground shaking prevented conclusive findings related to the additional or alternative damage caused by ground failure. The latter would only be possible were it feasible to estimate the former with a high degree of certainty and compare these predictions with the observed damage.

• The damaged building stock in Adapazari was dominated by a single category, the beskat. The foundations of the affected buildings were also (surprisingly) uniform, and liquefaction typically manifested itself as settlement only, with very few cases of lateral deformation. By representing only a limited range of conditions, the usefulness of the database is inevitably restricted.

In the future, it is strongly recommended that reconnaissance in regions of ground-failure includes a return visit for the purposes of recording the repair methods of the surveyed buildings. The opportunity to gather an additional data set to that from Adapazari following a future earthquake with significant ground-failure would enable significant advances to be made. Improved analytical solutions are required in parallel to the need for field data, since even an improved database will be small and sporadic in comparison to those used to evaluate other insurance premiums.

9.3 Are current loss estimation methods getting it right?

*The methods HAZUS uses to evaluate settlement and lateral spread certainly are very crude. I hope you come up with procedures that might improve HAZUS's methodology. *" (R.V. Whitman, pers. comm., 2003).

In terms of recognising the need to have a ‘ground-failure component’ and including all the necessary steps to analyse damage due to ground failure, HAZUS (FEMA, 2003) has made a significant step in the right direction. However, this thesis has shown that HAZUS, or rather the HAZUS ‘default-methodology’, is not getting it right. This is true on a number of levels, from the details of the calculations and assumptions to, more importantly, the fundamental principles of the approach. To estimate building damage due to a geotechnical phenomenon, with the only input data being a susceptibility rating between none and very high, is intuitively flawed. The advantages of this simplification, with respect to attempting to reduce data input to manageable levels and keeping calculations straightforward are not to be disregarded; however, they lose their relevance in the light of the overall uncertainty.

If the expected earthquake losses due to liquefaction had to be estimated in the absence of any geotechnical information, then the author believes that the steps presented in HAZUS are
probably as good as any, and the assumptions made, although tenuous and heavily reliant upon the judgement of the developers, are necessary in order to attain this objective. The point is however, that expected earthquake losses due to liquefaction should not be estimated in the absence of any geotechnical information, at least not in a quantitative manner, and not from the limited information currently available to guide us in this field. To do so represents a false simplification, because of the complex, numerous and, most importantly, unquantified uncertainties associated with the subsequent results.

9.4 An improved framework for ground failure-induced losses

Figure 9-1 shows the principal stages in the evaluation of building damage in a region with liquefaction potential. Each of these stages has been discussed in some detail in the preceding chapters.

Appropriate consideration of the uncertainties associated with the ground failure component is probably the most significant recommendation of this research. Relative to the uncertainties associated with ground-shaking demand and building response, those associated with the occurrence and consequences of liquefaction are, if not larger, certainly more complex, and a number of additional uncertain variables are necessary to define them.

The uncertainties come from the variability of stratigraphy and strength of naturally occurring soils, which may be treated as aleatory, and from a number of epistemic sources relating to the choice of appropriate methodologies and input parameters for the evaluation of liquefaction probability, liquefaction-induced permanent ground deformation, and building response. Finally there are the uncertainties related to the necessary simplifications required in terms of grouping similar soil conditions and grouping buildings that are expected to respond in similar ways to liquefaction. In reality, each soil profile and building will respond differently, and understanding and therefore quantifying the effects of this grouping is one of the most significant requirements of regional damage estimations.

Complete definition of each stage Figure 9-1 becomes complex. A full flow chart, with every decision and option included, risks becoming incomprehensible. Figure 9-1 therefore presents a schematic overview, to be referred to in conjunction with other flow charts presented throughout this thesis, which provide additional guidance on the relevant decisions.
Chapter 9: Conclusions

START - decision to include liquefaction in model

Calculate $P(\text{liquefaction})$ versus depth in each typical soil profile for liquefiable zones.

Define scenarios combining $P(L)$ and $T_l$ (thickness of liquefiable layer).

Define probable modes of permanent ground deformation ($PGD$) where $PGD = f(T_l)$ as well as other variables.

