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Seismic Risk Assessment of Reinforced Concrete Buildings Using Fuzzy Based Techniques

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Seismic Risk Assessment of Reinforced Concrete Buildings Using Fuzzy Based Techniques

By

Solomon Tesfamariam

A thesis submitted to
The Faculty of Graduate Studies and Research
in partial fulfilment of
the degree requirements of
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in
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Abstract

Lessons learned from performance of reinforced concrete buildings during previous earthquakes and researches over the last three decades have led to the development of improved codes for the seismic design of buildings. However, the existing buildings are vulnerable because they were designed to the older codes and/or posses structural irregularities. Most of these buildings are still occupied which makes evaluation and retrofit necessary in order to minimize the damage induced by earthquakes. Because of large volume of vulnerable buildings, however, consideration of a risk-based seismic assessment and retrofit prioritization is plausible.

Seismic (earthquake) risk is the probability (likelihood) that a specified loss will exceed some quantifiable value during a given exposure time. Seismic risk assessment is intricately dependent on site seismic hazard, building vulnerability, and importance/ exposure factor. This intricate process is modelled through a two-tier heuristics-based approach. The two-tier models utilized a hierarchical structure by incorporating wisdom and intuitive knowledge obtained from practitioners and experts.

The Tier 1 model considers building performance modifiers (factors) in congruence with the FEMA 154 rapid visual screening manual including: i) building type, ii) vertical irregularity, iii) plan irregularity, iv) year of construction, and v) construction quality. These performance modifiers can readily be obtained from a *walk down* survey and engineering drawings. The Tier 2 model is an extension of the building vulnerability module of Tier 1 that incorporates detailed performance modifiers as specified in FEMA 310.

In the present study seismic risk assessment is based on evaluating risk index of the RC building, which will be obtained following a seven-step aggregation scheme. Uncertainty due to subjectivity involved in the evaluation process is handled through the fuzzy set theory and fuzzy rule based modelling is utilized to inferences through the proposed hierarchical structure. The proposed methods are demonstrated and validated using the data from 1994 Northridge Earthquake (California) and the 2003 Bingöl Earthquake (Turkey). The proposed methodology in modular form is implemented in a prototype MS Excel based software tool (CanRisk). The Canadian site seismic hazard is also incorporated into the CanRisk.

To *my parents* and *sister* who have given me lifelong support and strength

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The exposure to the uncertain world of uncertainty is thanks to the inspiration of my two colleagues and friends, Drs. Rehan Sadiq and Homayoun Najjaran. Dr. Sherif's critical review and feedback on this thesis is very much appreciated.

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Nomenclature

A, A_i	Input fuzzy sets
$\mathrm{A_g}$	Structural walls
A_{gf}	Area of the critical story (usually the ground story)
a_i (i=1 to 7)	Coefficients of the discriminate analysis
$\mathrm{A_k}$	Masonry infill wall areas
a_L, b_M, c_H	Vertices of triangular fuzzy number, *minimum*, *most likely, maximum* values
A_{rms}	rms acceleration
A_{tr}	Tributary area for a typical column
$\mathrm{A_w}$	Effective web area of column cross section
a_x, a_y, L_x	building plan dimension used for determining plan irregularity
B_i	Consequent fuzzy set
C'	External context index for EDRI
$C1$	Concrete moment frame building
$C2$	Concrete shear wall buildings
$C3$	Concrete frames with infill masonry shear walls
C_A	Architectural feature coefficients
C_D	Collapse damage state
C_e	Elastic base shear coefficient
CI	Column ratio
C_M	Construction quality feature coefficients
D, $DI_{i\ (i=1,2)}$	Damage index
D_A	Damage level for filed-stone structures
DEQ	Damage from previous earthquake

D_H	*Heavy* damage states
D_L	*Light* damage states
D_M	*Moderate* damage states
$D_{M\text{-}D}$	*Major-destroyed* damage states
$D_{N\text{-}S}$	*None-slight* damage states
D_S	Expected damage condition of building
E'	Exposure index for EDRI
E_H, dE	Hysteretic energy demanded by earthquake ground motion
f_x, f_y	Number of continuous frame lines in the critical story in x and y directions, respectively
F_y	Yield strength
g	Ground acceleration
H'	Hazard index for EDRI
h_n	Building height (in meter)
I	Earthquake intensity
I'	Defuzzified index of each FRB
I^{BD}	Building damageability index
I^{BV}	Building vulnerability index
I_c	Characteristic intensity factor
IF1, IF2	Fully infilled frame, low and high strength masonry
IF1P	Infilled frame with open ground storey, low-strength masonry
IF2P	Infilled frame with open ground storey
$I_i^{BD'}$	Observed building damage index
I^{IE}	Building importance /exposure index
I^R	Seismic risk index
I_s	Structural seismic capacity index
I^{SH}	Seismic hazard index
K	Normalization factor for Shannon's Entropy
K_P	Number of inhabitant living in the low earthquake resistant buildings

K_R	Seismic risk connected with destruction of building
K_S	Ratio of an area occupied by relatively low earthquake resistance
L, L'	Effective and captivated column length
L_{50}	Earthquake loss expected in 50 years
L_D	Light damage state
M_D	Moderate damage state
M_{nb}	Nominal flexural resistance of the beam
M_p	Plastic moment capacity
M_{rc}	Factored flexural resistance of the column
M_w	Moment magnitude
N	Number of stories
n	Total number of rules
N_B	Number of training datasets for the RMSE
N_D	None damage state
nrr	normalized redundancy ratio
$P(I)$	Probability of earthquake intensity I
q_i	Quality-ordered weights, $q_i \in [0, 1]$
Q_y	Yield strength
R'	Emergency response and recovery capability index for EDRI
$\mathbf{R_i}$ (i = 1,2,…7)	FRB for the hierarchical earthquake risk assessment
R_i	Represents the i^{th} rule of a FRB
$R_t(T)$	Vibration characteristic factor
S_a	Spectral acceleration
$S_a(T_1)$	Spectral acceleration at the fundamental period T_1 of the building
SB	Spacing between adjacent buildings
S_D	Severe damage state
T_1	Fundamental period of a building
t_o	Story motion duration
u_y	Yield deformation

V'	Vulnerability index for EDRI
V_{code}	Code specified shear strength computation
V_e	Shear demand
$V_i\,(i = 1,\ldots,4)$	inputs into a hierarchical structures
V_{yw}	Yield base shear capacity of infill wall
w_H, w_E, w_V, w_C, w_R	Weights for the H', E', V', C' and R' used in the EDRI
WI	Infill wall ratio
y	Output (consequent) linguistic variable
Z	Seismic zone factor
η_{ci}	Strength irregularity factor
$\alpha_{i(i = 1,\,2)}$	Constant coefficients
β'	A non-negative constant of Park and Ang damage index
β	Second-moment seismic safety margin index
β''	Seismic margin index
δ_u, δ_m	Ultimate and maximum deformation under monotonic loading
$\mu_{N-S}^{BD}, \mu_L^{BD}, \mu_M^{BD}, \mu_H^{BD}, \mu_{M-D}^{BD}$	Fuzzy membership function for *none-slight*, *light*, *moderate*, *high*, and *major-destroyed* building damageability
μ	Displacement ductility
$\mu_L^{ID}, \mu_M^{ID}, \mu_H^{ID}$	Fuzzy membership function for L, M, and H of increase in demand
$\mu_L^{PI}, \mu_M^{PI}, \mu_H^{PI}$	Fuzzy membership function for L, M, and H of plan irregularity
$\mu_L^{VI}, \mu_M^{VI}, \mu_H^{VI}$	Fuzzy membership function for L, M, and H of vertical irregularity
μ_e	Maximum elastic portion of the ductility
μ_{mon}	Monotonic displacement ductility capacity
μ_x	Fuzzy membership function for uncertain quantity x
$\mu_{VL}^{BV}, \mu_L^{BV}, \mu_M^{BV}, \mu_H^{BV}, \mu_{VH}^{BV}$	Fuzzy membership function for *VL, L, M, H* and *VH* of building vulnerability

$\mu_{VL}^{SSH}, \mu_{L}^{SSH}, \mu_{M}^{SSH}, \mu_{H}^{SSH}, \mu_{VH}^{SSH}$

Fuzzy membership function for *VL, L, M, H* and *VH* of site seismic hazard

ζ Site seismic parameter obtained from historical data

Acronyms

AHP	Analytical hierarchical process
AI	Arias intensity
ASCE	American Society of Civil Engineers
ATC	Applied Technology Council
BCPI	Basic capacity index
BD	Building damageability
BF	Bare frame
BSS	Building structural system
BU_{IE}	Building use importance exposure
BV	Building vulnerability
CAV	Cumulative absolute velocity
CE	Code enforcement
CPI	Capacity index
CQ	Construction quality
DC	Diaphragm discontinuity
DQ	Design quality
DR	Decrease in resistance
EDRI	Earthquake disaster risk index
EERI	Earthquake Engineering Research Institute
E_{IE}	Economy importance exposure
EMS98	European Macroseismic Scale
EPS	Expected performance score
FEMA	Federal Emergency Management Agency
FIS	Fuzzy inference system

FRB	Fuzzy rule base
FSE	Fuzzy synthetic evaluation
GDP	Gross domestic product
H	High
HO	Heavy over
ID	Increase in demand
IO	Immediate Occupancy
L	Low
LS	Life Safety
M	Medium
MCDM	Multiple criteria decision-making
MCE	Maximum considered earthquake
MMI	Modified Mercalli Intensity
MNLSI	Minimum normalized strength ratio
MNLSTFI	Minimum normalized lateral stiffness index
MSK	Medvedev-Sponheuer-Karnik
NRS	Normalized redundancy score
OHR	Over hang ratio
O_{IE}	Occupancy importance exposure
OWA	ordered weighted averaging
PGD	Peak ground displacement
PI	Plan irregularity
RC	Reinforced concrete
REC	Re-entrant corners
Red.	Redundancy
RMSE	Minimizing the root mean squared error
RSJ	Relative strength at the joints
RSL	Risk of seismic loss
RVS	Rapid visual screening
SCE	Short column effects

SD	Structural deficiency
SDW	Structural degradation/weakening
SRA	Seismic risk analysis
SS	Soft story
SSH	Site seismic hazard
TI	Torsional irregularity
UBC	Uniform Building Code
VH	Very high
VI	Vertical irregularity
VL	Very low
WAM	Weighted arithmetic mean
WS	Weak story
YC	Year of Construction

Chapter 1
Introduction

"Earthquakes Don't Kill People, Buildings Do"

Lessons learned from performance of buildings during previous earthquakes and research over the last three decades has resulted in improved seismic design codes for buildings. However, the learning processes have to be optimized in order to focus on new lessons rather than re-learning old ones (Naeim and Lew 2000). In most earthquake reconnaissance reports, there is a recurring theme for the causes of deaths, building damage and consequently the loss of lives. This reconfirms the old adage "*earthquakes don't kill people, buildings do*."

Vulnerability of existing buildings can be attributed to design according to older codes, structural irregularities and/ or changes in initial design parameters. Most of these buildings are still occupied which makes evaluation and retrofit necessary so as to minimize damage initiated by earthquakes. Because of large volume of vulnerable buildings, however, consideration of a risk-based seismic assessment and retrofit prioritization is plausible. Seismic risk may be defined as the probability that a specified loss will exceed some quantifiable value during a given exposure time (EERI Committee on Seismic Risk 1989). A generalized notion of earthquake risk assessment is illustrated in Figure 1.1 with a Venn diagram. A seismic risk assessment can be undertaken by integrating site seismic hazard, building vulnerability (likelihood of failure), and importance/exposure factor (consequence of failure) (Figure 1.1).

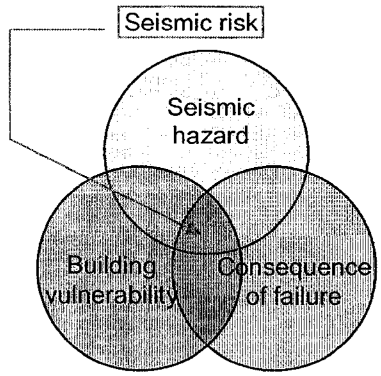

Figure 1.1: Earthquake risk assessment.

Once the seismic risk assessment is conducted and the buildings are prioritized, mitigation of any future damage has to be executed through earthquake risk management framework. The earthquake risk management entails carrying out risk assessment, evaluating different mitigation alternatives, retrofitting to bring risk to an acceptable level and considering insurance. A detailed outline of the seismic risk assessment shown Figure 1.1 is provided in Figure 1.2 that also incorporates the seismic risk management. Definitions of the nomenclature shown in Figure 1.2 are:

I^{SH}= seismic hazard index,

I^{BV} = building vulnerability index,

I^{BD} = building damageability index,

I^{IE} = building importance/exposure index,

I^{R} = risk index.

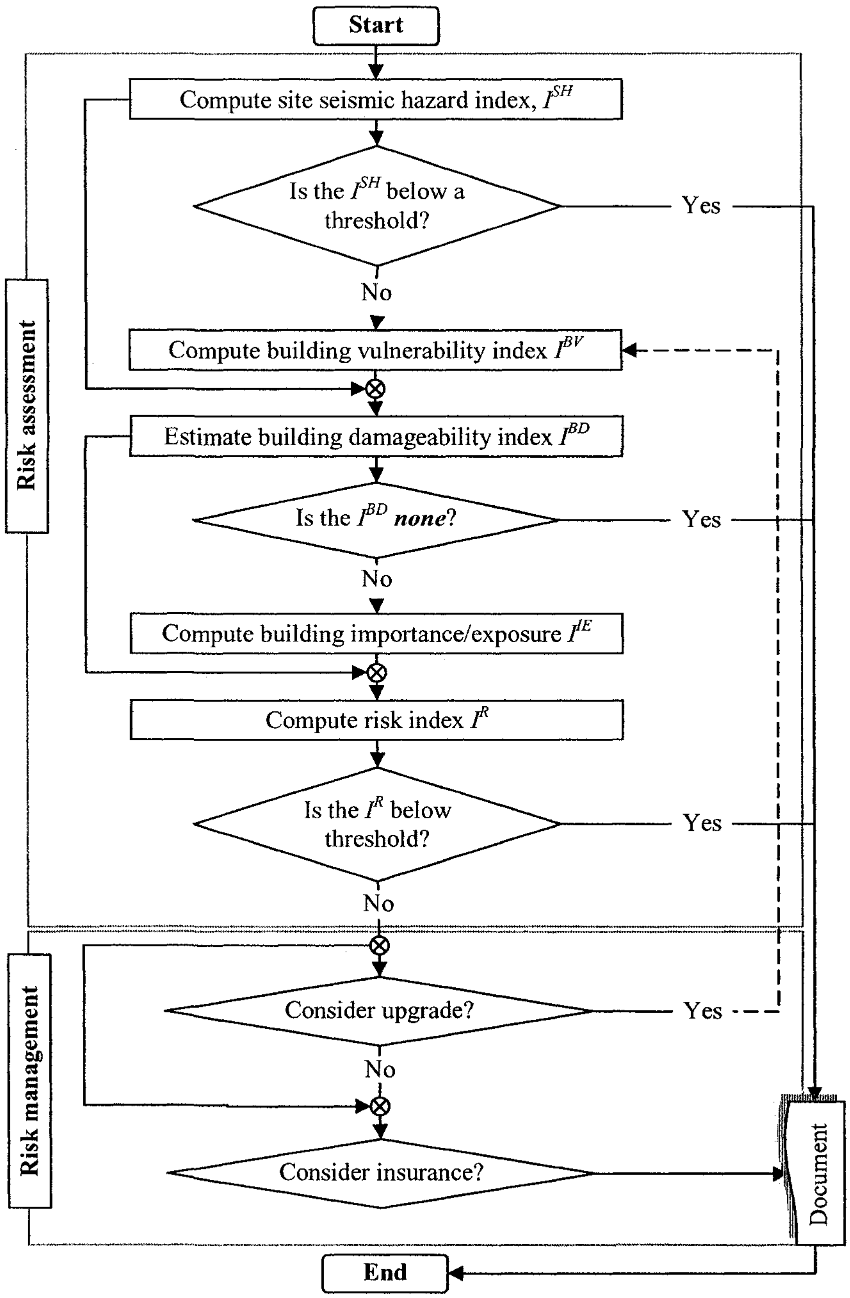

Figure 1.2: Proposed procedure for seismic risk assessment and risk management of RC buildings.

In a broader sense, earthquakes impact entire physical infrastructure, including buildings and lifelines (such as water and sewage networks, bridges, power supply). Developing a comprehensive model for all types of structures and lifelines is a daunting task; consequently, the proposed research is limited to the quantification of earthquake risk in reinforced concrete

(RC) buildings. Three RC building types considered are moment resisting frames (*C1*), moment resisting frames with infill masonry walls (*C3*) and shear wall (*C2*) buildings. However, the proposed methodology, albeit, with slight modification, can be adopted for any civil infrastructure.

The purpose of this chapter is to review the fundamental elements encapsulated in seismic risk analysis/assessment and seismic risk management. The following section will introduce the basic concepts required in earthquake risk assessment and management. At the end of this chapter, shortcomings of existing models will be highlighted and objectives and scope of research discussed.

1.1 Seismic risk analysis

Risk analysis may be defined as the probability that a specified loss will exceed some quantifiable value during a given exposure time (Ricci *et al.* 1981). Risk assessment is defined as risk analysis applied in a particular situation (Molak 1997). In this thesis, the natural hazard considered is earthquake, thus, ensuing discussion is for seismic risk analysis (SRA). The seismic risk analysis is used to calculate the probability of adverse economic or social effects of an earthquake or series of earthquakes. Results of risk analysis can be used to decide if retrofit is feasible (Dean 1997). Traditional seismic risk assessment is carried out through deterministic analysis, whereas, currently probabilistic methods are utilized (EERI Committee on Seismic Risk 1989). Probabilistic methods incorporate uncertainty and random nature of earthquakes; hence furnish more realistic results.

There are three elements of probabilistic seismic risk analysis (EERI Committee on Seismic Risk 1989):

(i) an exposure time over which the risk is evaluated,

(ii) a loss or quantification of the adverse effects (often in the form of monetary losses or loss of life), and

(iii) and a specification of the probability of incurring the loss during the specified exposure time.