Building classification (superstructure, foundation type)

Damage distribution $P(DS)$ (slight, moderate, extensive or complete) due to liquefaction only

Combine with ground shaking damage

Interaction $P(DS) = P(DS|\text{shaking then liquefaction})$

Figure 9-1: Estimation of damage in a zone of liquefaction susceptibility. Main components are on the left hand side. A simplified illustration is shown for $P(T_l) = 0.5$, differential vertical movements only and RC frame buildings with flexible foundations.
9.5 A unified damage scale

An important issue has been raised related to the damage scales currently used in damage surveys and prediction methodologies (e.g. Coburn & Spence, 2002; Grünthal, 1998; HAZUS, FEMA, 2003). These scales all classify damage in terms of cracks, deformations, or collapse of beams, columns and walls, i.e. of structural components. A typical mode of liquefaction-induced damage, observed where buildings are on stiff shallow foundations, manifests as ‘rigid-body’ settlement, tilt or lateral displacement of the building foundations, often with little or no associated cracking or other structural damage. In Adapazari, for example, many of the buildings that underwent severe foundation failure were, by the strict application of the damage scales, classified as ‘undamaged’. This is further complicated by the fact that surveyors, recognising this to be unrealistic, may make judgements regarding the damage state of tilted and settled buildings in the field, but in the absence of an accepted unified scale, these judgements could vary considerably.

The proposed unified damage scale, presented in Chapter 8, aligns structural damage and rigid-body response to ground failure, such as settlement or tilt. With this type of scale, future field measurements can be made that relate the degree of ground failure to the probable repair cost of the structure, and loss estimations will be able to directly relate the expected behaviour of the soil to the vulnerability of the buildings expected to respond as rigid-bodies. There is a need for validation of this scale using cost of repair data from field reconnaissance.

9.6 Building damage due to ground failure

The determination of the impact that liquefaction-induced deformations will have on existing structures, has been found to be a ‘grey’ area, in research and practice. Haldar and Luetich (1985) noted: “This is an extremely difficult problem. Damage is a highly controversial subject due to its qualitative nature.” It has been shown that within a regional framework, the difficulties are even greater than for site-specific analysis. There are many reasons for this, not least being that the preferred design approach tends to be to mitigate liquefaction potential by ground improvement or site relocation.

The heavy reliance on empirical data that is encountered in so many aspects of geotechnical earthquake engineering, due to the complexity and variability of soil response, and the difficulties in obtaining test data of sufficient reliability, creates the continuous problem that all our models are adequately confirmed for the last earthquake but not necessarily for the next. The analytical solutions for building vulnerability to ground deformations, presented in Chapter
9.7 Further research

The solutions and recommendations in this thesis have been presented from the perspective of seeking to improve the bigger picture. Having done this, there are a number of detailed aspects to be resolved in order to present a complete solution.

9.7.1 Numerical modelling of soil-structure interaction in liquefiable zones

As has been noted previously, in many cases the preferred design solution for zones of expected ground failure is avoidance, through relocation or mitigation. However, there is also the option to carry out detailed analysis using finite element modelling to determine the response of the soil to cyclic loading and pore water pressure increases, and the ensuing interaction with foundations and structures in either two- or three-dimensions. Notwithstanding the shortage of data for calibration of these complex models, they are inherently unsuitable to the requirements of a regional loss estimation as the demands of time and data acquisition are prohibitive. However, there is potential to use these models in the future for parametric studies aimed towards the calibration or development of vulnerability relationships, and to provide additional data for the derivation of permanent ground deformation predictive relationships (see below).

9.7.2 Improved procedures for estimating permanent ground deformation

The input to the analytical procedure presented in Chapter 8 is the expected magnitude of the differential settlement and lateral deformation, defined in terms of the mean, variance and statistical distribution. The expected ‘rigid body’ damage states, also defined in Chapter 8 are dependent upon the absolute amplitude of permanent ground deformation for settlement and lateral movements, and on the differential settlement for tilt. This definition of the demand input in a regional context is presently the area of the greatest uncertainty. A gap exists between the definition of the probability of liquefaction, and the definition of building vulnerability to ground deformations – given that these deformations will occur. A flow chart for the analysis
of this component is suggested in Figure 7-7, but a significant amount of additional work is required to develop this into a procedure suitable for incorporation into loss models.

9.7.3 A simplified approach for estimating losses in liquefiable zones

The framework presented in Figure 9-1 will significantly increase both the data requirements and the computational complexity of any loss model through the inclusion of liquefaction. The research presented in this thesis suggests that a reliable yet simple procedure, compatible with the requirements of a loss estimation model, is still some way from being a realistic possibility, but such a procedure should be the ultimate goal of this research.