Causal link in the development of seismic induced loss estimation is illustrated in Figure 1.3, which shows that the occurrence of earthquake is manifested as faulting, shaking, liquefaction, landslide, and/or Tsunami. This is followed by damage of vulnerable buildings (structural) and functional components (nonstructural). As well, there is a potential for secondary hazard/damage, i.e. fire, flooding, etc. Finally, loss follows the damage. The loss can be classified into two broad categories: primary loss and secondary loss. Different types of losses, under the two categories are summarized in Table 1.1.

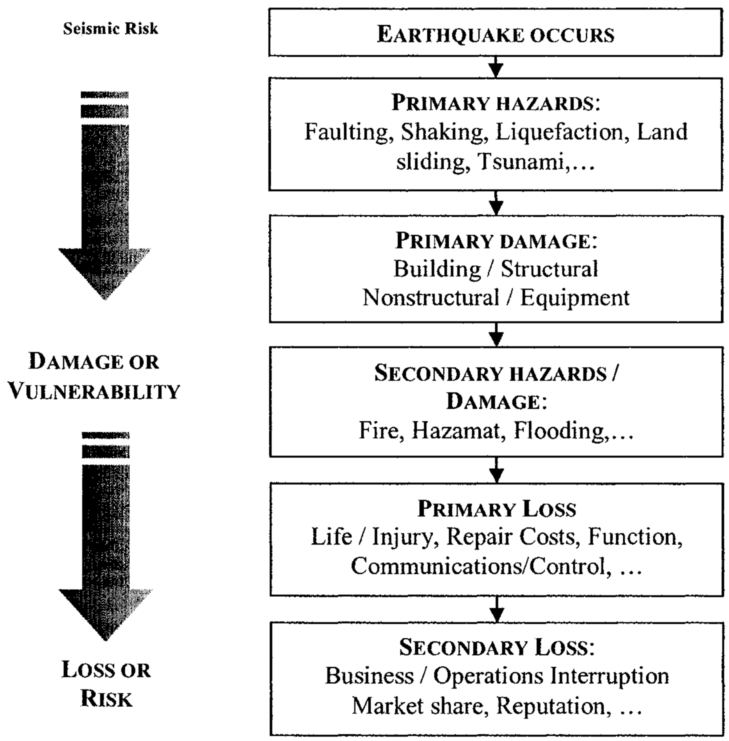

Figure 1.3: Earthquake loss process (after Scawthorn 2003).

1.1.1 Seismic hazard analysis

The objective of seismic hazard analysis is to identify the likelihood, or probability of occurrence of a specific seismic hazard, in a specific future time period, as well as its intensity and area of impact. Ultimately, it is used to assess the damaging consequence of a given earthquake, which manifests as ground shaking, liquefaction, landslides, and tsunami. The ground shaking is of particular importance since its effects spread over a larger area, and it is being the predominant cause of damage as opposed to liquefaction, landslide and tsunamis that are a direct consequence but have localized effects (EERI Committee on Seismic Risk 1989). Seismic hazard analysis is typically carried out in four distinct steps; i) seismic source characterization and assessment; ii) determining the earthquake recurrence relationship; iii) working out the ground-motion attenuation relationship; and iv) hazard assessment.

1.1.2 Building vulnerability

Building vulnerability is used to assess the performance of buildings under investigation to site seismic hazard. A well-designed building can withstand the prevalent earthquake hazard; however, due to structural degradation, year of construction (used to infer seismic codes consideration and ductility), vertical and plan irregularities, vulnerability of the building increases. Different techniques have been proposed to assess building vulnerability with different levels of complexity, ranging from a simple scoring method to more complex methods of nonlinear structural analyses (FEMA-249 1994; Ghobarah 2001; Boissonnade and Shah 1985). The complexity of building assemblage model and its response to seismic loading can be handled through system theory.

A system is defined as an "assemblage of components acting as a whole" (Meirovitch 1967). Building structures are essentially an assemblage of different components, e.g. beams, columns, slabs; hence can be described as a *system*. Each system in turn encapsulates different subcomponents each of which can be described as a subsystem. In structural safety and evaluation, system response to earthquake loading is of paramount importance. The system can be represented using continuous or discrete analytical models. Typically, system identification technique (Yao 1985) is used to develop and validate the model. The different techniques can be described through mathematical models, which are an abstraction of the actual building. Joslyn and Booker (2005) have succinctly described the limitations of models: *all models are necessarily incomplete*; *all models are necessarily somewhat in error*; and *the system being modeled may have inherent variability or un-measurability in its behavior*. Nevertheless, despite these limitations, systems approach of building assessment has a utility in screening deficient buildings.

1.1.3 Importance factor

Importance factor is used to assess the functional use of a building after earthquake. Typically, two performance levels are defined for both structural and nonstructural components: Life Safety (LS) and Immediate Occupancy (IO). Structures needed for post-earthquake recovery are evaluated for immediate occupancy; hence, the damage tolerance is minimal. On the other hand, LS performance is to avoid collapse, and consequently save lives.

1.1.4 Damage and loss estimation

The key building block for estimating seismic risk is to estimate damage to infrastructure as a function of ground motion. The ground shaking is the classic intermediate step between earthquake occurrence and damage, because it allows the use of results from one earthquake (e.g., empirical observations of damage) to estimate damage from future events. The major impact of earthquakes is loss of human life. The definition of loss or quantification of an "adverse effect," is a value judgment (Molak 1997). There is nothing inherent in an SRA that specifies an acceptable risk for society or a property owner. Hence defining this threshold is a challenging task.

Table 1.1: Earthquake loss[§].

Direct losses	Indirect losses	
loss of life loss of property	Disruption of banking industry	• Government • Individuals
	Disruption of social life	• Family separation • Emotional distress • Physical distress • Community stress • Unemployment.
	Disruption of services	• Highways • Electric power • Communications • Water • Sanitation • Supplies • Transportation • Fire protection • Police

[§]Adopted from (Seismic Safety Commission 1999)

1.2 Seismic risk management

Risk Management entails the process of quantifying risk and subsequently developing strategies to manage the risk. Two options that can be combined in a global risk-management strategy are (Paté-Cornell 1996):

(i) insurance and loss sharing in case of an accident (or a damaging earthquake), and

(ii) reducing expected loss by implementing preventive or mitigation measures.

Earthquake loss, earthquake threat and mitigation alternatives are summarized in Table 1.2. The risk management is a process of weighting alternatives (options) and selecting the most appropriate action, integrating the results of risk assessment with engineering data, social, economic, and political concerns to reach an acceptable decision. Generally, risk assessment process involves objectivity, whereas risk management involves preferences and attitudes, which have both objective and subjective elements (Asante-Duah 1993). Risk management poses a challenge for decision maker to select an alternative based on multiple and often-conflicting criteria that necessitate the use of decision support aid. Multiple criteria decision-making (MCDM) is used for this purpose. MCDM techniques deal with problems whose alternatives are predefined and the decision-maker ranks available alternatives. MCDM has proved to be a promising and growing field of study since early 1970s and many applications in the fields of engineering, business and social sciences have been reported. Carlsson and Fullér (1996) classified MCDM methods into four distinct types, namely (i) outranking, (ii) utility theory (iii) multiple objective programming, and (iv) group decision and negotiation theory.

Table 1.2: Buildings at risk§.

Asset: Loss	Earthquake Threat	Mitigation Alternatives
People: death and injury	Building damage/collapse, via fault rupture, shaking, ground failure, etc.	• Investigate site for potential faulting • Strengthen the building • Base isolate the building • Provide supplemental damping • Provide ground or foundation improvements, if ground failure is the issue • Replace the building (i.e., move, or new construction)
	Building contents damage	• Inventory all contents and brace or otherwise reduce damage • Modify building motions via base isolation, supplemental damping, etc
	Equipment malfunction	• Identify and review critical equipment for continuity of functionality during and after an earthquake (e.g., check for relay chatter, backup power, water, fuel, etc.) • Assure equipment will not be damaged (i.e., brace, etc.) • Provide redundant equipment • Develop emergency plans and procedures for equipment malfunction
	Offsite threats	• Identify and review neighbourhood for earthquake hazards (e.g., tsunami, landslide) and threats (e.g., nearby hazardous operations, such as a chemical process plant) • Develop Emergency plans and procedures, including possible warning mechanisms • Build protective barriers • Acquire protective equipment and training (e.g., fire brigades) • Modify offsite threat (e.g., earthmoving, for a landslide; or buy out nearby hazardous operations; or move)
Property: financial loss	Same as above Inventory	• Same as above, plus • Emergency plans and procedures to minimize damage (e.g., recovery of inventory, quick shut-down of broken sprinklers) • Earthquake insurance
Function: business interruptions, revenue, market share	Same as above plus loss of infrastructure (e.g., transportation), loss of vendors	• Contingency planning for loss/replacement or recovery of facilities (e.g., backup sites or suppliers, rapid recovery via pre-arranged inspection and repair contractors) • Financial planning for loss of revenue • Earthquake/loss of profits insurance • Planning for alternative production/transportation to maintain market share

§Adopted from (Seismic Safety Commission 1999)

1.3 Research needs

The seismic risk assessment requires consideration of site seismic hazard, building vulnerability and building importance and exposure factors, which require a multidisciplinary approach. With high inventory of buildings, however, a thorough investigation of individual buildings is not feasible due to limited human resources and available funds, which highlights the importance of using a simple *walk down* survey. However, the *walk down* survey is prone to subjectivity of the evaluator, and vagueness uncertainty is introduced. Furthermore, the final decision is affected by different consequence of failures, which necessitates risk-based assessment. Thus, there is a need for developing a risk-based rapid visual screening method and tool to screen out deficient buildings for further investigation. Also, the proposed method needs to consider the vagueness uncertainty.

1.4 Objective

The objective of this thesis is to develop a two-tiered decision-making tool for seismic risk assessment of RC buildings. The decision making process for the two-tiered model is schematically depicted in Figure 1.4. Details of the two-tier model are described below;

- The Tier 1 model utilizes a hierarchical structure that encapsulates site seismic hazard, building vulnerability and importance/ exposure factor. Tier 1 model entails integration of building vulnerability information obtained from a *walk down* survey and engineering drawings. Furthermore, information on the building vulnerability is aggregated with site-specific seismic hazard and consequence of damage. The ultimate objective of Tier 1 evaluation is to develop a simple, yet, intuitive integration of different building performance modifiers. Epistemic uncertainty due to the subjective judgment of the *walk down* survey is handled using fuzzy set theory.
- Tier 2 model is intended for buildings found to have high risk and/ or high uncertainty from results of the Tier 1 model, and entails the quantification of performance modifiers and their integration through a hierarchical-based heuristic model. Similar to the Tier 1 evaluation, epistemic uncertainty is captured using a fuzzy based technique. The originality of this model is the computation of earthquake demand and structural capacity.

A global flow chart for the Tier 1 and 2 evaluations are depicted in Figure 1.4. The decision to proceed with the suggestion of retrofit alternatives depends on the result of Tier 1 or 2 evaluation, and the decision maker's ultimate objective.

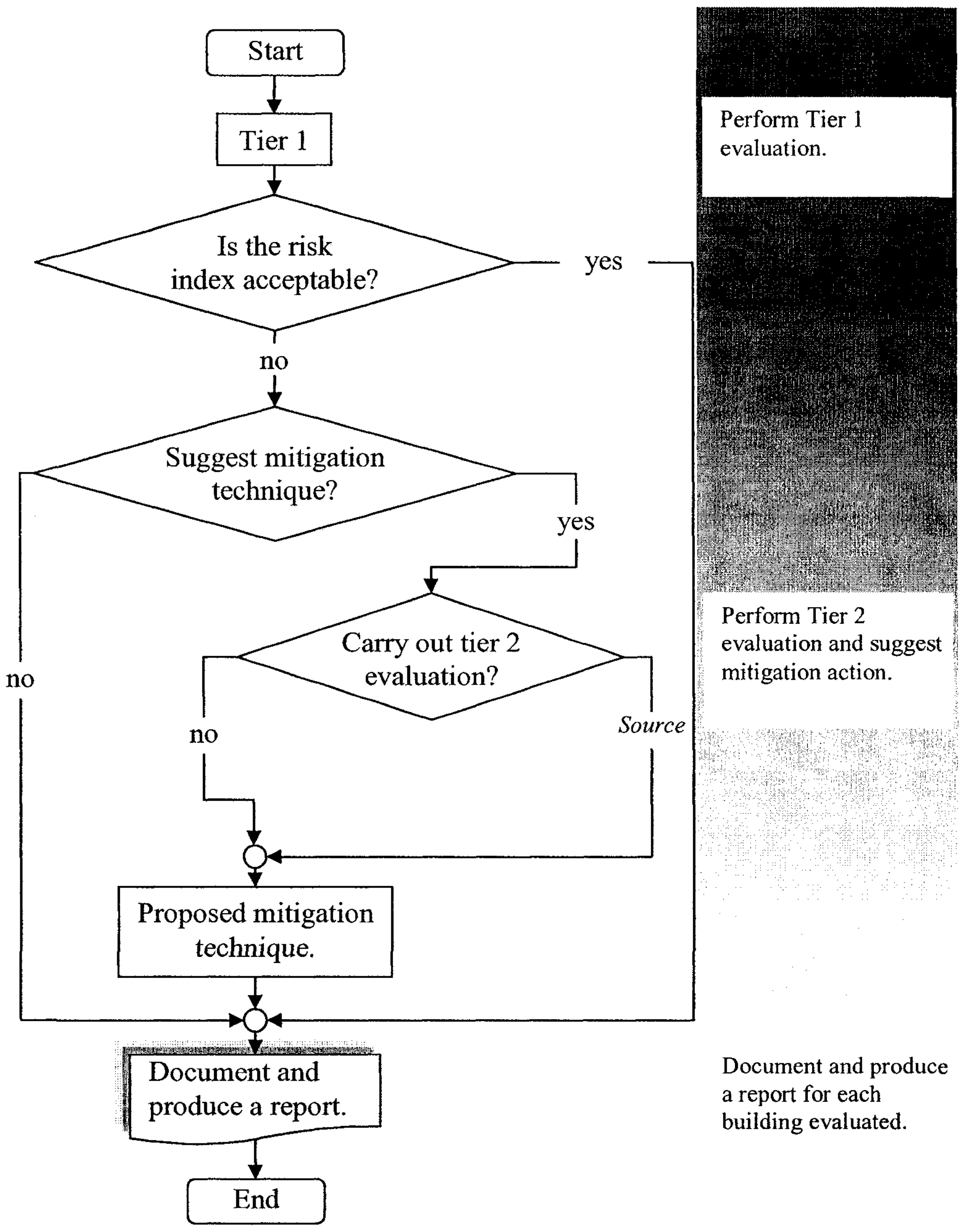

Figure 1.4: Flow Chart of the risk based building evaluation techniques.

1.5 Scope

Scope of the proposed research is as follows:

- Review available literature on risk-based seismic evaluation of structures.
- Identify critical parameters for Tier 1 and Tier 2 building vulnerability assessments.
- Integrate site seismic hazard with building vulnerability through a fuzzy-based technique to quantify building damageability.
- Incorporate the importance and exposure of a building into building damageability through a fuzzy-based technique to quantify risk.
- Develop a computer-based tool for Tier 1 and Tier 2 evaluations.
- Demonstrate the applicability of procedures by using case studies.
- Present the procedures and prepare a thesis.

1.6 Thesis structure

The structure of this thesis is depicted Figure 1.5and briefly discussed below:

Chapter 1 presents a general background discussion on risk analysis, earthquake risk assessment, and risk management. A general outline of parameters considered in risk assessment is outlined. Further, the objective and scope of the thesis are provided.

Chapter 2 provides a state of the art literature review on building vulnerability assessment of RC buildings.

Chapter 3 develops a Tier 1 model for seismic risk assessment of RC buildings. The Tier 1 model utilizes a hierarchical structure that encapsulates site seismic hazard, building vulnerability and importance/ exposure factor. Use of soft computing technique to propagate uncertainty through the Tier 1 model is discussed, and the weighted average mean (WAM), ordered weighted averaging (OWA) and fuzzy synthetic evaluation are explored. Importance of performance modifier is generated through a relative weighting for each parameter using the analytical hierarchical process (AHP). The vagueness uncertainty propagation is undertaken using fuzzy synthetic evaluation (FSE). The efficacy of the FSE method is illustrated through the 2003 Bingöl Earthquake and 1994 Northridge Earthquake data.

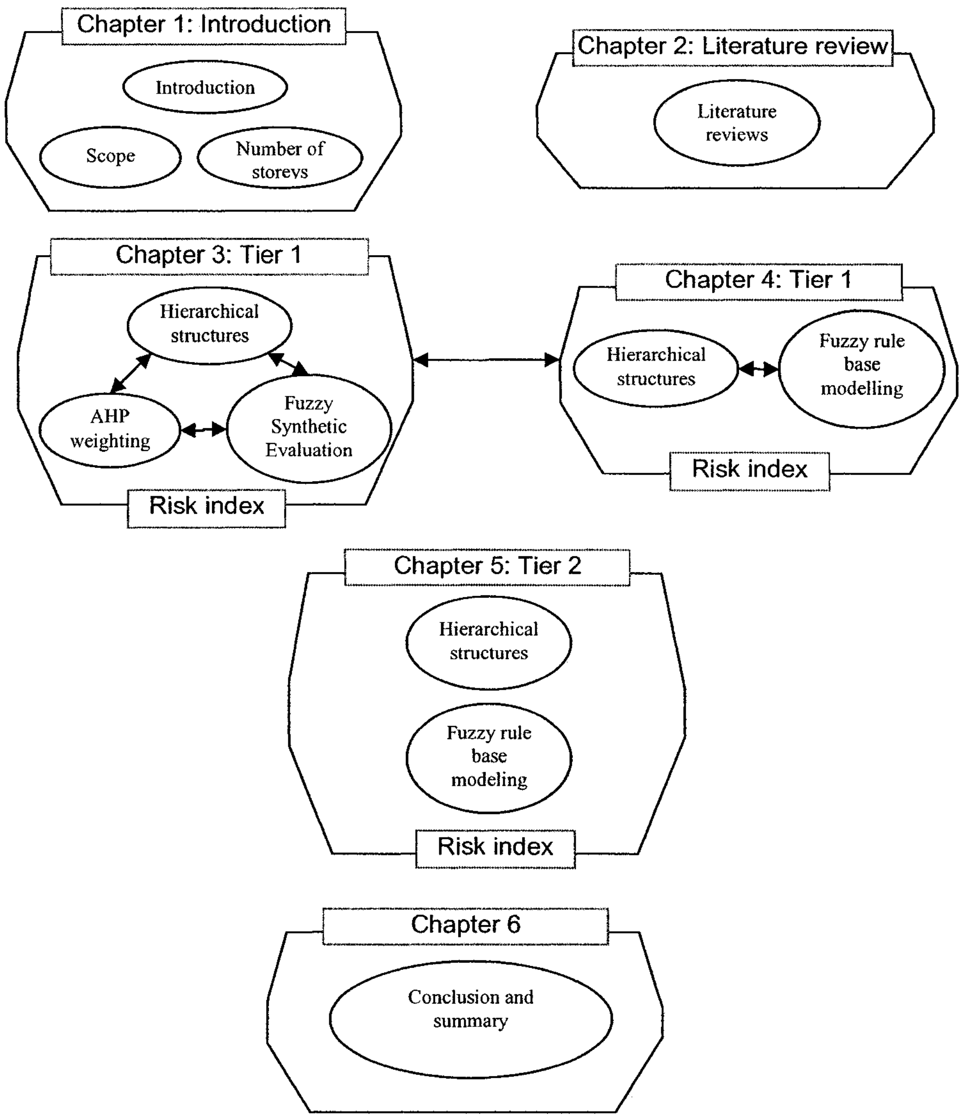

Figure 1.5: Organisation of thesis.

Chapter 4 demonstrates the use of Tier 1 model, where vagueness uncertainty is handled through fuzzy set theory and the uncertainty propagation is carried out through fuzzy rule base modelling. A case study is provided based on the 1994 Northridge Earthquake data. Finally, Results of the Tier 1 model are used to decide if Tier 2 model is warranted.

Chapter 5 proposes and develops a Tier 2 model that considers detailed building vulnerability assessment for RC buildings. The uncertainty propagation is aggregated through the fuzzy rule base modelling. A case study is provided based on the 2003 Bingöl Earthquake.

Chapter 6 summarises the work carried out in the current study and outlines future research needs.

Chapter 2
Building Vulnerability Assessment-State-of-the-art Review

Reasonable trade-offs, be they with respect to operating regulations, below-standard performance, or system malfunction, cannot be made without a quantitative method of evaluating the seismic risk at a site.
Allin Cornell (1968)

An integral component of seismic risk assessment is building vulnerability. Results of the building vulnerability provide relationship between seismic intensity and performance of the structure presented as a damage state or damage ratio (Boissonnade and Shah 1985). The damage states are characterized either by a simple linguistic statement (for example, *light*, *moderate*, *severe*; or *safe*, *uncertain*, *unsafe*) or by a more refined description (number of cracks, width of cracks in structural and nonstructural elements, strength, displacement, and so on). Life Safety (LS) and Immediate Occupancy (IO) performance levels are used to determine acceptable damage states. The damage ratio is characterized either by the ratio of repair cost to replacement cost or by the ratio of repair cost to market value.

A building located in a high site seismic hazard (SSH), but designed with the current state-of-the-art code and non-deficient structural components may incur little or no damage. Similarly, a building in a very low SSH, even when designed with an older design code and exhibiting structural deficiency, will not be damaged. Building vulnerability is intricately dependent on the SSH and building design consideration (e.g. design code, continuity in the lateral load transfer mechanisms).

Detailed analysis of all buildings is not economically feasible and hence initial screening techniques are used to prioritize critical building for a detailed non-linear analysis. Various techniques are used to carry out building vulnerability assessment and loss estimation, albeit similar techniques, different nomenclature are reported. These techniques can be grouped into (FEMA-249 1994; Ghobarah 2001; Boissonnade and Shah 1985); empirical/statistical models; heuristic models; and analytical/mechanistic/theoretical/ engineering models. At times, where appropriate, a combination of these methods is used (Kappos *et al.* 1998; Dolce 2006). For brevity, in this thesis, the taxonomy used is *empirical*, *heuristic*, and *mechanistic models*.

The purpose of this chapter is to review available techniques used in the RC building vulnerability and seismic risk assessment. Boissonnade and Shah (1985) have carried out a thorough review of the damage models used to assess single and group of structures; and structural elements. Thus, this review is limited to techniques used and proposed within the last twenty years.

2.1 Empirical models

Empirical models involve gathering and correlating ground motion and damage information. They are based on statistical observations of buildings damage collated from earthquake reconnaissance reports. These methods often encompass different building types and correlate damage ratio with Modified Mercalli Intensity (MMI) scale. The damage versus ground motion relationship furnishes average damage ratio for a group of buildings having similar structural systems and materials of construction. However, little is known about the uncertainty of this ratio except that it is large. These methods are applicable for regional damage estimation and cannot be used for an individual building.

In the seismic hazard analysis, scatter in the MMI about the mean value represents uncertainty. The potential source of uncertainty (McGuire 2004) is inconsistency in the MMI assignments of ground motion intensity and damage estimation. Moreover, the MMI assignments are average observations over a community or small region. The use of MMI in damage estimation is criticized since it uses a *circular logic* (FEMA-249 1994) where the damage is estimated from ground-motion estimates, which in turn are derived from damage. The current state of practice for undertaking rapid loss estimation studies considers MMI values, although use of peak ground acceleration (PGA) and peak ground displacement (PGD) levels is growing in more detailed loss estimation studies. However, for studying past earthquakes, where no instrumental record is available, the MMI still has utility (Boissonnade and Shah 1985).

Reported techniques for quantifying building vulnerability using the empirical model are summarized in Table 2.1 and discussed further below.

Singhal and Kiremidjian (1996) have developed analytical fragility curves to estimate probability of exceeding different degree of damage. Singhal and Kiremidjian (1998) used Bayesian statistics coupled with analytical fragility curves derived from reported earthquake induced building damage. Most of their data was obtained from the 1994 Northridge earthquake. The ground motion severity parameter utilized was spectral acceleration (S_a). Due to limited available data, the Bayesian updating was carried out for low-rise RC frames.

Cabañas *et al.* (1997) developed a procedure to quantify structural damage due to earthquake induced ground motions. In their procedure arias intensity (AI) and standard cumulative absolute velocity (CAV) earthquake intensity measures are obtained from strong motion records. Initially, a relationship between field earthquake intensity parameter, such as Medvedev-Sponheuer-Karnik (MSK) and, AI and CAV, is established. Three types of building structures are considered; filed-stone, ordinary brick building and precast concrete skeleton construction. Using a regression analysis, an exponential relationship is established between the observed damage states and corresponding earthquake intensity measures (AI and CAV). The data used in the analysis were obtained from four Italian earthquakes: 23

November 1980 Campano-Lucano, 29 April 1984 Unbria, 29 April 1984 Lazio-Abruzzo, and 11 May 1984 Lazio-Abruzzo.

Table 2.1: Summary of empirical damage assessment

Author(s)	Data used	Input parameters	Damage equation	Remark
Singhal and Kiremidjian (1996)				Analytically formulated fragility cures for RC frames
Singhal and Kiremidjian (1998)	1994 Northridge earthquake	Spectral acceleration (S_a); Low rise concrete building	Park and Ang (1985) damage index.	Bayesian method is used to update fragility cures for low-rise RC frames.
Cabañas *et al.* (1997)	Earthquakes: 1980 Campano-Lucano, Unbria, 1984 Lazio-Abruzzo	Arias intensity, standard cumulative absolute velocity; Type of building structure	$\text{Log(AI)} = 0.75\ D_A + 1.49$ $\text{Log(CAV(20))} = 0.75\ D_A + 4.10$ D_A: Damage level for filed-stone structures	Average D_A estimation tool to assess the destructiveness of earthquake intensity
Davidson and Shah (1997)		Factors related to H', E', V', C', R' And corresponding weights w_H, w_E, w_V, w_C, w_R	$\text{EDRI} = w_H H' + w_E E' + w_V V' + w_C C' + w_R R'$ EDRI: earthquake disaster risk index; H: hazard; E: exposure; V: vulnerability; C: external context; R: emergency response and recovery capability indexes w_H, w_E, w_V, w_C, w_R: corresponding weights of H', E', V', C', and R'.	EDRI is global risk assessment technique good for quantifying resiliency of cities the final value is risk index and is not associated with any damage
Chan *et al.* (1998)		GDP, earthquake intensity	$L = \sum_I P(I) \cdot F(I, \text{GDP}) \cdot \text{GDP}$ L: loss expected; P(I): probability of earthquake intensity I; GDP: gross domestic product.	F(I, GDP) is a measure of the area's vulnerability to earthquake damage and the vulnerability of inventory in the region
Yong *et al.* (2002)			Risk index = L_{50} / GDP x100 L_{50} = earthquake loss expected in 50 years.	- L_{50} is a loss estimation computed from Chan *et al.* (1998)
Balassanian *et al.* (1999)		Seismic hazard I_{hz}, resistance of building $I_{rl,r}$.	$\text{RSL} = K_R \times K_S \times K_P$ RSL: risk of seismic loss; K_R: seismic risk connected with destruction of building ($I_{hz}/I_{rl,r}$); K_S: ratio of an area occupied by relatively low earthquake resistance; K_P: number of inhabitant living in the low earthquake resistant buildings.	
Yücemen *et al.* (2004)	1999 Duzce earthquake	Structural parameters	$D_s = a_1 N + a_2\ \text{SSI} + a_3\ \text{OHR} - a_4\ \text{MNLSTFI} - a_5\ \text{MNLSI} + a_6\ \text{NRS} + a_7$ D_s: Expected damage condition of buildings; a_i (i=1 to 7): constants; N: Number of stories; SSI: soft story index; OHR: over hang ratio; MNLSTFI: minimum normalized lateral stiffness index ; MNLSI: minimum normalized strength ratio; NRS: normalized redundancy score.	Discriminate analysis; The D_s value gives the expected damage condition of the building. This model do not take the SSH into consideration.

Davidson and Shah (1997) have developed an earthquake disaster risk index (EDRI) to assess urban cities. The EDRI quantifies earthquake risk by integrating hazard (ground shaking and collateral hazards), exposure and vulnerability (physical infrastructure, population, economy, social-political system), external context (economy, transportation, politics, culture) and emergency response and recovery capability (planning, resources, mobility and access). This method gives the general risk assessment of a city or country, and helps in allocating the mitigating resources at a global level. Davidson and Shah (1997) have used the aforementioned indicators to assess the global risk for different seismically active cities. Zobin and Ventura-Ramírez (2004) have applied the EDRI method for four towns of Colima state in México.

Chan *et al.* (1998) have developed a general method of estimating earthquake loss by considering gross domestic product (GDP). In comparison with the conventional approach, this method bypasses the difficulty of collecting detailed information and makes it possible to conduct loss assessment for any region with the available GDP and population data (Yong *et al.* 2002). Yong *et al.* (2002) have used this method for the estimation of seismic hazard and loss in Central America. Balassanian *et al.* (1999) have developed a similar method by considering building vulnerability, area of vulnerability, and number of inhabitants.

Yücemen *et al.* (2004) have developed a damage estimation method for low-rise RC buildings subject to severe earthquakes. They used discriminate analysis statistical technique. The modeling parameter considered in their method are number of stories, soft story index, overhang ratio, minimum normalized lateral stiffness and strength index, and normalized redundancy score. The observed damage states are categorized into five discrete states; none (D_N), light (D_L), moderate (D_M), severe (D_S) and collapse (D_C). Based on the five discrete damage states, two models are developed for LS and IO performance levels. For the LS model, two discrete damage states are considered (D_N+ D_L+D_M) and (D_S+D_C). For the IO model, the five damage states are grouped into (D_N+ D_L), (D_M), and (D_S+D_C). The proposed damage models do not consider earthquake magnitude and geotechnical variation. To increase the utility of this model, site seismic hazard, geotechnical conditions and consequence of failure can be incorporated. The model was validated using the 1992 Erzincan, and 1999 Bolu, Düzce, and Kaynasli earthquakes.

2.2 Mechanistic methods of damage estimation

Mechanistic methods are most commonly used to forecast damage for a single structure and consider dynamic characteristic of the RC building. This entails linear or non-linear analysis of the structure. They usually correlate ground motion intensity characteristics, such as PGA, velocity, or spectral acceleration, with response characteristics of the structures (stress, strain, inter story drift). Advantage of the mechanistic method is its ability to correlate to physical parameters used by engineers in seismic design. The main drawback is challenges involved in model development and computational efforts (Villaverde 2007). However, empirical relationships based on past damage data and engineering judgments are used to correlate these structural response parameters with the damage ratio.

Different mechanistic model for building vulnerability assessment are reported. Table 2.2 lists the available damage estimation and/or forecasting methods.

Park and Ang (1985) developed a mechanistic seismic damage index D to assess vulnerability of RC buildings. The damage index D is expressed as a linear combination of the damage caused by excessive deformation and hysteretic deformation. The damage potential of the ground motion is described by a "characteristic intensity" I_c factor, which also incorporates various combination of rms acceleration A_{rms} and story motion duration t_o. The proposed method is used to assess storey-level damage and overall damage sustained by a building. Building damage from two earthquakes, 1971 San Fernando earthquake and 1978 Miyagiken-Oki earthquake, is used for model calibration and verification.

Kanda *et al.* (1997) have developed a probability-based seismic safety evaluation of existing buildings through second-moment seismic safety margin index β. The β is used to assess damage at different story levels. Ground motion model incorporates duration of the motion, PGA, and *in situ* geotechnical condition. The proposed method was applied to eleven existing buildings in Japan.

Hassan and Sozen (1997) and Gülkan and Sozen (1999) have developed a simple analytical vulnerability index that considers orientation and cross-sectional size of the vertical components. The building type considered comprises of low- to mid-rise RC buildings. The Hassan and Sozen (1997) method entails computation of two factors, infill wall ratio (*WI*) and column ratio (*CI*). The *WI* is calculated by summing the area of shear and infill walls and subsequently normalized by the total floor area. Similarly, the *CI* is computed by normalizing the column area. Finally, the *WI* and *CI* values are plotted, which show a triangular damage state formulation at a specified threshold of *WI* and *CI* value.

Gülkan and Sozen (1999) have analytically showed the validity of the triangular formulation of Hassan and Sozen (1997), albeit, with slight modification of the vertices of the triangle. Yakut (2004) has improved the Hassan and Sozen (1997) and Gülkan and Sozen (1999) method by incorporating indices related to building configuration, construction quality related detailing and lack of technical control. The final capacity index (CPI) is computed by integrating basic capacity index (BCPI), architectural feature coefficients C_A, and construction quality feature coefficients C_M. This simple analytical technique can be used to discern buildings vulnerable to seismic damage. The above three reported studies do not consider site seismic hazard and importance factor of the building, and thus do not explicitly consider risk.

Table 2.2: Summary of mechanistic damage assessment.

Author(s)	Data used	Input parameters	Damage equation	Remark
Park and Ang (1985)	1971 San Fernando earthquake 1978 Miyagiken-Oki earthquake		$D = \frac{\delta_m}{\delta_u} + \frac{\beta'}{Q_y \delta_u} \int dE$ *D*: Damage index; Q_y: Yield strength; $\int dE$: dissipated hysteretic energy; δ_u: ultimate deformation under monotonic loading; δ_m: maximum response deformation; β': a non negative constant	$D > 1$ signifies collapse $D \leq 0.4$ signifies repairable damage Low rise and medium rise RC frames
Kanda *et al.* (1997)	Artificially generated ground motion		$\beta'' = \dfrac{\ln m_R - \frac{\zeta_R^2}{2} + \frac{\ln m_R}{\ln m_a}\left(\frac{\zeta_R^2}{2} + \ln \frac{a_v}{\mu_a}\right)}{\sqrt{\zeta_R^2 + \left(\frac{\ln m_R}{\ln m_a}\zeta a\right)^2}}$ β'': seismic margin index; ζ: site seismic parameter obtained from historical data	The structural resistance and elastic response are assumed to be log normally distributed The ultimate limit state are computed for shear strain, inter-story deflection, and cumulative plastic deformation ratio
Hassan and Sozen (1997)	1992 Erzican earthquake	Wall index (WI), column index (CI)	*If* (WI + CI) < 0.25 *then* sever damage *If* 0.25 < (WI + CI) < 0.50 *then* moderate damage *If* (WI + CI) > 0.50 then light damage	Simple analytical and heuristic technique used to asses building vulnerability
Gulkan and Sozen (1999)	1992 Erzican earthquake	Wall index, column index	Similar formulation to Hassan and Sozen (1997), with different boundary condition	Analytical techniques without taking shear wall into consideration
Yakut (2004)	1992 Erzican earthquake; 2002 Sultandagi earthquake; 2003 Bingöl earthquake		CPI = C_A C_M BCIP CPI: capacity index; C_A: architectural feature coefficients; C_M: construction quality feature coefficients; BCPI: basic capacity index; BCPI = V_{yw} / V_{code}; V_{yw}: yield base shear capacity of infill wall; V_{code}: code specified shear strength computation	A simple analytical technique used to discern vulnerable buildings A CPI value ranging from 1.1 to 1.2 found to correlate with actual data (used for classification purpose) A CPI cut off 1.5 is used for further analysis
Bozorgnia and Bertero (2003)	1994 Northridge and 1992 Landers earthquakes in the US; 1999 Kocaeli and Duzce earthquakes in turkey		$DI_1 = [(1-\alpha_1)(\mu - \mu_e)/(\mu_{mon} - 1)] + \alpha_1 (E_H / F_y u_y)/(\mu_{mon} - 1)$ $DI_2 = [(1-\alpha_2)(\mu - \mu_e)/(\mu_{mon} - 1)] + \alpha_2 [(E_H / F_y u_y)/(\mu_{mon} - 1)]^2$ DI_i (i = 1, 2): damage indices; α_i (i = 1, 2): constant coefficients; μ: displacement ductility; μ_e: maximum elastic portion of the ductility; μ_{mon}: monotonic displacement ductility capacity; E_H: hysteretic energy demanded by earthquake ground motion; F_y: yield strength; u_y: yield deformation.	A simple analytical technique used to estimate damage index. This method is calibrated against the Park and Ang (1985) analytical and experimental data
Otani (2000)		Earthquake spectrum	$I_s = \frac{C_e}{ZR_t(T)}$ I_s: structural seismic capacity index; C_e: elastic base shear coefficient; *Z*: seismic zone factor; $R_t(T)$: vibration characteristic factor	$I_s < 0.3$ likely collapse; $0.3 < I_s < 0.6$ possible to collapse $I_s \geq 0.6$ unlikely to collapse

Bozorgnia and Bertero (2003) have developed damage spectra to quantify the damage potential of recorded earthquake ground motion. The damage spectra are based on a combination of normalized hysteretic energy and deformation ductility of a series of inelastic single-degree-of-freedom systems. Once the damage spectra ordinates for a set of ground motion records are computed, the variation in damage spectra is correlated with source-to-site distance. The proposed method is utilized for the 1994 Northridge and 1992 Landers Earthquakes in the US, and 1999 Kocaeli and Duzce Earthquakes in turkey.