Where extensive in situ geotechnical information is not available, as may often be the case, an incremental increase in predicted damage levels in potentially liquefiable zones, such as that proposed by Bommer et al. (2002) for Turkey, is a rational approach. However, there are presently insufficient data to derive an appropriate 'liquefaction damage factor', which would be dependent on the characteristics of the building stock and the expected extent of the liquefaction hazard. To do so requires additional field data from liquefied sites, reduced uncertainty in the prediction of ground-shaking induced damage, and analytical solutions for all building classes subjected to permanent ground deformations. The procedures presented in Chapter 8 within the DBELA framework are an important first step in this respect.

9.7.4 Other further work

As noted in Section 9.1, research in the field of earthquake damage estimations must seek to develop comprehensive strategies for the assessment of damage in all conditions. The solutions presented in this thesis are for particular conditions only, and the following issues are just some of those that require similar treatment:

- The structural system currently considered is bare RC frame buildings. The addition of the contribution of infill walls to the response to both ground shaking and ground deformation is an important requirement within the DBELA framework.
- Other structural systems and foundation systems, beginning with unreinforced masonry structures, which tend to have shallow, relatively flexible foundations, and RC frame buildings with piled foundations or basements.
• Damage to other items of infrastructure, notably pipelines, roads and bridges, each of which have been noted as being susceptible to liquefaction damage and capable of generating significant indirect losses.

• Non-structural damage, which can cause greater economic losses than structural damage, particularly in moderate magnitude earthquakes. Some components of non-structural damage, for example ceilings, walls and facades, are also related to the induced deformation, therefore a similar approach to that presented in Chapter 8 could be used.

9.8 Closing comments

The need to evaluate expected losses due to natural disasters will not diminish. Stojanovski et al. (2003) report that industry spending on risk modelling is expected to increase from $111 million in 1995 to $432 million in 2005, and the hurricane losses in 2004 once again warned insurers of the essential need to be prepared for these events. Correspondingly, developments in the field of geotechnical engineering for the rigorous treatment of the many uncertainties are producing improved probabilistic methods in this field. There is thus an increasing compatibility between the engineering approaches and the risk management requirements of the issues discussed in this thesis.

Ultimately, the evaluation of the impacts of earthquake disasters on society and economy is the important issue. This research is only one element within a very large and complex field that involves significantly more disciplines than earthquake engineering. The end product should never be forgotten, or there is a risk that the solutions obtained will not be suitable to their purposes.

Building owners need to understand whether people in their buildings will be injured or killed, how long it will take to repair, and how much it will cost. Insurers need to know how many and what type of claims they will have to settle. Governments and emergency services need to know the scale and distribution of the damage that they will have to deal with and the costs. To each of these parties, the detail of how the damage is caused or evaluated may be of little concern. However, their concern is that the results they are presented with are realistic, and that uncertainties have been treated rigorously, not ignored. There is therefore an onus on engineers to provide the necessary parts as input to the whole. Although the estimation of losses in zones of ground failure is a small element on its own, the bigger picture can only be ultimately achieved by focussing initially on each of these smaller elements.
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APPENDIX A

Illustration of catastrophe insurance
(after Crosen & Kunreuther, 1999)
A.1 Catastrophe Insurance

The following extracts from a paper by Crosen & Kunreuther (1999) explain the key points of catastrophe insurance, including the key stakeholders, and why the ‘law of large numbers’ may not apply in the case of catastrophes.

The law of large numbers describes the tendency of the average of a sample set to revert to the mean of the entire population as the size of the sample is increased.

1. Key Stakeholders and Their Concerns

Although each country faces a different set of hazards and has a different set of institutional arrangements, the common thread is how to offer greater protection to potential victims. There are a set of key stakeholders, each of whom as their own set of objectives and concerns, that need to be considered when designing a set of financial arrangements to provide them with protection after a major disaster occurs.

Homeowners and Businesses at Risk are concerned with having insurance to cover their losses because they are risk-averse. In particular, they are willing to pay a relatively small price today to protect themselves against a large decrease in wealth tomorrow from a natural disaster. The ultimate risk they face is insolvency or bankruptcy (or a dramatic forced reduction in consumption) and they are anxious to avoid this state of the world.

Insurers offer protection against these risks by taking advantage of the law of large numbers. In other words, if they have a large enough (and sufficiently diversified) portfolio, they should be able to collect sufficient premiums to cover their losses even if worse-than-average periods occur. [...] An unusually severe catastrophe can make even a well-capitalized insurer insolvent, even if the insurer is, on average, profitable. A natural disaster, such as an earthquake, raises problems for them because of the high correlation among the losses in their portfolio. This dependence among risks may require them to raise premiums and/or reduce the extent of their coverage in hazard-prone areas to keep their chances of insolvency to an acceptable level.