Otani (2000) illustrated seismic vulnerability assessment method for RC buildings in Japan. The method discussed is derived from building design equations, where the seismic capacity is obtained by considering capacity and demand. The method is expanded for multi degree freedom system and calibrated to account for discontinuity in stiffness along the building height and eccentricity in plan.

2.3 Heuristic models

Although the empirical and mechanistic models are easy to apply, they have serious limitations. Lack of available data and incomplete knowledge base, necessitate the use of heuristic models. Various authors have proposed heuristic damage assessment models (e.g., Miyasato *et al.* 1986; Furuta *et al.* 1991). The heuristic models have been used in conjunction with empirical and mechanistic models.

Miyasato *et al.* (1986) have developed a simple hierarchical seismic risk evaluation method. The hierarchical structure is developed heuristically and is calibrated for structures in California. Uncertainty is captured through Certainty Factor.

Gülkan and Yakut (1996) have developed a rule base expert system for damage quantification in RC buildings. The main objective of this expert system is to tag buildings prone to failure in a subsequent after shock, and hence to save lives. The rule based expert systems integrate severity of the member damage states, extent of damage and relative importance of the structural component to obtain member structural damage score. This procedure is applied to the whole building, and using the if-then inference mechanism, the final damage state is determined.

Sucuoğlu and Yazgan (2003) have developed a heuristic model for building vulnerability assessment. The proposed method is a two level process, where level 1 entails observation from the street, while in level 2, structural data is obtained for each building. The proposed method is similar to FEMA 154 (ATC 2002a) and calibrated for RC buildings in Turkey. The calibration is undertaken using a multivariate analysis. Once the basic information is obtained through level 1 or level 2, aggregation is done using a weighted arithmetic mean to obtain final performance score.

Part of the heuristic models commonly used is the rapid visual screening of buildings: FEMA 154 (ATC 2002a) and Handbook for the Seismic Evaluation of Buildings: FEMA 310 (ASCE 1998). FEMA 154 type evaluation is also reported by Steimen *et al.* (2004). In their study damage level was defined in accordance with the definition of European Macroseismic Scale (EMS98). The information for each building was collected from a sidewalk, and they indicated that classification was prone to subjective error. As a result, large variability between the observed and actual data was observed.

This thesis builds on the building performance modifiers identified in FEMA 154/155 and Tier 1 evaluation of FEMA 310. The two methods will further be expounded in more details.

2.4 Regional damage estimation

2.4.1 Rapid Visual Screening of Buildings for Potential Seismic Hazard: A Handbook (FEMA 154 report - Second Edition)

The rapid visual screening (RVS) (ATC 2002a) procedure presented in the FEMA 154 report has been formulated to identify, inventory, and rank buildings that are prone to seismic damage. The RVS procedure can be undertaken rather quickly and inexpensively to develop a list of potentially hazardous buildings without incurring the high cost of a detailed seismic analysis of individual buildings. The final score values of the RVS evaluation range from 0 to 7, where higher values are indicators of good seismic resistance. If a building receives a low score on the basis of this RVS procedure, it should be a candidate for further detailed evaluation. On the basis of this detailed inspection, engineering analyses, and other detailed procedures, a final determination of the seismic adequacy and need for rehabilitation can be made. Hence, this technique can be used as an inexpensive and rapid screening criterion.

The RVS procedure is designed to be implemented without performing structural analysis. It utilizes a simple additive scoring system that requires the user to 1) identify the primary structural lateral-load-resisting system (i.e. building type); and 2) identify building attributes that modify the seismic performance expected of this lateral-load-resisting system (i.e., story height, vertical irregularity, plan irregularity, code consideration, etc.). Results are recorded on one of three Data Collection Forms depending on the seismicity of the region being surveyed (e.g. for high seismicity rapid visual inspection form see Figure 2.1). The Data Collection Form includes space for documenting building identification information, including its use and size, a photograph of the building, sketches, and documentation of pertinent data related to seismic performance, including the development of a numeric seismic hazard score. The scores are based on average expected ground shaking levels for the seismicity region as well as the seismic design and construction practices for that region. Buildings may be surveyed from the sidewalk (without the benefit of building entry), structural drawings, or structural calculations. Reliability and confidence in building attribute determination are increased, however, if the structural framing system can be verified either during interior inspection or on the basis of a review of construction documents.

Rapid Visual Screening of Buildings for Potential Seismic Hazards

FEMA-154 Data Collection Form

HIGH Seismicity

Address: ____________________

____________________ Zip ______

Other Identifiers ____________________

No. Stories ______ Year Built ______

Screener ______ Date ______

Total Floor Area (sq. ft.) ______

Building Name ______

Use ______

Scale:

PHOTOGRAPH

OCCUPANCY				SOIL TYPE	FALLING HAZARDS
Assembly Commercial Emer. Services	Govt. Historic Industrial	Office Residential School	Number of Persons 0 – 10 11 – 100 101-1000 1000+	A Hard Rock, B Avg. Rock, C Dense Soil, D Stiff Soil, E Soft Soil, F Poor Soil	☐ Unreinforced Chimneys ☐ Parapets ☐ Cladding ☐ Other: ______

BASIC SCORE, MODIFIERS, AND FINAL SCORE, *S*

BUILDING TYPE	W1	W2	S1 (MRF)	S2 (BR)	S3 (LM)	S4 (RC SW)	S5 (URM INF)	C1 (MRF)	C2 (SW)	C3 (URM INF)	PC1 (TU)	PC2	RM1 (FD)	RM2 (RD)	URM
Basic Score	4.4	3.8	2.8	3.0	3.2	2.8	2.0	2.5	2.8	1.6	2.6	2.4	2.8	2.8	1.8
Mid Rise (4 to 7 stories)	N/A	N/A	+0.2	+0.4	N/A	+0.4	+0.4	+0.4	+0.4	+0.2	N/A	+0.2	+0.4	+0.4	0.0
High Rise (> 7 stories)	N/A	N/A	+0.6	+0.8	N/A	+0.8	+0.8	+0.6	+0.8	+0.3	N/A	+0.4	N/A	+0.6	N/A
Vertical Irregularity	-2.5	-2.0	-1.0	-1.5	N/A	-1.0	-1.0	-1.5	-1.0	-1.0	N/A	-1.0	-1.0	-1.0	-1.0
Plan Irregularity	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5	-0.5
Pre-Code	0.0	-1.0	-1.0	-0.8	-0.6	-0.8	-0.2	-1.2	-1.0	-0.2	-0.8	-0.8	-1.0	-0.8	-0.2
Post-Benchmark	+2.4	+2.4	+1.4	+1.4	N/A	+1.6	N/A	+1.4	+2.4	N/A	+2.4	N/A	+2.8	+2.6	N/A
Soil Type C	0.0	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4	-0.4
Soil Type D	0.0	-0.8	-0.6	-0.6	-0.6	-0.6	-0.4	-0.6	-0.6	-0.4	-0.6	-0.6	-0.6	-0.6	-0.6
Soil Type E	0.0	-0.8	-1.2	-1.2	-1.0	-1.2	-0.8	-1.2	-0.8	-0.8	-0.4	-1.2	-0.4	-0.6	-0.8

FINAL SCORE, *S*

COMMENTS

Detailed Evaluation Required

YES NO

* = Estimated, subjective, or unreliable data
DNK = Do Not Know

BR = Braced frame
FD = Flexible diaphragm
LM = Light metal

MRF = Moment-resisting frame
RC = Reinforced concrete
RD = Rigid diaphragm

SW = Shear wall
TU = Tilt up
URM INF = Unreinforced masonry infill

Figure 2.1: FEMA 154 Rapid Visual Inspection Form.

2.4.2 Rapid Visual Screening of Buildings for Potential Seismic Hazard: Supporting Documentation (FEMA 155 report - Second Edition)

A companion volume to FEMA 154, Rapid Visual Screening of Buildings for Potential Seismic Hazards: Supporting Documentation (ATC 2002b) documents the technical basis for the RVS procedure described in the handbook, including the method for calculating the Basic Structural Scores and Score Modifiers.

2.4.3 Handbook for the Seismic Evaluation of Buildings (FEMA 310)

The current state of the art in North America follows the FEMA 310 (ASCE 1998) handbook for evaluating buildings for seismic vulnerability. This handbook has an initial evaluation requirements followed by a three-tier investigation.

The initial evaluation requirements are examination of existing documents, determination of the building type, testing requirements, site visit to determine existing conditions, establishing the desired level of performance, and identifying the level of seismicity. Once the evaluation requirements are established, Tier 1 – Screening Phase is undertaken.

The Tier 1 screening phase is carried out using three checklists that allow a rapid evaluation of the structural, non-structural and foundation/geologic hazard elements of the building and site conditions. At times, due to some incompatibility of the checklist and performance-based methodology with design provisions in current codes, a benchmark building provision is also provided. The purpose of Tier 1 evaluation is to screen out buildings that comply with the provisions in the FEMA 310 Handbook or identify potential deficiencies. If potential deficiencies are identified, results from the Tier 1 screening leads to the follow up evaluation, Tier 2 or with some buildings Tier 3.

The Tier 2 – Evaluation Phase is carried out for complete analysis of the building that addresses all of the deficiencies identified in Tier 1. This analysis is carried out using a linear static or a dynamic analysis. The purpose of this evaluation is to screen out buildings not requiring rehabilitation. Following Tier 2 evaluation, if the building is deficient, Tier 3 evaluation is invoked.

Tier 3 – Detailed Evaluation Phase is carried out using nonlinear analysis. Result from Tier 3 evaluation can be used to propose and implement different mitigation techniques.

In order to carry out the aforementioned three-tier evaluation, the following basic information has to be obtained: site visit, type of concrete building, level of performance, and evaluation requirements. Each of the required information will be further expounded below.

Site Visit

A site visit is conducted by the evaluating design professional to verify existing data or collect additional data, determine the general condition of the building, and verify or assess the site conditions. Relevant building data that should be determined through a site visit includes:

General building description - number of stories, year(s) of construction, and dimensions.

Structural system description - framing, lateral-force-resisting system(s), floor and roof diaphragm construction, basement, and foundation system.

Nonstructural element description - nonstructural elements that could interact with the structure and affect seismic performance.

Building type(s) - Categorize the building as one or more of the Common Building Types, if possible.

Performance Level - Note the performance level required in the evaluation.

Region of Seismicity - Identify the seismicity of the site to be used for the evaluation.

Soil type - Note the soil type.

Building Occupancy - The occupancy of the building should be noted.

Historic Significance - Identify any historic elements in the building. Any impacts or areas of the building affected by the evaluation should be noted.

A first assessment of the evaluation statements may indicate a need for more information about the building. The design professional may need to re-visit the site to do the following:

1. Verify existing data;
2. Develop other required data;
3. Verify the vertical and lateral-force resisting systems;

4. Check the condition of the building;
5. Look for special conditions and anomalies;
6. Address the evaluation statements again while in the field; and
7. Perform material tests, as necessary.

Concrete Building Type

The building being evaluated shall be classified as one or more of the building types listed in Table 2.3 based on the lateral force-resisting system(s). Two separate building types shall be used for buildings with different lateral-force-resisting systems in each of the two orthogonal directions. In this thesis, three concrete building types are considered; concrete moment frame (*C1*), concrete shear wall buildings (*C2*), and concrete frames with infill masonry shear walls (*C3*). The basic functional description of each building, as outlined in FEMA 310 is summarized in Table 2.3.

Table 2.3: Concrete Building Type[§].

Building Type 8: Concrete Moment Frames	
C1	These buildings consist of a frame assembly of cast-in-place concrete beams and columns. Floor and roof framing consists of cast-in-place concrete slabs, concrete beams, one-way joists, two-way waffle joists, or flat slabs. Lateral forces are resisted by concrete moment frames that develop their stiffness through monolithic beam-column connections. In older construction, or in regions of low seismicity, the moment frames may consist of the column strips of two-way flat slab systems. Modern frames in regions of high seismicity have joint reinforcing, closely spaced ties, and special detailing to provide ductile performance. This detailing is not present in older construction. Foundations consist of concrete spread footings or deep pile foundations.
Building Type 9: Concrete Shear Wall Buildings	
C2	These buildings have floor and roof framing that consists of cast-in-place concrete slabs, concrete beams, one-way joists, two-way waffle joists, or flat slabs. Floors are supported on concrete columns or bearing walls. Lateral forces are resisted by cast-in-place concrete shear walls. In older construction, shear walls are lightly reinforced, but often extend throughout the building. In more recent construction, shear walls occur in isolated locations and are more heavily reinforced with boundary elements and closely spaced ties to provide ductile performance. The diaphragms consist of concrete slabs and are stiff relative to the walls. Foundations consist of concrete spread footings or deep pile foundations.
Building Type 10: Concrete Frames with Infill Masonry Shear Walls	
C3	This is an older type of building construction that consists of a frame assembly of cast-in-place concrete beams and columns. The floors and roof consist of cast-in-place concrete slabs. Walls consist of infill panels constructed of solid clay brick, concrete block, or hollow clay tile masonry. The seismic performance of this type of construction depends on the interaction between the frame and infill panels. The combined behavior is more like a shear wall structure than a frame structure Solidly infilled masonry panels form diagonal compression struts between the intersections of the frame members. If the walls are offset from the frame and do not fully engage the frame members, the diagonal compression struts will not develop. The strength of the infill panel is limited by the shear capacity of the masonry bed joint or the compression capacity of the strut. The post-cracking strength is determined by an analysis of a moment frame that is partially restrained by the cracked infill. The shear strength of the concrete columns, after cracking of the infill, may limit the semiductile behavior of the system. The diaphragms consist of concrete floors and are stiff relative to the walls.

[§] Source *FEMA 310 (FEMA 1998): Table 2-2: Common Building Type*

Level of Performance

This thesis defines and uses performance levels in a manner consistent with FEMA 310. The process for defining the appropriate level of performance is the responsibility of the design professional or the authority having jurisdiction. Considerations in choosing an appropriate level of performance should include achieving basic safety, a cost-benefit analysis, the building occupancy type, economic constraints, etc. (ASCE 1998). Two performance levels for both structural and nonstructural components are defined: Life Safety (LS) and Immediate Occupancy (IO). For both performance levels, the seismic demand is based on Maximum Considered Earthquake (MCE) spectral response acceleration values.

In general, buildings classified as essential facilities should be evaluated to the IO performance level. These buildings are typically required for the post disaster management, e.g. hospital, fire rescue, etc. The 2003 NEHRP Recommended Provisions for Seismic Regulations for New Buildings categorizes the following buildings as essential facilities "...required for post-earthquake recovery":

- Fire or rescue and police stations,
- Hospitals or other medical facilities having surgery or emergency treatment facilities,
- Emergency preparedness centers including the equipment therein,
- Power generating stations or other utilities required as emergency back-up facilities for other facilities listed here,
- Emergency vehicle garages,
- Communication centers, and
- Buildings containing sufficient quantities of toxic or explosive substances deemed to be dangerous to the public if released.

Evaluation Requirements

Table 2.4 summarizes the Tier 1 basic and supplemental structural checklists for RC building types *C1*. Information from the Tier 1 evaluation statements are marked as compliant, non-compliant, or not applicable. Compliant statements identify performance modifiers that are acceptable, while non-compliant statements identify performance modifiers that require further investigation.

Table 2.4: Summary of basic and supplementary structural check list for Building Type *C1*.

C1: Concrete Moment Frames	Basic structural checklist	Supplementary structural checklist
Building system	Load path Adjacent buildings Mezzanines Weak story Soft story Geometry Vertical discontinuity Mass Torsion Deterioration of concrete Post-tensioning anchors	
Lateral force resisting system	Redundancy Interfering walls Shear stress check Axial stress check	Flat slab frames Prestressed frame elements Short captive columns No shear failures Strong column/weak beam Beam bars Column-bar splices Beam-bar splices Column-tie spacing Stirrup spacing Joint reinforcing Joint eccentricity Stirrup and hooks Deflection compatibility Flat slabs
Connections	Concrete columns	Lateral load at pile caps
Diaphragms		Diaphragm continuity Plan irregularity Diaphragm reinforcement at openings

2.5 Discussion

This chapter has presented a review of the different techniques used for quantifying building vulnerability. Results of the building vulnerability are correlated with a damage parameter. Damage ratio is defined as the ratio of the repair cost to replacement cost, or by the ratio of the number of buildings of a certain type having a specified amount of damage in the region under consideration to the total number of buildings of this type in that region.

The type of analysis technique selected and the level of details vary with the intended use of the information. For general assessment and resource allocation, empirical models are attractive alternative. Spatial SSH variability and uniqueness of each building necessitates carrying out building vulnerability assessment on each individual building. As a result, heuristic or mechanistic models are better alternatives.

Each reported method considers level of damage by correlating different building performance parameters, consequently, variation in the final output results. This problem is further compounded due to parameter uncertainty as well as model uncertainties. Moreover, there is considerable variation in the final result and this induces undue burden on the decision making process. Thus, any proposed method needs to account for the model and parameter uncertainty and implementation of decision making under uncertainty.

The aim of this thesis is to develop a reliable rapid visual screening methodology and tool. The proposed solution builds on the existing knowledge developed by FEMA 154/155 (Rapid Visual Screening of Buildings for Potential Seismic Hazards), and Tier 1 evaluation of FEMA 310 (Handbook for the Seismic Evaluation of Buildings). The proposed model handles demand and capacity heuristically and estimates the building vulnerability. Commonly encountered uncertainty in the walk down survey is handled using fuzzy based techniques. The next chapter outlines the proposed methodology.