Reinsurers provide protection to private insurers in the same way that insurers cover the policy holder or property owner – that is, they provide coverage against unforeseen or extraordinary losses. In a reinsurance contract, one insurance company (the reinsurer, or assuming insurer) charges a premium to indemnify another insurance company (the ceding insurer) against all or part of the loss it may sustain under its policy or policies of insurance. For all but the largest insurance companies, reinsurance is almost a prerequisite for offering insurance.

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That is, even if their revenues and investment gains exceed their administrative costs plus expected claims (including the costs of fully funding catastrophic losses if these losses were to occur in their expected proportions over a long period of time).
against hazards where there is the potential for catastrophic damage. Reinsurers have similar concerns to those of the insurers and hence will limit their exposure in catastrophe prone areas. They will follow the same options listed above for insurers to keep the chances of insolvency to an acceptable level.

**Investors in catastrophe bonds** will want to obtain a large enough return on their investment, in the form of higher than normal interest rates when no disaster occurs, to justify the risks of losing their principal and/or receiving a lower interest rate after a disaster.

**Government** is the reinsurer of last resort and has the ability to tax different stakeholders to raise money for providing financial payments when there is a catastrophic loss. A sovereign government which faces a hard debt constraint, however, may find borrowing additional funds difficult or impossible in a catastrophe were to strike.

II. An illustrative example

To better understand how these different stakeholders interact we construct a simple example consisting of two property owners, each of whom is subject to the probability \( p \) of a loss \( L \) from a natural disaster, such as a flood. There are thus four states of the world depending on whether or not each individual suffers a loss:

\[
(0,0) ; (0,L) ; (L,0) ; \text{ and } (L,L). \]

Here we present the qualitative results; they hold more generally for the realistic case where there are many properties at risk.

**Independent and Correlated Risks**

Even in the simple case where the risks are independent, there is one state of the world where both individuals suffer a loss: state \((L,L)\), which occurs with probability \( p^2 \). This state results in a net loss for the insurer unless it charges a premium much greater than the actuarial rate \([...]\). If the two homeowner risks were perfectly correlated (as would be the case if both properties were equally affected by a hurricane or an earthquake), then there are only two states of the world: \((0,0)\) and \((L,L)\) and there is a much higher probability \( p \) rather than \( p^2 \) that the insurer will suffer a net loss \([...]\). In reality, the damages to property from catastrophic events are partially correlated, so that the distribution of losses, insurer profits, and the probability of negative earnings fall somewhere between the two extremes.

Two generalizations follow from the above analysis. For independent risks, the probability of a large net loss (as a percentage of total premiums collected) in any given scenario falls as the number of insured increases, due to the law of large numbers. As the risks become more correlated, the chances of a large negative net profit in any given scenario increases for any given insured portfolio.

If the insurer cannot rely on other sources of funds (e.g. reinsurance, cat bonds, or government bailout) for protection against the catastrophic loss, then it has two ways of protecting itself: (1) it can have surplus capital available to pay for these losses should they occur, or (2) it can raise premiums to the point where insolvency is impossible even in the worst case.
Insolvency risk occurs when an insurer with a limited capital base covers a book of business that includes the possibility of losses exceeding the insurer’s ability to pay. The insurer is most likely to declare insolvency in an \((L,L)\) state and thus pay off only a portion of its claims, precisely when the property owners are most in need of capital.

More specifically, the risk-averse homeowners will be denied the beneficial option of offloading their risks and stabilizing their own wealth levels. If a catastrophe will create insurer insolvency, and policyholders will not be fully paid, the expected value of the insurance policy decreases and hence the policyholders’ willingness to pay for coverage. If the insurer charges lower premiums to generate demand then its chance of insolvency increases even further, thus reducing insurance demand even further. This downward spiralling of premiums and upward spiralling of insolvency risk may eventually produce a situation where the insurer would prefer not to offer this type of coverage at all because it cannot cover its marketing and administrative costs. The market will thus fail to clear, as an indirect consequence of the insolvency risk, leaving consumers uninsured against moderate-level risks as well as catastrophes.