Chapter 3
Seismic Risk Analysis of RC Buildings using Soft Computing Techniques - Tier 1 Model

"As complexity rises, precise statements lose meaning and meaningful statements lose precision."

Lotfi Zadeh

Information related to seismic hazard, and local soil conditions is often readily available from seismic hazard maps and through site inspection. Similarly, importance of a building can be established with relative ease based on its use and occupancy. Building vulnerability assessment involves consideration of building characteristics and conditions, and hence poses a challenge. Different building vulnerability assessment techniques have been proposed with different levels of complexity, ranging from a simple scoring to more complex nonlinear structural analyses. Ghobarah (2000) has summarized and discussed advantages and limitations of these methods. The level of sophistication required for such an assessment depends on many factors, including intended use of the available information and availability of funds for a subsequent retrofit strategy.

Building vulnerability to ground shaking and associated damage can be grouped into two categories (Saatcioglu *et al.* 2001); performance modifiers contributing to an increase in seismic demand (e.g., soft story frame, weak column-strong beam, vertical irregularities); and performance modifiers contributing to reduction in ductility and energy absorption capacity (e.g., construction quality, year of construction, structural degradation). Indeed, these performance modifiers are reported as the potential causes of failure in the recent earthquake reconnaissance reports summarized in Table 3.1.

Table 3.1: Summary of recent earthquakes and reported causes for building failures.

	Earthquake name	Date	M_w	Author(s)	Short column effect	Discontinuous columns	Soft /Weak story frame	Torsion	Construction design quality	Structural degradation / weakening	Problem of adjacency	Seismic design consideration (year of construction)	joints	Weak column/strong beam	Ground shaking	Liquefaction	Land slide	Rupture
					1	2	3	4	5	6	7	8	9	10	11	12	13	14
1	El Asnam, Algeria	10/10/1980	7.3	EERI 1983	●		●	●	●	●	●	●			□			
2	Mexico, Mexico	19/09/1985	8.1	Popov 1987			●	●	●	●	●	●		●	□			
3	Loma Prieta, US	17/10/1989	6.9	EERI 1989			●				●	●			□	■		
4	Luzon, Phillippines	16/07/1990	7.8	Hopkins 1993			●	◊	●						□	■		
5	Erzincan, Turkey	13/03/1992	6.7	Saatcioglu and Bruneau 1993	●		●	●	●			●		●	□			
6	Northridge, US	17/01/1994	6.7	EERI 1994	●			●		●		●	●◊		□			
7	Kobe, Japan	17/01/1995	6.9	EERI 1995; AIJ 1995; Whittaker et al. 1995; Nakashima et al. 1995				●◊	●	●	●	◊	●		□	■		■
8	Kocali, Turkey	17/08/1999	7.4	Saatcioglu et al. 2001	●		●	●	●			●	●	●	□			
9	Chi-Chi, Taiwan	21/09/1999	7.6	Naeim et al. 2000; Tsai et al. 2000; Su 2001	●	●	●		●			●	●	●	□			
10	Bhuj, India	26/01/2001	7.7	Humar et al. 2001	●		●	●	●				●		□	■		
11	Bingöl, Turkey	01/05/2003	6.4	Doğangün 2004	●		●		●		●	●	●	●	□		○	
12	Lefkada, Greek	14/08/2003	6.2	Karakostas et al. 2005	●		●	●	●			●			□			
13	Asia Earthquake	26/12/2004	9.3	CAEE 2005; Ghobarah et al. 2006	●		●		●			●	●	●	□			

● Moment resisting reinforced concrete (C1) and Moment resisting with masonry infill (C3) ◊ Shear wall buildings (C2)
□ Presence of ground shaking hazard ■ Presence of liquefaction ○ Presence of landslide ■ Presence of ground rupture

For a preliminary seismic risk assessment of reinforced concrete (RC) buildings, however, obtaining and incorporating exhaustive information reported in Table 3.1 is not feasible. Thus, some of these performance modifiers reported in Table 3.1 is used in the proposed Tier 1 model. The performance modifiers considered to quantify building vulnerability are in congruence with the FEMA 154 (ATC 2002) rapid visual screening (RVS) manual that include: i) *building type*, ii) *vertical irregularity*, iii) *plan irregularity*, iv) *year of construction,* and v) *construction quality*. Moreover, these performance modifiers can readily be obtained from a *walk down* survey and engineering drawings. The proposed Tier 1 model of seismic risk assessment of RC buildings is modeled through a heuristic base hierarchical structure.

3.1 Development of a hierarchical structure for seismic risk analysis RC buildings

The complex problem of seismic risk analysis can be grouped into a simple and manageable hierarchical structure. The hierarchical structure follows a logical order where the causal relationship for each supporting argument is further subdivided into specific contributors. Figure 3.1 illustrates a schematic of the proposed general framework to quantify seismic risk of RC buildings.

The first three main components include; i) site seismic hazard, ii) building vulnerability, and iii) building importance/exposure factors. These three main factors are labelled as *Main (Hierarchical) Modules*. The expansions of hierarchical modules are illustrated in Figure 3.2 following a format similar to that proposed by Miyasato *et al.* (1986). Once inputs on performance modifiers of the main modules are gathered, the corresponding indices are computed through the process of transformation and/ or fuzzification (where appropriate) and aggregation as part of the *Evaluation Module*. Finally, the final building damageability index

I^{BD}, building importance/exposure index I^{IE} and the resulting seismic risk index I^{R} are computed through the *Index Modules*.

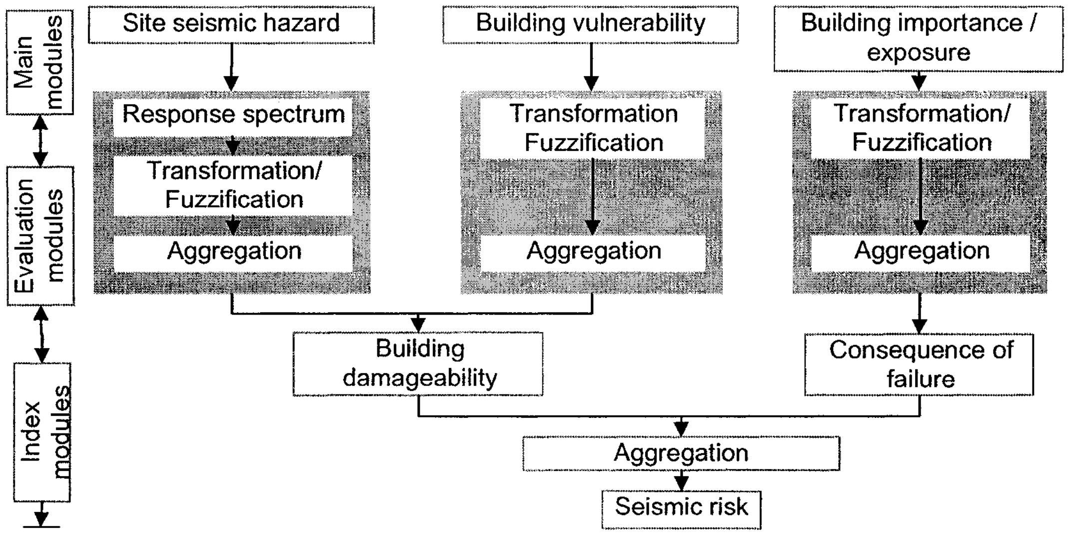

Figure 3.1: Seismic risk analysis of RC buildings.

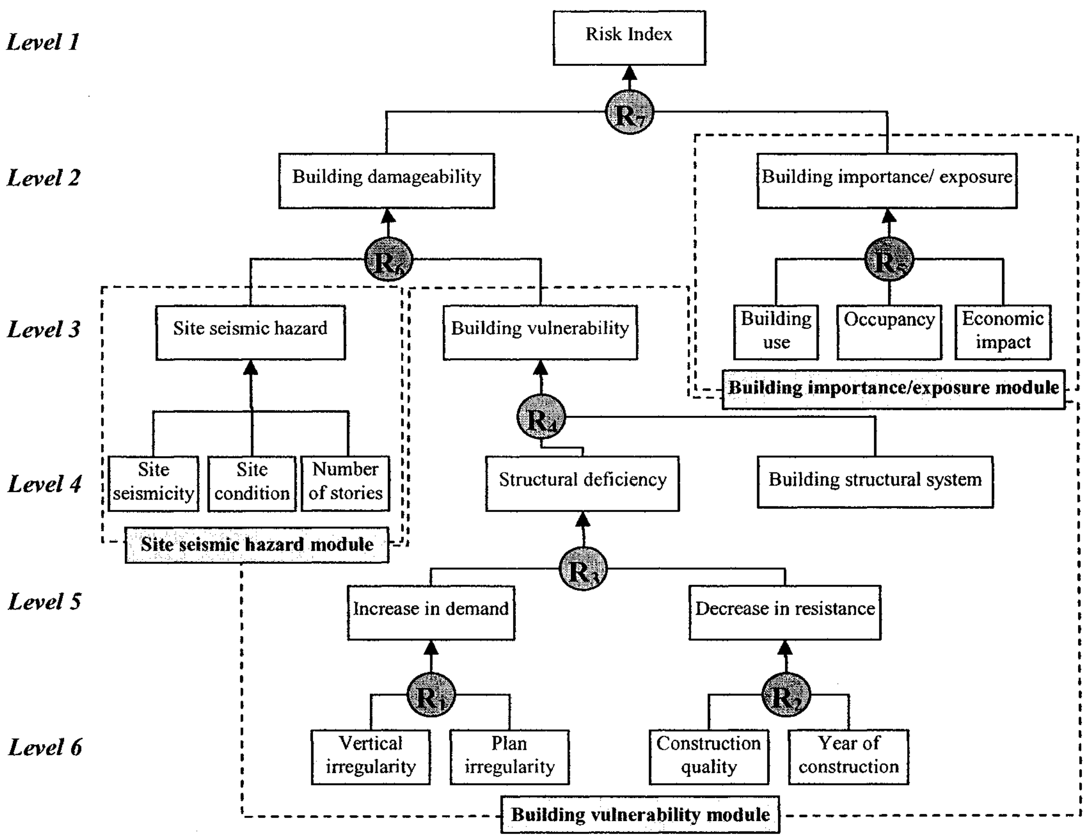

Figure 3.2: Hierarchical seismic risk analysis of RC buildings.

The process of quantifying the seismic risk index involves seven steps as indicated below:

Step 1. Collect all relevant performance modifiers of the hierarchical structure,

Step 2. Transform inputs of the performance modifiers into commensurable units through the process of transformation and/ or fuzzification,

Step 3. Aggregate the hazard performance modifiers to obtain the site seismic hazard index I^{SH},

Step 4. Aggregate the building performance modifiers to obtain the building vulnerability index I^{BV},

Step 5. Compute the building damageability index I^{BD},

Step 6. Aggregate importance / exposure performance modifiers to obtain the building importance/exposure index I^{IE},

Step 7. Compute the risk index I^{R}.

3.1.1 Site seismic hazard module

The site seismic hazard is usually computed by aggregating information about earthquake induced ground shaking, including the effects of ground conditions, while sometimes also incorporating the effects of landslides and liquefaction. Researchers often identify ground shaking as the dominant seismic hazard factor in inducing building damage (Bird and Bommer 2004), which is also reflected in Table 3.1. Landslide and liquefaction, on the other hand, are classified as consequences of ground shaking (EERI Committee on Seismic Risk 1989). Therefore, in the current preliminary screening analysis, landslides and liquefaction are not included in quantifying site seismic hazard index I^{SH}.

Another factor affecting site seismic hazard is condition of the site through which seismic waves travel. There is a substantial difference in the magnitude and frequency content of seismic waves between building sites that consist of soft soils and hard rock. This difference has to be reflected in quantifying I^{SH}. Seismic hazard is often expressed in terms of response spectra that describe maximum hazard quantities as a function of building period. Therefore, the fundamental period of a building T_1 should be incorporated as a parameter in defining seismic hazard. A simple indicator of structural period can be number of stories. Thus, quantification of I^{SH} includes site seismicity, site classification and number of stories. The interaction of these three parameters is a non-linear process, which is best described through response spectra. Figure 3.3 illustrates a schematic presentation of the proposed procedure of obtaining I^{SH} and a step-by-step procedure is outlined below.

Step (1): Compute the fundamental period (T_1) of the building using the number of stories

For a given building height h_n (in meter), T_1 of a concrete frame building or a shear wall building can be computed by using the expressions given below (Saatcioglu and Humar 2003):

$T_1 = 0.075(h_n)^{3/4}$ concrete frame building **(3.1a)**

$T_1 = 0.05(h_n)^{3/4}$ shear wall building **(3.1b)**

In a simple *walk down* survey, the building height h_n may not be readily available. Hence, for simplification, it can be assumed that each floor is 4m high, consequently h_n is computed by multiplying the number of stories N, by 4.

Step (2): Compute spectral acceleration $S_a(T_1)$

The $S_a(T_1)$ can be obtained from site specific design response spectrum. The response spectrum may be available through building codes or can be constructed using the existing representative earthquake records. Once the response spectrum is established, the spectral acceleration $S_a(T_1)$ is obtained by using the fundamental period of the structure T_1 determined in step (1) (Figure 3.3).

Step (3): Transform the spectral acceleration $S_a(T_1)$

The $S_a(T_1)$ is used as a surrogate indicator of site seismic hazard index I^{SH}. The transformation of $S_a(T_1)$ into a commensurable unit is context dependent. Fuzzification of $S_a(T_1)$ into five granules is illustrated in Figure 3.3. However, the $S_a(T_1)$ can also be transformed into any commensurable units without fuzzification.

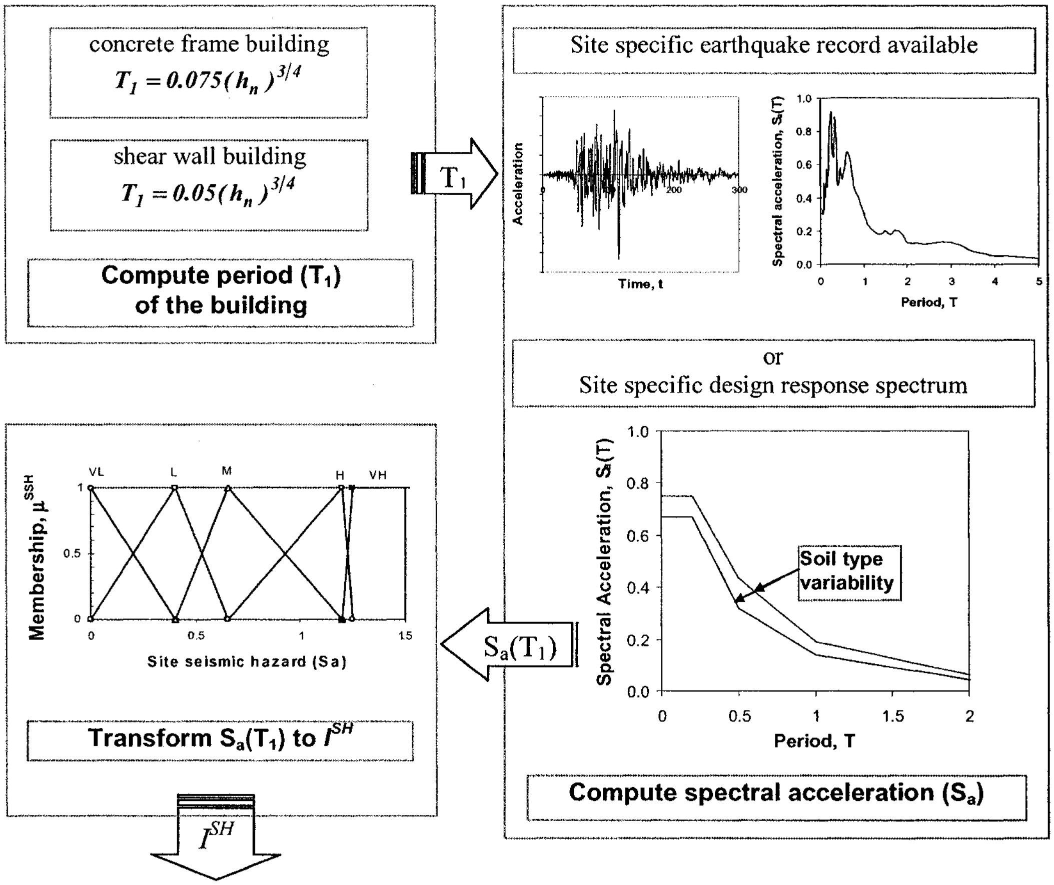

Figure 3.3: Quantification of site seismic hazard index I^{SH}.

3.1.2 Building vulnerability module

The *building vulnerability* (shown as *level 3* in Figure 3.2) is computed by integrating inherent system deficiencies in the structure. It consists of; i) building structural system, e.g. shear wall or moment resisting frame buildings, and ii) structural deficiency, e.g. vertical irregularity. The structural deficiency is sub divided into performance modifiers that contribute to an increase in seismic demand and decrease in structural resistance (capacity). Parameters that contribute to an increase in seismic demand are vertical irregularity and plan irregularity. On the other hand, parameters that contribute towards the decrease in resistance are construction quality and year of construction.

Aggregation through the hierarchical structure depicted in Figure 3.2 is used to compute the building vulnerability index I^{BV}. Each of the performance modifiers used in the computation of I^{BV} is further expounded below.

Building structural system

The type of lateral force resisting system used in a building plays a major role in terms of its resistance to seismic load. The three reinforced concrete (RC) building types considered in this thesis are moment resisting frames (*C1*), shear wall (*C2*) buildings and moment resisting frames with infill masonry walls (*C3*).

Shear walls of sufficient rigidity resist seismic forces almost entirely when used in buildings. Though the term "shear wall" is well accepted and used within the engineering community, their predominant mode of behaviour can be flexure when used in medium to high-rise buildings. They typically act as vertical cantilevers and provide lateral bracing to the entire system while receiving lateral forces from diaphragms and transmitting them to the foundation. The size and location are critical for seismic resistance. Earlier shear walls, used prior to the enactment of modern seismic design codes, are lightly reinforced flexible elements and tend to extend throughout the building height. In recent years, shear walls occur in isolated locations and are more heavily reinforced. Shear wall structures have been reported to behave well under moderate to strong earthquake excitations (Saatcioglu *et al.* 2001; Mitchell *et al.* 1995).