(Crosens & Kunreuther, 1999, pages 2 – 5)
APPENDIX B

Selected extracts from The European Macroseismic Intensity Scale, EMS 98 (Grünthal, 1998)
## EMS Intensity and Description

<table>
<thead>
<tr>
<th>EMS Intensity</th>
<th>Definition</th>
<th>Description of Typical Observed Effects (Abstracted)</th>
</tr>
</thead>
<tbody>
<tr>
<td>I</td>
<td>Not felt</td>
<td>Not felt.</td>
</tr>
<tr>
<td>II</td>
<td>Searcely felt</td>
<td>Felt only by very few individual people at rest in houses.</td>
</tr>
<tr>
<td>III</td>
<td>Weak</td>
<td>Felt indoors by a few people. People at rest feel a swaying or light trembling.</td>
</tr>
<tr>
<td>IV</td>
<td>Largely observed</td>
<td>Felt indoors by many people, outdoors by very few. A few people are awakened. Windows, doors and dishes rattle.</td>
</tr>
<tr>
<td>V</td>
<td>Strong</td>
<td>Felt indoors by most, outdoors by few. Many sleeping people awake. A few are frightened. Buildings tremble throughout. Hanging objects swing considerably. Small objects are shifted. Doors and windows swing open or shut.</td>
</tr>
<tr>
<td>VI</td>
<td>Slightly damaging</td>
<td>Many people are frightened and run outdoors. Some objects fall. Many houses suffer slight non-structural damage like hair-line cracks and fall of small pieces of plaster.</td>
</tr>
<tr>
<td>VII</td>
<td>Damaging</td>
<td>Most people are frightened and run outdoors. Furniture is shifted and objects fall from shelves in large numbers. Many well built ordinary buildings suffer moderate damage: small cracks in walls, fall of plaster, parts of chimneys fall down; older buildings may show large cracks in walls and failure of fill-in walls.</td>
</tr>
<tr>
<td>VIII</td>
<td>Heavily damaging</td>
<td>Many people find it difficult to stand. Many houses have large cracks in walls. A few well built ordinary buildings show serious failure of walls, while weak older structures may collapse.</td>
</tr>
<tr>
<td>IX</td>
<td>Destructive</td>
<td>General panic. Many weak constructions collapse. Even well built ordinary buildings show very heavy damage: serious failure of walls and partial structural failure.</td>
</tr>
<tr>
<td>X</td>
<td>Very destructive</td>
<td>Many ordinary well built buildings collapse.</td>
</tr>
<tr>
<td>XI</td>
<td>Devastating</td>
<td>Most ordinary well built buildings collapse, even some with good earthquake resistant design are destroyed.</td>
</tr>
<tr>
<td>XII</td>
<td>Completely devastating</td>
<td>Almost all buildings are destroyed.</td>
</tr>
<tr>
<td>Type of Structure</td>
<td>Vulnerability Class</td>
<td></td>
</tr>
<tr>
<td>--------------------------------------</td>
<td>---------------------</td>
<td></td>
</tr>
<tr>
<td>rubble stone, fieldstone</td>
<td>A</td>
<td></td>
</tr>
<tr>
<td>adobe (earth brick)</td>
<td>B</td>
<td></td>
</tr>
<tr>
<td>simple stone</td>
<td>C</td>
<td></td>
</tr>
<tr>
<td>massive stone</td>
<td>D</td>
<td></td>
</tr>
<tr>
<td>unreinforced, with manufactured stone units</td>
<td>E</td>
<td></td>
</tr>
<tr>
<td>unreinforced, with RC floors</td>
<td>F</td>
<td></td>
</tr>
<tr>
<td>reinforced or confined</td>
<td></td>
<td></td>
</tr>
<tr>
<td>frame without earthquake-resistant design (ERD)</td>
<td>A</td>
<td></td>
</tr>
<tr>
<td>frame with moderate level of ERD</td>
<td>B</td>
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<tr>
<td>frame with high level of ERD</td>
<td>C</td>
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<tr>
<td>walls without ERD</td>
<td>D</td>
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<tr>
<td>walls with moderate level of ERD</td>
<td>E</td>
<td></td>
</tr>
<tr>
<td>walls with high level of ERD</td>
<td>F</td>
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<td>steel structures</td>
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<td></td>
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<tr>
<td>timber structures</td>
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○ most likely vulnerability class; — probable range; —— range of less probable, exceptional cases
### Classification of damage to masonry buildings