The moment-resistant frames resist lateral forces predominantly through the flexural action of columns and beams, which are connected by moment connections. The columns are responsible for overall strength and stability and hence are critical elements. Their strength relative to the adjoining beams plays an important role in seismic resistance in terms of dictating the sequence of plastification among members. Their inelastic deformability, as governed by concrete confinement and shear capacity, are critical. The detailing of beam-column connections is also important for seismic performance. Frames are susceptible to excessive lateral drifts and associated secondary (P-Δ) moments. Many older frame buildings include masonry infill panels. Though unreinforced masonry behaves in a brittle manner and is regarded as undesirable construction material for seismically active regions, may act as masonry infill panels shear walls in controlling deformations, and may save non-ductile concrete frames until their elastic limit of such panels is exceeded. There have been many cases of non-ductile frames that have survived strong earthquakes due to the participation of masonry infill walls, especially when the wall-to-floor area ratio is high.

Vertical irregularity (VI)

The earthquake induced inertia load is transferred from the floors, where most of the mass is concentrated to the foundation through the lateral load resisting system. A good design practice avoids discontinuities and/or abrupt changes in this load path so that localized stress concentrations are avoided. The *vertical irregularity* parameter reflects the presence of discontinuity and/or abrupt change in strength and stiffness along the building height. Vertical irregularities consist of vertical and reverse setbacks, soft stories, variation in column stiffness, discontinuity in shear walls, weak columns and strong beams, and any possible modifications introduced to the primary structural system (Arnold and Reitherman 1982). In a *walk down* survey, vertical irregularity is determined by *yes* or *present*, and *no* or *not present*.

Plan irregularity (PI)

The plan irregularity is used to determine building vulnerability to torsion and potential area of high stress concentration. A good design practice is to have a symmetrical plan layout. Lack of symmetry in strength and/or stiffness along the perimeter of building, re-entrant corners and the eccentricity of mass relative to centre of rigidity give rise to torsion (Arnold and Reitherman 1982). Forces due to torsion are accentuated more during dynamic response, leading to the overloading of some structural elements. In a *walk down* survey, the plan irregularity is determined by *yes* or *present*, and *no* or *not present*.

Construction quality (CQ)

Response of buildings to seismic events is determined by their seismic design, detailing and quality of materials and construction. Poor quality of construction and/ or materials used is detrimental to ensuring the protection intended in seismic design. Causes of poor construction quality include; construction errors, improper construction procedures, lack of reinforcement anchorage in beams and columns, additional anchorage problems associated with the use of plain bars, insufficient splice length, use of non-seismic hooks and improper seismic detailing, and poor concrete quality. The construction quality is qualitatively evaluated as *poor*, *average* or *good*.

Year of construction (YC)

During the initial site investigation, the original design drawings may not be readily available. The year of construction can be used to infer important information about the seismic design provisions employed and the resulting strength, stiffness, ductility and detailing in the structure. A good example of the effect of the year of construction is failures associated with lack of column confinement and in appropriate seismic detailing. Furthermore, the ductility of the construction material used may be measured by the year of construction (YC). The YC can be classified into three distinct states (NIBS 1999); low code $(YC \leq 1941)$, moderate code $(1941 < YC < 1975)$, and high code $(YC \geq 1975)$. It should be pointed out, however, that the threshold values are selected as representatives of North American practice. Therefore, the threshold values need to be adjusted for applications that involve specific geographic location and construction practice.

After determining the transformed values at each transition year, the following three linear transformation functions are fitted for the three distinct YC considered:

$$\begin{cases} 0.90 & YC \leq Low\ code \\ -0.01YC + 20.25 & Low\ code < YC < High\ code \\ -0.02YC + 39.9 & YC \geq High\ code \end{cases} \tag{3.2}$$

For the transformation function provided in Eq. (3.2), the minimum cut off is set at 0.10, i.e., for $YC \geq 1990$, the transformation value is taken as 0.10.

3.1.3 Building importance/exposure module

Building importance/exposure index I^{IE} is used to quantify the expected human loss, emergency response capacity and/or economic loss for a given earthquake. The expected loss can be direct physical damage (general building stock, emergency facility), casualties, economic loss, and social impact (FEMA-249 1994). While building codes primarily target life safety (casualties) and post disaster use (e.g., emergency facility), economical considerations also play an important role in assessing building importance. For example, the Northridge earthquake resulted in little casualty but the impact on economy was crippling (FEAM-249 1994, Elms 2004). Hence, the incorporation of economic impact in I^{IE} is paramount. At *level 3*, the building importance/exposure index I^{IE} is computed by integrating *building use*, *occupancy* and *economic impact*.

Building use

Building use is intended to infer post disaster utility of the building and associated potential damage that can be tolerated. The FEMA 450 (BSSC 2004) guideline for design of new buildings specifies three distinct groups. The first level is Immediate Occupancy (IO) Performance Level, where the level of damage tolerated is negligible and light. Buildings required for post-earthquake recovery, e.g. hospitals; fire rescue and police stations; communication centers; etc., fall under the IO category. The second level is Life Safety (LS), where a moderate damage state is tolerated. Typically, buildings used for public assembly; schools; structures with more than 5000 people capacity; etc., fall under LS. In this category, the structural failure is not imminent and life safety can be ensured. Any other structure that is not classified as IO or LS is assigned to low importance building category. Severe and complete collapses are not acceptable levels of performance.

Occupancy

The occupancy of a building is used to infer possible casualty in case of earthquake induced damage. The level of tolerance for causalities is a societal value judgment (Dey 2004, 2001; Flynn *et al.* 1999). The occupancy (described through number of person) is categorized into 4 discrete groups; 0-10, 11-100, 100-1000, and more than 1000.

Economic impact

The Northridge earthquake has demonstrated once again the importance of economic consequence of failures in risk analysis (Elms 2004). The direct economic impact of a building can be categorized as market losses and non-market losses (FEMA-249 1994), or as loss of use (down-time) and direct repair cost (Porter *et al.* 2001). Detailed discussion and derivation of these factors is beyond the scope of this thesis. However, for the purpose of I^{IE} computation, the economic impact of building is categorized into *negligible*, *average*, and *significant*.

3.1.4 Building damageability index I^{BD}

As previously discussed, the I^{BD} value is computed by integrating seismic hazard index I^{SH} and building vulnerability index I^{BV}. A building located in a region with high I^{SH} may incur little or no damage if designed properly following building codes that are based on contemporary design concepts. A building located in a region with low I^{SH} and designed with older design codes may again not be damaged even if it lacks adherence to proper seismic design practices. Building damageability assessment is intricately dependent on I^{SH} and I^{BV}. The I^{BD} has to be calibrated using existing field or analytical data (Dolce *et al.* 2006).

Various building damage classifications are reported. In this thesis, the ATC-13 (ATC 1985) damage States are adopted with a slight modification. The ATC-13 damage states are classified into 7 distinct states, which are associated with different damage factors. However, in the proposed approach, the damage states none and slight are combined as *none-slight* (D_{N-S}), and major and destroyed are combined as *major-destroyed* (D_{M-D}). Consequently, five discrete damage states are defined: *none-slight* (D_{N-S}), *light* (D_L), *moderate* (D_M), *heavy* (D_H) and *major-destroyed* (D_{M-D}). This is illustrated in Table 3.2.

Table 3.2: Building damage state classifications.

Damage state	Damage Factor Range (%)	Central Damage Factor (%)	Description	Damage state§
None	0	0	No damage.	D_{N-S}
Slight	0-1	0.5	Limited localized minor not requiring repair.	
Light	1-10	5	Significant localized damage of some components generally not requiring repair.	D_L
Moderate	10-30	20	Significant localized damage of many components warranting repair.	D_M
Heavy	30-60	45	Extensive damage requiring major repairs.	D_H
Major	60-100	80	Major widespread damage that may result in the facility being razed, demolished, or repaired.	D_{M-D}
Destroyed	100	100	Total destruction of the majority of the facility.	

§Damage state grouping adopted for North American data.

3.1.5 Risk index I^R

In the proposed hierarchical structure, the risk index I^R is quantified by aggregating the building damageability index I^{BD} and importance and exposure index I^{IE}. Indeed, similar to the quantification of I^{BD}, it can be argued that the quantification of risk is intricately associated with potential for building damage, and if there is any damage, with the consequence of failure. The final risk index I^R value is in a unit interval $I^R \in [0, 1]$.

For decision making purpose, however, the risk index I^R value can be converted into a linguistic constant. In this thesis, four linguistic constants are considered for final decision making purpose: *Negligible, Marginal, Critical* and *Catastrophic*. Different I^R "cut off" values need to be established for this classification. Besides establishing these cut off values, uncertainty in the estimation of the building damageability has a major impact on the reliability of the estimated risk index value. The uncertainty in the final building damageability index is quantified by computing the entropy (or degree of fuzziness) that will be discussed in Section 4.1.6. The entropy is quantified in a unit interval, entropy $\in$ [0, 1]. Thus, the risk index value I^R coupled with the entropy will be used for linguistic risk classification. Assigning equal weights to the first three intervals, the I^R values are categorized into [0, 0.2]; [0.2, 0.4], [0.4, 0.6] and [0.6, 1.0]; whereas the entropy is clustered into three distinct states: [0, 0.3], [0.3, 0.6], and [0.6, 1]. Specification of the threshold values are subject to the decision maker's risk tolerance, and need to be calibrated and a general consensus established. The preliminary suggested I^R "cut off" values coupled with entropy are summarized in Table 3.3.

Table 3.3: Risk index thresholds and entropy.

I^R	Entropy		
	[0, 0.3]	[0.3, 0.6]	[0.6, 1]
[0, 0.2]	Negligible	Marginal	Critical
[0.2, 0.4]	Marginal	Marginal	Catastrophic
[0.4, 0.6]	Critical	Critical	Catastrophic
[0.6, 1.0]	Catastrophic	Catastrophic	Catastrophic

3.2 Soft computing techniques

Uncertainty is an unavoidable component of decision-making process. The number of definitions of uncertainty within artificial intelligence and engineering community is large, and often, conflicting taxonomies are provided (e.g. Klir and Yuan 1995; Jousselme *et al.* 2003). Klir and Yuan (1995) taxonomy identifies uncertainties as fuzziness (lack of sharp distinction, vagueness), non-specificity (two or more alternatives are left unspecified) and discord or conflict (disagreement in choosing among several alternatives). Maupin and

Jousselme (2004) have further sub-classified vagueness uncertainty into three categories, namely, ontological, linguistic and epistemic vagueness. Ontological vagueness deals with the physical nature of objects. Linguistic vagueness arises due to the limitation of the natural languages. Epistemic vagueness is due to the limitation of sensorial apparatus, lack of knowledge or computational limitations.

In any *walk down* survey, the performance modifiers are solicited in linguistic terms (*strongly compliant*, *compliant*, *non-compliant*, *strongly non-compliant*, etc.). This evaluation process is prone to subjective/ qualitative judgments (Hadipriono and Ross 1991), which is dominated by vagueness uncertainty. The vagueness uncertainty can be handled through the fuzzy set theory, which was first proposed by Zadeh (1965). A fuzzy set describes the relationship between an uncertain quantity x and a membership function μ_x, which ranges between 0 and 1. A fuzzy set is an extension of the traditional set theory (in which x is either a member of set A or not) in that x can be a member of set A with a certain degree of membership μ_x. Membership function essentially embodies the knowledge base in the fuzzy system.

The uncertainty in the quantification of risk index can be handled through soft computing techniques. The soft computing is a conglomerate of computing techniques that include fuzzy-based approaches, neuro-computing, genetic computing, probabilistic reasoning, genetic algorithms, chaotic systems, belief networks, and learning theory (Zadeh 1997). The soft computing techniques effectively explore the relationship among independent and dependent variables without any assumptions about the relationship (e.g., a linear relationship) between the various variables.

Various soft computing models are explored for seismic risk analysis of RC buildings. Aggregation over the proposed Tier 1 model can be handled using weighted based models, for example a weighted arithmetic mean (WAM), ordered weighted averaging (OWA), and fuzzy synthetic evaluation (FSE). The inferencing methods of WAM and FSE use linear equations and may lack flexibility to deal with complex interactions. On the other hand, the OWA operation is nonlinear, however, it does not consider interactions among performance modifiers. The fuzzy rule base (FRB) provides an option to elucidate non-linearity through rule-sets to deal with complex interactions. Fuzzy-based approach provides a completely new way of modeling complex systems like buildings. The FRB modeling doesn't use any weighting system rather the knowledge base and importance factor are embedded in the rule-set. It can deal with the nature of uncertainty in system and human error. Traditionally, uncertainties in earthquake engineering were handled using probabilistic methods, which necessitates acquiring large historical data (Dong *et al.* 1987). However, besides the challenge of acquiring large historical data, seismic application must deal with uncertainties resulting from *ignorance*, *imprecision*, *vagueness*, and *subjective judgment*.

In this chapter, the WAM and OWA are discussed, and the utility of the FSE in Tier 1 model is illustrated with two case studies. In Chapter 4 and Chapter 5, the FRB modeling is applied to the Tier 1 and Tier 2 models, respectively. In Table 3.4, the different aggregators, and parameters considered are summarized.

Table 3.4: Proposed models for seismic risk analysis of RC building

Characteristics	Inferencing methods			
	WAM	OWA	FSE	FRB
Transformation to a commensurable units	•	•	•	•
Modeling of nonlinearity		•		•
Uncertainty		§	•	•
Requirement of weights	•	•	•	§§
Inferencing Aggregation	•	•	•	•§§§

§ Uncertainty encountered are within the subjectivity of the OWA weight generation
§§ Weights are embedded within the FRB generation
§§§ Mamdani or Sugeno type inferencing

3.3 Weighted arithmetic mean (WAM)

The weighted arithmetic mean (WAM) uses the importance weights of contributory performance modifiers to make inference, which is a three step process; i) transformation of the performance modifiers into commensurable units; ii) obtaining the relative importance of each performance modifier; and iii) aggregation using weighted arithmetic mean. Some of the performance modifiers are defined linguistically, e.g., *low*, *moderate*, *high* and some have numeric values, e.g., year of construction. The WAM has been used in seismic risk analysis (Davidson and Shah 1997; Sucuoğlu and Yazgan 2003; Casciati and Farvelli 1991).

Since units of the performance modifiers are non commensurable, transformation functions are used to translate the actual values into an interval of [0, 1], where "0" corresponds to the *best* value and "1" corresponds to the *worst* case. Transformed values of the building vulnerability performance modifiers are summarized in Table 3.5, where for example, the linguistic evaluation for vertical irregularity described as *yes* and *no* are transformed into 0.8 and 0.1, respectively. Similar transformation value of the other performance modifiers can be obtained from Table 3.5. Where sufficient earthquake damage data is available, the transformation values can be calibrated by minimizing the root mean square error between the observed and estimated damage states.

Table 3.5: Transformation of performance modifiers for building vulnerability assessment.

Performance modifiers	Transformation
Vertical irregularity	{Yes, No} → {0.80, 0.10}
Plan irregularity	{Yes, No} → {0.80, 0.20}
Construction quality	{Good, Average, Poor} → {0.01, 0.70, 0.99}
Year of construction	Equation (3.5)
Structural system	{*C1*, *C2*, *C3*} → {0.70, 0.25, 0.35}

Different methods are proposed for generating relative importance weights. The relative importance of each performance modifiers can be estimated through analytic hierarchy process (AHP) (Saaty 1980). AHP estimates the relative importance of each performance modifiers in a group using pairwise comparisons. The levels of the pairwise comparisons range from 1 to 9 (Table 3.6), where "1" represents that two performance modifiers are *equally important*, while the other extreme "9" represents that one performance modifier is absolutely *more important* than the other.

Table 3.6: AHP pairwise comparison table (Saaty 1980).

Relative importance	Definition	Explanation
1	Equal importance	Two activities contribute equally to the objective
3	Weak importance	Experience and judgment slightly favour one activity over another
5	Essential or strong importance	Experience and judgment strongly favour one activity over another
7	Demonstrated importance	One activity is strongly favoured and demonstrated in practice
9	Extreme importance	The evidence favour one activity over another is of highest possible order of affirmation
2, 4, 6, 8	Intermediate values between two adjacent judgments	When compromise is needed

A judgement matrix J can be established, where each performance modifier, j_{mn}, in the upper triangular matrix, expresses the importance intensity of a performance modifiers m with respect to n. Each element in the lower triangle of the matrix is the reciprocal of an element in the upper triangle, i.e., $j_{nm} = 1/j_{mn}$. This is shown in Eq. (3.3), for computing the relative importance of two performance modifiers *vertical irregularity* (VI) and *plan irregularity* (PI) over increase in demand (ID). For example, if the relative importance intensity of VI over PI is assigned a value of 2.25, then the J is established as:

$$J = \begin{array}{c} \\ VI \\ PI \end{array} \begin{array}{c} \begin{array}{cc} VI & PI \end{array} \\ \begin{bmatrix} 1 & 2.25 \\ 1/2.25 & 1 \end{bmatrix} \end{array} \tag{3.3}$$

Once the judgement matrix is populated, the eigenvalues (λ) and eigenvectors ($w_1, w_2, \ldots, w_n$) are obtained by eigenvalue formulation, $(J-\lambda I)W=0$. Alternatively, a matrix J' could be determined by taking the geometric mean of each row and then the AHP weights vector W could be derived by the normalization of matrix J':

$$j' = \begin{bmatrix} 1.50 \\ 0.67 \end{bmatrix} \quad \Rightarrow \quad W = \begin{bmatrix} 0.69 \\ 0.31 \end{bmatrix} \tag{3.4}$$

This could be interpreted as the relative importance associated with VI is 0.69 and PI is 0.31. The AHP weights for the building vulnerability module are summarized in Table 3.7.

Table 3.7: AHP weights for building vulnerability module.

Basic parameter	AHP weights
Vertical irregularity	0.69
Plan irregularity	0.31
Construction quality	0.69
Year of construction	0.31
Increase in demand	0.25
Decrease in resistance	0.75
Structural system	0.31
Structural deficiency	0. 69
Site seismic hazard	0.10[§]
Building vulnerability	0.90
[§] For the Northridge earthquake database,	0.25
the weights used are	0.75

A WAM operator of dimension n is a mapping of $R^n \rightarrow R$ (where $R \in$ [0, 1]), which has an associated n number of performance modifiers $W=(w_1,w_2,\ldots,w_n)^T$, where $w_j \in$ [0, 1] and $\sum_{j=1}^{n} w_j = 1$. Hence, for a given n performance modifiers vector ($a_1, a_2, \ldots, a_n$), the WAM aggregation is performed as follows:

$$WAM = \sum_{j=1}^{n} a_j w_j \tag{3.5}$$

At each level of the hierarchy, result of the WAM is used as an input to the higher hierarchy.