<table>
<thead>
<tr>
<th>Grade</th>
<th>Description</th>
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</thead>
<tbody>
<tr>
<td>Grade 1: Negligible to slight damage (no structural damage, slight non-structural damage)</td>
<td>Hair-line cracks in very few walls. Fall of small pieces of plaster only. Fall of loose stones from upper parts of buildings in very few cases.</td>
</tr>
<tr>
<td>Grade 2: Moderate damage (slight structural damage, moderate non-structural damage)</td>
<td>Cracks in many walls. Fall of fairly large pieces of plaster. Partial collapse of chimneys.</td>
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<tr>
<td>Grade 3: Substantial to heavy damage (moderate structural damage, heavy non-structural damage)</td>
<td>Large and extensive cracks in most walls. Roof tiles detach. Chimneys fracture at the roof line; failure of individual non-structural elements (partitions, gable walls).</td>
</tr>
<tr>
<td>Grade 4: Very heavy damage (heavy structural damage, very heavy non-structural damage)</td>
<td>Serious failure of walls; partial structural failure of roofs and floors.</td>
</tr>
<tr>
<td>Grade 5: Destruction (very heavy structural damage)</td>
<td>Total or near total collapse.</td>
</tr>
</tbody>
</table>
### Classification of damage to buildings of reinforced concrete

| Grade 1: Negligible to slight damage  
| (no structural damage, slight non-structural damage)  
| Fine cracks in plaster over frame members or in walls at the base.  
| Fine cracks in partitions and infills.  |

| Grade 2: Moderate damage  
| (slight structural damage, moderate non-structural damage)  
| Cracks in columns and beams of frames and in structural walls.  
| Cracks in partition and infill walls; fall of brittle cladding and plaster. Falling mortar from the joints of wall panels.  |

| Grade 3: Substantial to heavy damage  
| (moderate structural damage, heavy non-structural damage)  
| Cracks in columns and beam column joints of frames at the base and at joints of coupled walls. Spalling of concrete cover, buckling of reinforced rods.  
| Large cracks in partition and infill walls, failure of individual infill panels.  |

| Grade 4: Very heavy damage  
| (heavy structural damage, very heavy non-structural damage)  
| Large cracks in structural elements with compression failure of concrete and fracture of rebars; bond failure of beam reinforced bars; tilting of columns.  
| Collapse of a few columns or of a single upper floor.  |

| Grade 5: Destruction  
| (very heavy structural damage)  
| Collapse of ground floor or parts (e.g. wings) of buildings.  |
Appendix B
European Macroseismic Scale

VII. Damaging

a) Most people are frightened and try to run outdoors. Many find it difficult to stand, especially on upper floors.
b) Furniture is shifted and top-heavy furniture may be overturned. Objects fall from shelves in large numbers. Water splashes from containers, tanks and pools.
c) Many buildings of vulnerability class A suffer damage of grade 3; a few of grade 4.
   Many buildings of vulnerability class B suffer damage of grade 2; a few of grade 3.
   A few buildings of vulnerability class C sustain damage of grade 2.
   A few buildings of vulnerability class D sustain damage of grade 1.

VIII. Heavily damaging

a) Many people find it difficult to stand, even outdoors.
b) Furniture may be overturned. Objects like TV sets, typewriters etc. fall to the ground.
   Tombstones may occasionally be displaced, twisted or overturned. Waves may be seen on very soft ground.
c) Many buildings of vulnerability class A suffer damage of grade 4; a few of grade 5.
   Many buildings of vulnerability class B suffer damage of grade 3; a few of grade 4.
   Many buildings of vulnerability class C suffer damage of grade 2; a few of grade 3.
   A few buildings of vulnerability class D sustain damage of grade 1.

IX. Destructive

a) General panic. People may be forcibly thrown to the ground.
b) Many monuments and columns fall or are twisted. Waves are seen on soft ground.
c) Many buildings of vulnerability class A sustain damage of grade 5.
   Many buildings of vulnerability class B suffer damage of grade 4; a few of grade 5.
   Many buildings of vulnerability class C suffer damage of grade 3; a few of grade 4.
   Many buildings of vulnerability class D suffer damage of grade 2; a few of grade 3.
   A few buildings of vulnerability class E sustain damage of grade 2.
APPENDIX C

Vulnerability parameters and calculation procedure for Turkish building damage assessment
## Appendix C: Turkish Damage Calculation Spreadsheet

### SHEET 1 - Enter Hazard and Capacity Parameters

#### 1. Hazard Parameters

<table>
<thead>
<tr>
<th>Attenuation Relationship</th>
<th>Site Name</th>
<th>Run ID</th>
<th>Distance (km)</th>
<th>$T=0.3s$</th>
<th>$T=1.0s$</th>
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<tbody>
<tr>
<td>Gulkar &amp; Kaikan '02</td>
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<td>5.55</td>
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<td>Gulkar &amp; Kaikan '02</td>
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<td>3.95</td>
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<td>Gulkar &amp; Kaikan '02</td>
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#### 2. Capacity Parameters

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<th>BUILDING CLASSIFICATION</th>
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<th>Building Classification</th>
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<th>TC1LP</th>
<th>TC1LG</th>
<th>TC1MP</th>
<th>TC1MG</th>
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<tr>
<td>Mean Bldg Height H (m)</td>
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<td>5.6</td>
<td>5.6</td>
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<td>14</td>
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<td>Fraction of building height at elevation where pushover mode displacement equals spectral displacement $\alpha_2$</td>
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<td>Design Strength Coefficient $CS$</td>
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<td>1.25</td>
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<tr>
<td>Overstrength factor relating ultimate strength to yield strength $\lambda$</td>
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<td>Variance (beta)</td>
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Modified from original spreadsheet courtesy R.J.S.Spence
Appendix C

Turkish damage calculation spreadsheet

SHEET 2 - Calculate demand

Calculations based upon capacity spectrum method in HAZUS (FEMA, 2003)

<table>
<thead>
<tr>
<th>Hazard parameters</th>
<th>Gelinç, S. Ghulam &amp; Balkan '02</th>
<th>Capacity curve parameters</th>
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<td>$S_a(T=0.3)$</td>
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<td>$C_e$</td>
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<td>$S_a(T=1)$</td>
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<td>$T_s$</td>
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<td>$\lambda$</td>
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<td>$\kappa$</td>
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<tr>
<td>$\beta_{eff}(%)$</td>
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<td>1.25</td>
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<tr>
<td>$H (m)$</td>
<td>14</td>
<td>$\alpha_1$</td>
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<td>$\alpha_2$</td>
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<td>0.75 sec</td>
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\[
\begin{align*}
A_u & = 0.1875 \\
D_u & = 0.105469 \\
\kappa & = 0.27 \\
\beta_{eff}(\%) & = 5 \\
H (m) & = 14 \\
\alpha_2 & = 0.75 \\
\end{align*}
\]

\[
\begin{align*}
S_a(T=0.3) & = 1.039755 \\
S_a(T=1) & = 0.662569 \\
T_s & = 0.637255 \\
\end{align*}
\]

\[
A_u = 0.1875 \\
D_u = 0.105469 \\
\kappa = 0.27 \\
\beta_{eff}(\%) = 5 \\
H (m) = 14 \\
\alpha_2 = 0.75 \\
\]

Demand Difference

\[
\begin{align*}
\frac{S_d}{D_e} & = A_u \\
\beta_{eff}(\%) & = \frac{5 + \beta_{eff}}{10} \\
\end{align*}
\]

Displacement/Capacity

\[
\begin{align*}
S_a(T=0.3) & = 1.039755 \\
S_a(T=1) & = 0.662569 \\
T_s & = 0.637255 \\
\end{align*}
\]

Modified from original spreadsheet courtesy R.J.S.Spence
**SHEET 3 - Results calculator**

| Hazard Run: | 2 |
| Vulnerability Run: | W |
| Building type: | tc1mp |
| Location | Golcuk S |
| Attenuation type | Gulkan & Kalkan '02 |

### tc1mp / Golcuk S / Base Parameters

<table>
<thead>
<tr>
<th>Base Parameters</th>
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<th>Sa</th>
</tr>
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<tr>
<td></td>
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\[
P(d|s_d) = \Phi\left[\frac{1}{\beta}\ln\left(\frac{s_d}{s_{d,ds}}\right)\right]
\]

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<tr>
<th>Drift ratio</th>
<th>Undamaged</th>
<th>Slight</th>
<th>Moderate</th>
<th>Extensive</th>
<th>Complete</th>
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<td>Variance, Beta</td>
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| Pds, Sd | 0.996843 | 0.968555 | 0.836662 | 0.5859638 |
| % in ds | 0.003157152 | 0.028287 | 0.131894 | 0.250698 | 0.5859638 |
| MDR | 0.73 |

Check sum of % in ds = 1

Modified from original spreadsheet courtesy R.J.S. Spence
APPENDIX D

History of land reclamation in Hong Kong

From Hong Kong SAR Lands Department, Survey & Mapping Office
APPENDIX E

SIREN – 1-d non-linear site response analysis (Courtesy Arup Geotechnics)