3.4 Ordered weighted averaging (OWA)

Most multi-criteria decision analysis problems require neither strict "*anding*" (minimum) nor strict "*oring*" of the s-norm (maximum). To generalize this idea, Yager (1988) introduced a new family of aggregation techniques called the ordered weighted average (OWA) operators, which form general mean type aggregators. The OWA operator provides flexibility to utilize the range of "anding" or "oring" to include the attitude of a decision maker in the aggregation process. There are an increasing numbers of reported applications of OWA operators in the disciplines of civil engineering (Tesfamariam and Sadiq 2008; Sadiq and Tesfamariam 2007a, b; Makropoulos and Butler 2006, 2005, 2004; Smith 2006, 2002). There is also reported application of OWA in earthquake engineering. Rashed and Weeks (2003) applied it in assessing vulnerability to earthquake hazards, whereas Tesfamariam and Saatcioglu (2007) applied it in seismic risk analysis of RC buildings.

An OWA operator of dimension n is a mapping of $R^n \rightarrow R$ (where $R \in [0, 1]$), which has an associated n number of performance modifiers $W=(w_1,w_2,\ldots,w_n)^T$, where $w_j \in [0, 1]$ and $\sum_{j=1}^{n} w_j = 1$. Hence, for a given n performance modifiers vector $(a_1, a_2, \ldots, a_n)$, the OWA aggregation is performed as follows:

$$OWA(a_1, a_2, \ldots, a_n) = \sum_{j=1}^{n} w_j b_j \qquad \textbf{(3.6)}$$

where b_j is the j^{th} largest element in the vector $(a_1, a_2, \ldots, a_n)$, and $b_1 \geq b_2 \geq \ldots \geq b_n$. Therefore, the weights w_j of OWA are not associated with any particular value a_j, rather they are associated with the ordinal position of b_j. The linear form of OWA equation aggregates multiple performance modifiers vector $(a_1, a_2, \ldots, a_n)$ and provides a nonlinear solution (Yager and Filev 1999).

One of the major challenges in OWA method is to generate weights. Since the introduction of OWA operators by Yager (1988), different methods of OWA weight generation and extension of OWA operators have been proposed in the literature. Sadiq and Tesfamariam (2007) and Xu (2005) have discussed the current state of the art for OWA weight generation. In a case where earthquake damage data are available, however, the OWA weights can be generated by training on data (Filev and Yager 1998; Beliakov 2003).

3.5 Fuzzy synthetic evaluation (FSE)

A *synthetic* evaluation of a given object in terms of multiple factors (e.g. ground shaking, soil type, number of stories) that contribute to some feature of the object (e.g. site seismic hazard) may be regarded as a system with multiple inputs and single output (Wang *et al.* 1998). Similarly, aggregation of multiple inputs into a single output can also be viewed as the whole is synthesis of the parts (Ross 2004). There is increasing use of FSE for civil engineering applications, e.g. pipe deterioration modeling (Rajani *et al.* 2006); drilling waste discharges (Sadiq *et al.* 2004); damage evaluation of bridges (Liang *et al.* 2001); and reservoir water quality (Lu *et al.* 1999).

The seismic risk analysis of RC buildings as illustrated in Figure 3.2 can be carried out through the FSE method. The FSE process entails four steps, i) quantification and fuzzification of the basic risk item inputs obtained from a walk down survey, ii) weight generation, iii) aggregation, and iv) defuzzification.

3.5.1 Step 1: Membership functions – fuzzification

The range of transformed values of each performance modifiers is known as a universe of discourse; say from 0 to 1, [0, 1]. The transformed values can be grouped into linguistic values such as *very low* (*VL*), *low* (*L*), *medium* (*M*), *high* (*H*), and *very high* (*VH*). The process of assigning these linguistic values can be viewed as a form of data compression, which is known as granulation (Zadeh 1994). The fuzzification process assigns for each transformed values corresponding memberships with respect to the specified granularities.

Different methods of assigning membership values or functions are available (Ross 2004; Medasani *et al.* 1998), albeit having different nomenclatures. Medasani *et al.* (1998), among others, have summarized different methods of assigning a membership functions based on: i) perception, ii) heuristic method, iii) histogram based methods, iv) transformation of probability distribution into possibility distribution, v) fuzzy nearest neighbour technique, vi) neural network based methods and vii) methods based on clustering. Each of these methods has their strength and shortcomings as outlined in Ross (2004) and Medasani *et al.* (1998). In general, Medasani *et al.* (1998) have highlighted that there is no single best method to generate membership functions; rather the choice is context dependent. For the FSE, the histogram based methods of generating membership values are explored, and in Chapter 4 and Chapter 5 heuristic based method of assigning membership function is illustrated.

Histogram based method

Histograms provide salient information on the relationship between input and output values (Medasani *et al.* 1998). In a case where information of past earthquake damage are available, histograms can be used to map membership values and consequently the granularity is associated with the level of damage states. ATC-13 (1985) has highlighted that the discernable level of damage states is constrained by the "limited precision feasible in making judgmental estimates of damage by the earthquake engineering specialists." Generally, 5 to 9 damage states are reported in the literature. Thus, in this section, five granules, in congruence with Souflis and Grivas (1986), are used to classify the risk levels: *very low* (*VL*), *low* (*L*), *medium* (*M*), *high* (*H*), and *very high* (*VH*). After fuzzification, each input value of the basic attribute is expressed by an array of five-tuple fuzzy set $(\mu_{VL}, \mu_L, \mu_M, \mu_H, \mu_{VH})$, where μ_i refers to the membership to each fuzzy subsets and the subscript describes the corresponding risk level.

For example, histogram extracted from the damage database of Turkish and Northridge earthquakes are plotted in Figure 3.4 and Figure 3.5, respectively. For Figure 3.4, the vertical irregularity, plan irregularity, year of construction and building type are extracted from the 1999 Duzec Earthquake[1], whereas the construction quality is obtained from the 2003 Bingöl Earthquake[2]. For Figure 3.5, the histogram is extracted from the ATC-38 report (ATC 2001). The granulation for the year of construction follows the Turkish and North American building design codes.

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[1] SERU, Middle East Technical University, Ankara, Turkey; Archival Material from Duzce Database located at website http://www.seru.metu.edu.tr.

[2] SERU, Middle East Technical University, Ankara, Turkey; Archival Material from Bingöl Database located at website http://www.seru.metu.edu.tr.

The concept of generating the membership function is illustrated below. For example, from Figure 3.4, for construction quality of *poor* and *good*, the frequency distributions are (0, 2, 4, 8, 2) and (3, 3, 0, 0, 0), respectively. Normalizing the frequency distribution to a unity provides corresponding fuzzification for *poor* and *good* construction quality (0, 0.125, 0.25, 0.5, 0.125) and (0.5, 0.5, 0, 0, 0), respectively. The normalized values will be used as the corresponding five-tuple membership values, $(\mu_{VL}, \mu_L, \mu_M, \mu_H, \mu_{VH})$. Membership values generated from Figure 3.4 and Figure 3.5 are summarized in Table 3.8. It is interesting to note that most of these membership values follow our intuitive understanding of the system. Counter intuitive fuzzification results are due to the fact that the Histogram based method does not consider interaction between different performance modifiers. Similarly, for the Turkish data year of construction membership values show counter intuitive results. This is due to the fact that although the Turkish building code has improved; the code enforcement and construction practice has not caught up with it. Nevertheless, membership values obtained through this technique provide deeper insight into the system.

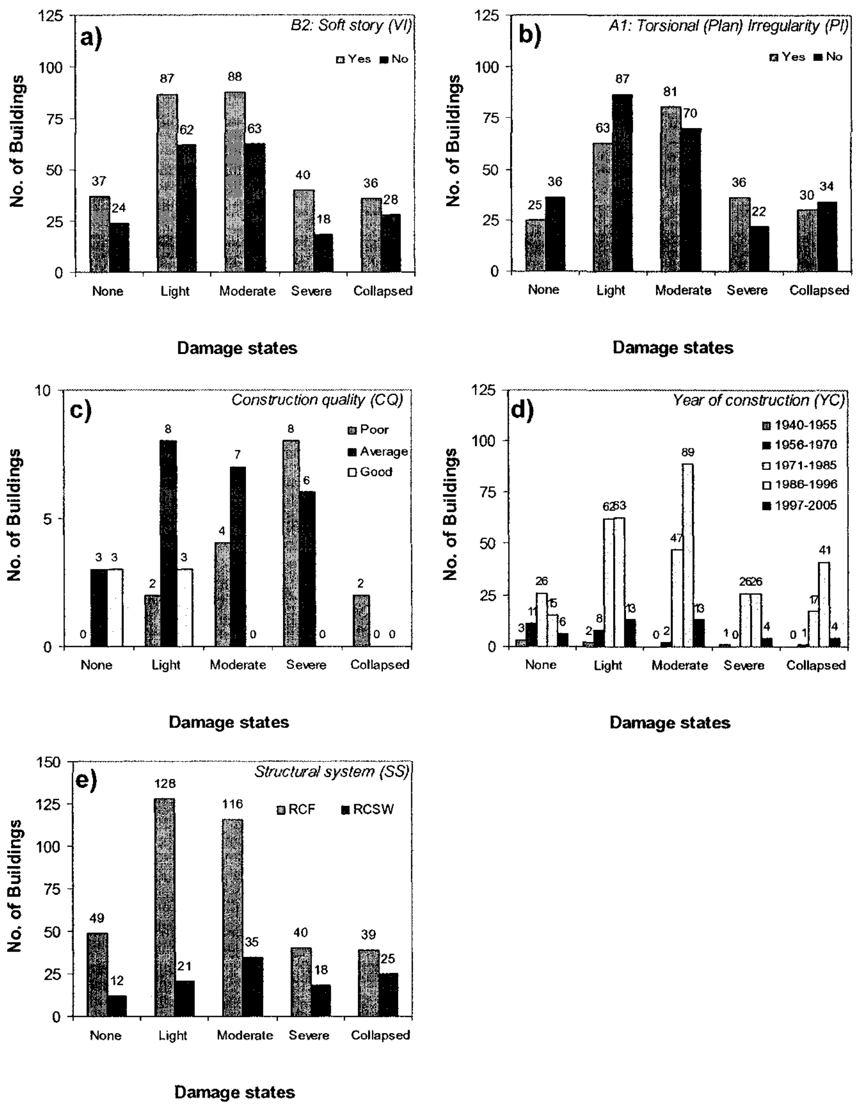

Figure 3.4: Performance modifiers and mapping over different damages states for RC buildings (Turkish Earthquake) a) vertical irregularity b) plan irregularity c) construction quality d) year of construction e) structural system.

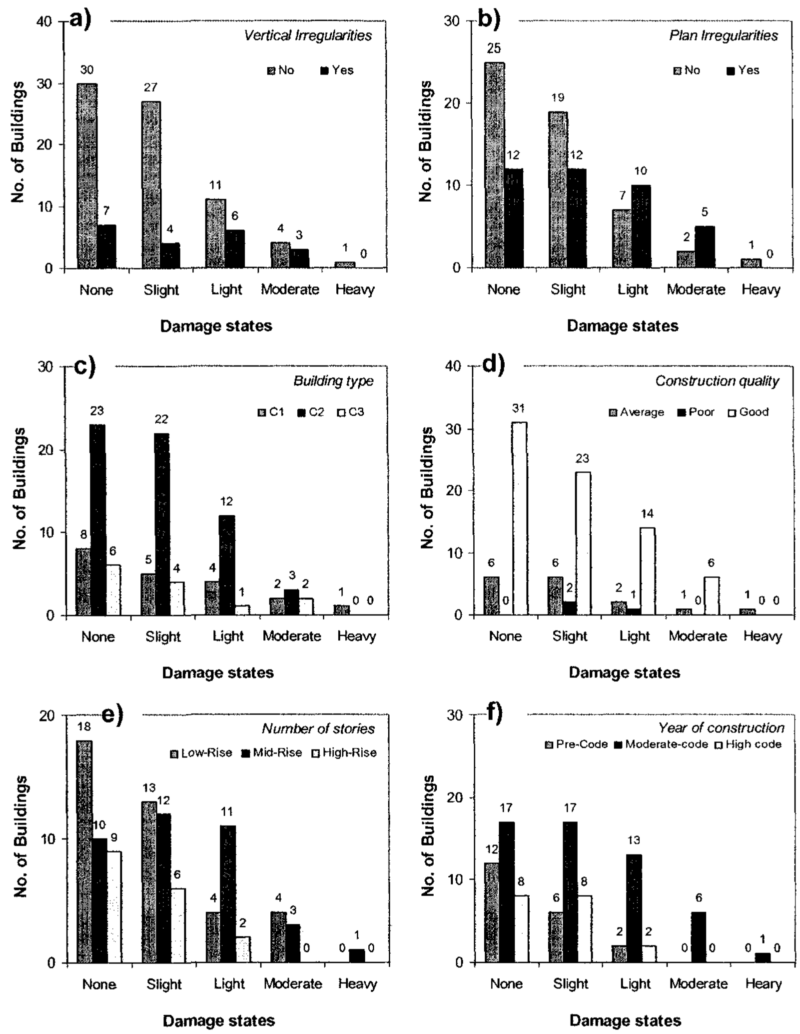

Figure 3.5: Performance modifiers and mapping over different damages states for RC buildings (Northridge Earthquake) a) vertical irregularity b) plan irregularity c) building type d) construction quality e) number of stories f) year of construction

Table 3.8: Transformation of linguistic inputs (Histogram based method)

Basic parameter	Linguistic parameter	Fuzzification $(\mu_{VL}, \mu_L, \mu_M, \mu_H, \mu_{VH})$ Turkish Earthquake	1994 Northridge Earthquake
Vertical irregularity	Yes	(0.18, 0.30, 0.25, 0.21, 0.05)	(0.35, 0.20, 0.30, 0.15, 0)
	No	(0.12, 0.31, 0.32, 0.11, 0.14)	(0.41, 0.37, 0.15, 0.05, 0.01)
Plan irregularity	Yes	(0.11, 0.27, 0.34,0.15, 0.13)	(0.31, 0.31, 0.26, 0.13, 0)
	No	(0.14, 0.35,.0.28, 0.09, 0.14)	(0.46, 0.35, 0.13 0.04, 0.02)
Construction quality	Poor	(0, 0.13, 0.25, 0.50, 0.13)	(0, 0.67, 0.33, 0, 0)
	Average	(0.13, 0.33, 0.29, 0.25, 0)	(0.38, 0.38, 0.13, 0.06, 0.06)
	Good	(0.50, 0.50, 0, 0, 0)	(0.42, 0.31, 0.19, 0.08, 0)
Year of construction (Turkish building code)	1940-1955	(0.50, 0.33 , 0, 0.17, 0)	
	1956-1970	(0.50, 0.36, 0.09, 0, 0.05)	
	1971-1985	(0.15, 0.35, 0.26, 0.15, 0.10)	
	1986-1996	(0.06, 0.27, 0.38, 0.11, 0.18)	
	1997-2005	(0.15, 0.33, 0.33, 0.10, 0.10)	
Year of construction (North American building code)	Low code		(0.88, 0.06, 0.06, 0, 0)
	Moderate code		(0.62, 0.24, 0.12, 0.02, 0)
	High code		(0.86, 0.14, 0, 0, 0)
Building type	RCF/ *C1*	(0.13, 0.34, 0.31, 0.11, 0.10)	(0.40, 0.25, 0.20, 0.10, 0.05)
	RCSW/ *C2*	(0.11, 0.19, 0.32, 0.16, 0.23)	(0.38, 0.37, 0.20, 0.05, 0)
	C3	N/A	(0.46, 0.31, 0.08, 0.15, 0)

3.5.2 Step 2: Analytic Hierarchy Process (AHP) - weighting scheme

See the AHP weights provided in Table 3.7 and procedure outlined in Section 3.3.

3.5.3 Step 3: Aggregation

For the FSE, the aggregation is carried out through weighted arithmetic mean (WAM) method. Initially, as illustrated earlier, each performance modifier i (i = 1,...,n) is fuzzified: $\mu^i_{VL}, \mu^i_L, \mu^i_M, \mu^i_H, \mu^i_{VH}$. Thus, for n performance modifiers to be aggregated, there will be n five-tuple fuzzy numbers grouped as a fuzzy judgement matrix. The WAM computation entails matrix multiplication of the AHP weights ($w_1, w_2, \ldots, w_n$) and the fuzzy judgement matrix:

$$[\mu_{VL}, \mu_L, \mu_M, \mu_H, \mu_{VH}] = [w_1, w_2, \ldots, w_n] \begin{bmatrix} \mu^1_{VL} & \mu^1_L & \mu^1_M & \mu^1_H & \mu^1_{VH} \\ \vdots & & \ddots & & \vdots \\ \mu^n_{VL} & & \cdots & & \mu^n_{VH} \end{bmatrix} \quad \textbf{(3.6)}$$

3.5.4 Step 4: Defuzzification

Defuzzification is a process of converting fuzzy output into a crisp number. Various defuzzification techniques are reported, such as center of gravity, center of area and mean of maximum methods. In this thesis, at each level of the hierarchy, the WAM method (Sadiq *et al.* 2004; Liou and Lo 2005) is used for defuzzification to obtain an index I' of each FSE output:

$$I' = \sum_{i=1}^{n} q_i * \mu_{R,i} \tag{3.7}$$

where q_i is quality-ordered weights, $q_i \in [0, 10]$, and the q_i values assigned for the five-tuples fuzzy sets are: $q_{VL} = 0$, $q_L = 2.5$, $q_M = 5$, $q_H = 7.5$ and $q_{VH} = 10$. The q_i values correspond with n-tuples fuzzy sets associated with the granularity of the specific output rule base. The q_i values are generated by assuming equal importance between each interval, and as will be illustrated in the case study, they are a good approximation.

3.5.5 Illustrative example

The computation of building damageability is illustrated through the information obtained for *building ID* BNG-10-3-10 (Table 3.9). From Table 3.9, the response to the performance modifiers is:

Building type = RCF;

Vertical irregularity = *Yes*;

Plan irregularity = *Yes*;

Construction quality = *Poor*;

Year of construction = *Unknown*.