Sample input data
### Site and Bedrock Details

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<th>Site X</th>
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<td>Period (s):</td>
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<table>
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<th>Bedrock Details</th>
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<tr>
<td>Elevation (m):</td>
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<tr>
<td>Density (T/m³):</td>
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<tr>
<td>Shear wave velocity (m/s):</td>
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<td>Shear modulus (kN/m²):</td>
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### Soil Properties

<table>
<thead>
<tr>
<th>Soil</th>
<th>Description</th>
<th>Density (T/m³)</th>
<th>Viscous damping ratio</th>
<th>Shear strain (%)</th>
<th>Shear modulus (kN/m²)</th>
<th>Shear stress (kN/m²)</th>
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... etc.
Page 2 – soil profile definition

### SOIL PROFILE

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<th>Stress mult.</th>
<th>yield factor</th>
<th>G max (kN/m²)</th>
<th>Vs max (m/s)</th>
<th>Nat freq (Hz)</th>
<th>Visc damp ratio</th>
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*Small strain shear modulus, $G_0$ (MPa)*

Soil number – see Page 1

G max (kN/m²) | Vs max (m/s) | Nat freq (Hz) | Visc damp ratio
---|-------------|---------------|----------------|
50000 | 160.13 | 33.98 | 0 |
53000 | 164.86 | 34.985 | 0 |
55000 | 167.94 | 35.639 | 0 |
57000 | 170.97 | 36.281 | 0 |
59000 | 173.94 | 36.912 | 0 |
62000 | 178.31 | 37.939 | 0 |
64000 | 181.16 | 38.444 | 0 |
66000 | 183.97 | 39.04 | 0 |
68000 | 186.74 | 39.627 | 0 |
70000 | 189.47 | 40.206 | 0 |
72000 | 192.15 | 40.776 | 0 |
75000 | 196.12 | 41.617 | 0 |
77000 | 198.71 | 42.168 | 0 |
80000 | 202.55 | 42.982 | 0 |
84000 | 207.55 | 33.033 | 0 |
88000 | 212.43 | 33.81 | 0 |
92000 | 217.21 | 34.57 | 0 |
95000 | 220.72 | 35.129 | 0 |
37000 | 137.75 | 43.846 | 0 |
37000 | 137.75 | 43.846 | 0 |
37000 | 137.75 | 43.846 | 0 |
37000 | 137.75 | 43.846 | 0 |
37000 | 137.75 | 43.846 | 0 |
2.80E+05 | 374.17 | 39.7 | 0 |
2.80E+05 | 374.17 | 39.7 | 0 |
2.80E+05 | 374.17 | 39.7 | 0 |
2.80E+05 | 374.17 | 39.7 | 0 |
2.80E+05 | 374.17 | 39.7 | 0 |
2.80E+05 | 374.17 | 39.7 | 0 |
2.80E+05 | 374.17 | 39.7 | 0 |
2.80E+05 | 374.17 | 39.7 | 0 |
2.80E+05 | 374.17 | 39.7 | 0 |
2.80E+05 | 374.17 | 39.7 | 0 |
Appendix E

Page 3 – Output specification. Allows displacement, velocity and acceleration time histories at each node, and shear stress and shear strain time histories for each element.

**OUTPUT SPECIFICATION**

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Page 4  Definition of input motion

**ANALYSIS CONTROL DATA**

Input time history file: 2475_2  
Title: Artificial time history  
Time step (s): 0.02  Final time (s): 82.16  
Excitation by: Base motion  
Max. value of Acceleration (m/s²): 1.104  
Multiplier on input time history: 0.9993  
Transmitting boundary at base: Yes  
Max. calculation time step (s): 0.0065  
Calculation time step (s): 2.00E-03  Finish time (s): 55  
No. of time steps between output: 10  
Input time history interpolation: Linear
APPENDIX F

Post-earthquake building damage survey sheet (proposed)
# Appendix F Post-earthquake building damage survey sheet

| Surveyed by: | .......................................................................................... |
| Date | | |
| Location | Name of city/town/village/district |
| Street name | | |
| Building Number | | |
| Map ID | assigned |
| GPS | | |

### Classification

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### Survey approach

| Spoke to owners? | | |
| Internal inspection? | | |
| Time taken | | |
| Weather | | |
| Photos | | |

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<td><em>Ground heave, sand boils, lateral spreading</em></td>
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**Follow up**

| Date of follow up survey | |
| Repair method | |
| Repair cost | |
| Building cost | |
| Insurance claim or self-funded | |

**Further comments and sketches**