The approach used to quantify the *unknown* case it to consider worst and best case scenarios. This is a case for decision making under *ignorance*. For worst and best case scenario, YC can be considered to be 1965 and 1990, respectively. In the final analysis, an interval risk index value is obtained. A step-by-step procedure of the FSE outlined below.

Step 1: Fuzzification (Table 3.8)

Building type: (0.13, 0.34, 0.31, 0.11, 0.10);

Vertical irregularity: (0.18, 0.30, 0.25, 0.21, 0.05);

Plan irregularity: (0.11, 0.27, 0.34, 0.15, 0.13);

Construction quality: (0, 0.13, 0.25, 0.50, 0.13);

Year of construction:

Worst case (YC = 1965): (0.50, 0.36, 0.09, 0, 0.05)

Best case (YC = 1990): (0.06, 0.27, 0.38, 0.11, 0.18)

Step 2: AHP weights (see Table 3.7**)**

Step 3: Aggregation

Increase in demand

$$= [0.69 \quad 0.31]\begin{bmatrix} 0.18 & 0.30 & 0.25 & 0.21 & 0.05 \\ 0.11 & 0.27 & 0.34 & 0.15 & 0.13 \end{bmatrix} = [0.16 \quad 0.29 \quad 0.28 \quad 0.19 \quad 0.07]$$

Decrease in resistance

Worst case

$$= [0.69 \quad 0.31]\begin{bmatrix} 0 & 0.13 & 0.25 & 0.50 & 0.13 \\ 0.50 & 0.36 & 0.09 & 0 & 0.05 \end{bmatrix} = [0.16 \quad 0.20 \quad 0.20 \quad 0.35 \quad 0.10]$$

Best case

$$= [0.69 \quad 0.31]\begin{bmatrix} 0 & 0.13 & 0.25 & 0.50 & 0.13 \\ 0.06 & 0.27 & 0.38 & 0.11 & 0.18 \end{bmatrix} = [0.02 \quad 0.17 \quad 0.29 \quad 0.38 \quad 0.14]$$

Structural deficiency

Worst case

$$= [0.25 \quad 0.75]\begin{bmatrix} 0.16 & 0.29 & 0.28 & 0.19 & 0.07 \\ 0.16 & 0.20 & 0.20 & 0.35 & 0.10 \end{bmatrix} = [0.16 \quad 0.22 \quad 0.22 \quad 0.31 \quad 0.09]$$

Best case

$$= [0.25 \quad 0.75]\begin{bmatrix} 0.16 & 0.29 & 0.28 & 0.19 & 0.07 \\ 0.02 & 0.17 & 0.29 & 0.38 & 0.14 \end{bmatrix} = [0.05 \quad 0.20 \quad 0.29 \quad 0.33 \quad 0.12]$$

Building vulnerability

Worst case

$$= [0.69 \quad 0.31]\begin{bmatrix} 0.16 & 0.22 & 0.22 & 0.31 & 0.09 \\ 0.13 & 0.34 & 0.31 & 0.11 & 0.10 \end{bmatrix} = [0.15 \quad 0.26 \quad 0.25 \quad 0.24 \quad 0.10]$$

Best case

$$= [0.69 \quad 0.31]\begin{bmatrix} 0.05 & 0.20 & 0.29 & 0.33 & 0.12 \\ 0.13 & 0.34 & 0.31 & 0.11 & 0.10 \end{bmatrix} = [0.08 \quad 0.24 \quad 0.30 \quad 0.26 \quad 0.12]$$

Step 4: Defuzzification

Worst case

$$I^{BV} = \sum_{i=1}^{n} q_i * \mu_i = 0*0.15 + 2.5*0.26 + 5*0.25 + 7.5*0.24 + 10*0.10 = 4.70$$

Best case

$$I^{BV} = \sum_{i=1}^{n} q_i * \mu_i = 0*0.08 + 2.5*0.24 + 5*0.30 + 7.5*0.26 + 10*0.12 = 5.24$$

Thus, for building ID *BNG-10-3-10,* the building vulnerability index I^{BV} = [4.70, 5.24]. It should be noted that the worst case scenario is showing better I^{BV} indicator than the best case scenario. This is due to the counter intuitive fuzzification reported in Table 3.8. These intervals in building vulnerability are due to ignorance in the value performance modifiers, consequently, taking the worst and best case value of the performance modifiers will generate these intervals. Further, incorporating the decision maker's risk attitude, the interval risk values can further be reduced into a crisp number for ranking and prioritization of repair. In a similar way, the building damageability index I^{BD} and risk index I^{R} can be computed.

3.6 Case study

3.6.1 May 1, 2003 city of Bingöl Earthquake, Turkey

On May 1, 2003, the city of Bingöl, Turkey, was struck by an earthquake of moment magnitude M_w = 6.4, resulting in 168 casualties, 520 injuries and damage to several buildings.

The total economic loss to the Turkish national economy was estimated to be over 400 million US dollars (Doğangün 2004). Summary of the Bingöl Database[3] is shown in Table 3.9. The damage is classified into five discrete stages: none (D_N), light (D_L), moderate (D_M), severe (D_S) and collapse (D_C). Failures due to poor construction and soft story are illustrated in Figure 3.6.

For the N10E component, the reported peak ground acceleration PGA (cm/s^2), peak ground velocity PGV (cm/s) and peak ground displacement PGD (cm) were 535.3, 36.1 and 26.6, respectively (Gülkan and Akkar, 2004). The geotechnical investigation (Bobet *et al.,* 2004) indicates that the buildings were located on an alluvial deposit, which is classified as stiff soil. The response spectrum provided in Gülkan and Akkar (2004) is used for quantification of the hazard. The five-percent damped response spectra reported in Gülkan and Akkar (2004) do not take the spatial variability of the hazard. Thus, the single response spectrum may not be representative for all buildings; however, it will be used as a general indicator. Also, the available data does not include sufficient information to compute building importance/exposure, thus no risk evaluation is provided.

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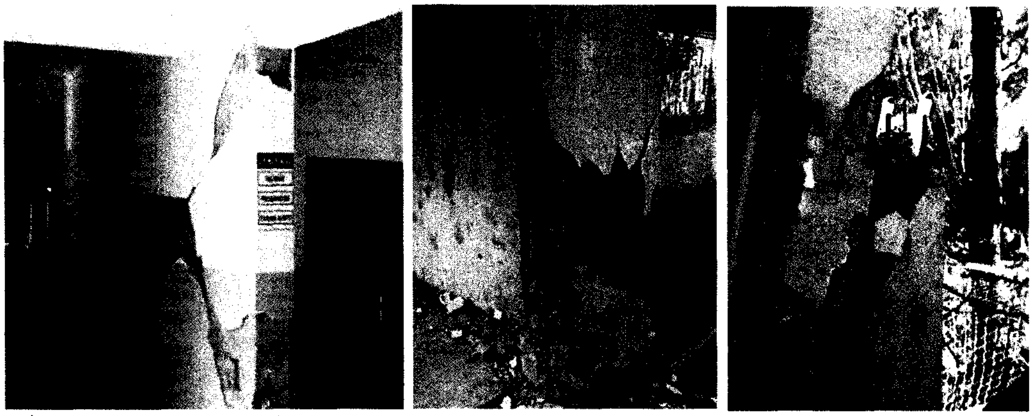
[3] SERU, Middle East Technical University, Ankara, Turkey; Archival Material from Bingöl Database located at website http://www.seru.metu.edu.tr.



a) Shear failure at shear wall due to insufficient amount of transverse reinforcement and poor concrete quality.

b) Shear failure at column base due to insufficient amount of transverse reinforcement.

c) Insufficient development length at the top of column

d) Collapsed 4- storey building due to soft storey

Figure 3.6: Causes of failure due to structural deficiency.

The FSE is performed on the datasets summarized in Table 3.9 and the corresponding I^{BD} values are computed. For plotting purpose, the five discrete damage states, *N*, *L*, *M*, *S* and *C*, are assigned numeric values, 1, 2, 3, 4, and 5, respectively. The plots of the I^{BD} and reported damage states are shown in Figure 3.7a and b, respectively, with corresponding linear best fit line. From Figure 3.7a and b, it can be discerned that I^{BD} increases with the severity of damage. The model uncertainty is reflected in the variability of I^{BD} values at each damage state, namely the scatter at each damage state. This scatter highlights the need to gather more information on the potential causes of building vulnerability and site seismic hazard.

The estimated damage states of 2003 Bingöl earthquake data are summarize in Table 3.9, which shows a good agreement between the estimated and observed damage states. The misclassifications reported in Table 3.9 are only by one damage state, and most of the misclassification is erring on the conservative side, i.e., estimated damage state of severe whereas the actual damage state is *moderate*.

Table 3.9: Summary of Bingöl database.

BUILDING ID	*BSS*	*N*	*YC*	*VI*	*PI*	*CQ*	*Sa*	Observed Damage	Fuzzification {VL, L, M, H, VH}	I^{BD}	Estimated Damage
BNG-10-3-10	RCF	3	-	Yes	Yes	Poor	1.10	Moderate	{0.07, 0.22, 0.27, 0.3, 0.14}	5.56	Severe
BNG-10-3-3	RCF	3	1975	No	No	Poor	1.10	Moderate	{0.07, 0.23, 0.27, 0.28, 0.15}	5.59	Severe
BNG-10-4-4	RCF	4	1998	Yes	Yes	Average	0.70	Moderate	{0.12, 0.3, 0.27, 0.24, 0.07}	4.58	Light
BNG-10-4-6	RCF	4	1976	Yes	Yes	Average	0.70	Light	{0.11, 0.29, 0.28, 0.25, 0.08}	4.72	Light[§]
BNG-10-4-7	RCF	4	1988	Yes	Yes	Average	0.70	Light	{0.11, 0.29, 0.28, 0.25, 0.08}	4.72	Light[§]
BNG-10-4-9	RCSW	4	2002	Yes	Yes	Good	0.70	None	{0.21, 0.25, 0.13, 0.2, 0.25}	5.20	None[§]
BNG-10-5-1	RCSW	5	1990	Yes	Yes	Average	0.70	Moderate	{0.09, 0.19, 0.22, 0.28, 0.25}	6.14	Moderate[§]
BNG-10-5-11	RCF	5	1988	Yes	No	Average	0.70	Light	{0.11, 0.29, 0.28, 0.24, 0.08}	4.70	Light[§]
BNG-10-5-2	RCSW	5	1990	No	Yes	Good	0.70	Light	{0.2, 0.25, 0.14, 0.19, 0.26}	5.26	None
BNG-11-2-3	RCF	2	-	No	No	Poor	1.00	Moderate	{0.13, 0.23, 0.22, 0.3, 0.12}	5.13	Moderate[§]
BNG-11-4-1	RCSW	4	1998-1999	Yes	Yes	Poor	0.70	Severe	{0.05, 0.13, 0.21, 0.36, 0.29}	6.91	Severe[§]
BNG-11-4-2	RCF	4	1989	Yes	Yes	Poor	0.70	Severe	{0.07, 0.22, 0.27, 0.33, 0.12}	5.49	Severe[§]
BNG-11-4-4	RCF	4	2000	Yes	Yes	Poor	0.70	Moderate	{0.08, 0.23, 0.26, 0.32, 0.11}	5.35	Severe
BNG-11-4-5	RCF	4	1997	No	Yes	Average	0.70	Light	{0.12, 0.3, 0.28, 0.23, 0.08}	4.64	Light[§]
BNG-3-4-1	RCF	4	1998	No	No	Poor	0.70	Light	{0.08, 0.23, 0.26, 0.31, 0.12}	5.38	Severe
BNG-3-4-2	RCF	4	1996	No	No	Average	0.70	None	{0.12, 0.3, 0.28, 0.23, 0.08}	4.61	Light
BNG-3-4-4	RCF	4	-	No	No	Average	0.70	None	{0.17, 0.3, 0.23, 0.24, 0.06}	4.31	None[§]
BNG-5-5-1	RCF	5	1990	Yes	Yes	Average	0.70	Light	{0.11, 0.29, 0.28, 0.25, 0.08}	4.72	Light[§]
BNG-6-2-8	RCF	2	1992	Yes	Yes	Poor	1.00	Severe	{0.07, 0.22, 0.27, 0.31, 0.14}	5.54	Severe[§]
BNG-6-3-1	RCF	3	1991	Yes	No	Average	1.10	Moderate	{0.11, 0.29, 0.28, 0.21, 0.1}	4.77	Light
BNG-6-3-10	RCF	3	1995	Yes	Yes	Good	1.10	None	{0.23, 0.34, 0.19, 0.14, 0.1}	3.85	None[§]
BNG-6-3-11	RCF	3	-	Yes	Yes	Average	1.10	None	{0.17, 0.3, 0.22, 0.23, 0.08}	4.35	None[§]
BNG-6-3-12	RCF	3	-	Yes	Yes	Average	1.10	Light	{0.17, 0.3, 0.22, 0.23, 0.08}	4.35	None
BNG-6-3-4	RCF	3	2003	No	No	Average	1.10	Light	{0.12, 0.3, 0.28, 0.2, 0.1}	4.68	Light[§]
BNG-6-4-2	RCF	4	2001	Yes	Yes	Poor	0.70	Severe	{0.08, 0.23, 0.26, 0.32, 0.11}	5.35	Severe[§]
BNG-6-4-3	RCF	4	2003	Yes	Yes	Poor	0.70	Collapse	{0.08, 0.23, 0.26, 0.32, 0.11}	5.35	Severe
BNG-6-4-5	RCF	4	1996	No	Yes	Good	0.70	None	{0.24, 0.35, 0.18, 0.15, 0.08}	3.70	None[§]
BNG-6-4-7	RCSW	4	1996	No	No	Poor	0.70	Severe	{0.07, 0.17, 0.22, 0.31, 0.27}	6.53	Severe[§]

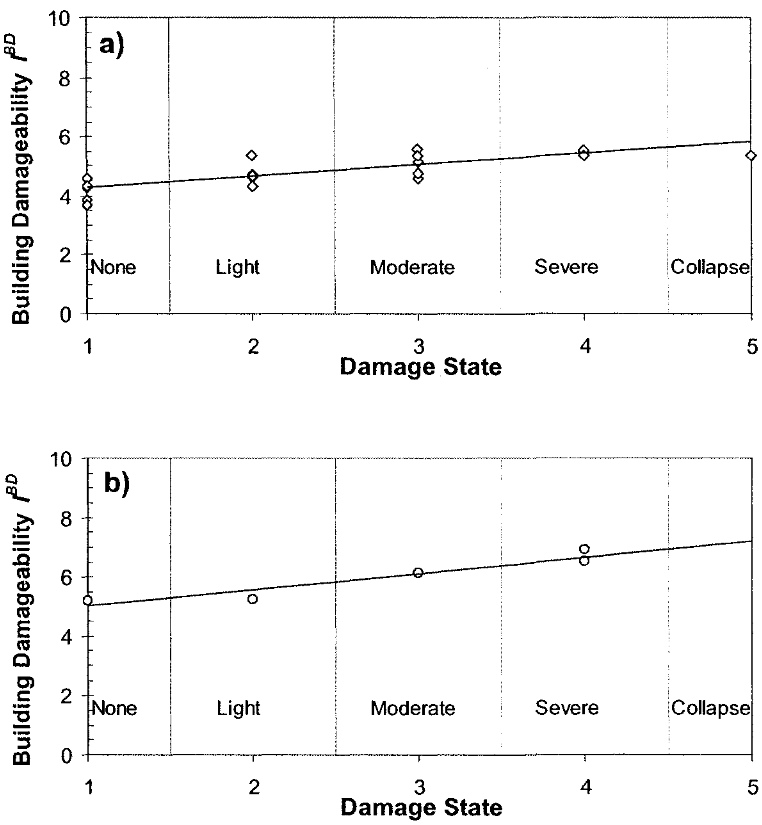


Figure 3.7: Building damageability for 2003 Bingöl Earthquake a) Frame RC building b) Shear wall RC building.

3.6.2 January 17, 1994 Northridge Earthquake, California, USA

The proposed method is further tested over the 1994 Northridge Earthquake data (Table 3.10). The Northridge Earthquake with a moment magnitude M_w = 6.7 struck the San Fernando Valley on January 17, 1994. Because of its proximity to communities in the Los Angeles basin, there was tremendous damage (EERI 1994). This earthquake and the ATC-38 (ATC 2001) building performance and strong motion data have been adopted for the case study presented in this section to demonstrate the application of the proposed risk-based assessment procedure.

Table 3.10: Northridge earthquake model validation data.

Building ID	Structural system	Building use	Vertical irregularity	plan irregularity	Spectral acceleration	Construction quality	Number of stories	Year of construction	Damage	Building damage index	Risk index	Rank	
									$I^{BD'}$	I^{BD}	I^{R}	I^{BD}	I^{R}
CDMG231-GZ-16	C1	S	N	N	0.27	Good	6	1966	2	1	0.06	7	7
CDMG579-SI-01	C1	O	Y	Y	0.16	Average	9	1924	1	1	0.06	7	7
USGS233-GZ-16	C1	H	N	N	0.08	Good	16	1960	1	1	0.26	7	1
CDMG567-GZ-02	C1	GV	Y	Y	0.09	Average	16	1990	1	1	0.06	7	7
CDMG231-ER-01	C3	S	N	Y	0.54	Good	3	1931	2	2	0.22	1	2
CDMG231-GZ-19	C3	S	N	Y	0.54	Good	3	1931	2	2	0.22	1	2
CDMG688-RE-04	C3	S	N	N	0.42	Good	5	1950	1	1	0.06	7	7
USC060-GTZ-01	C3	GV	N	Y	0.33	Good	1	1970	2	1	0.06	7	7
USGS233-GZ-02	C3	O	N	N	0.20	Good	3	1950	1	1	0.06	7	7
CDMG231-GZ-06	C2	S	N	N	0.55	Good	4	1950	1	1	0.06	7	7
CDMG231-GZ-10	C2	S	N	N	0.44	Good	5	1961	1	1	0.06	7	7
CDMG231-GZ-11	C2	S	N	N	0.38	Good	4	1961	1	1	0.06	7	7
CDMG231-GZ-12	C2	S	N	Y	0.29	Good	2	1961	1	2	0.22	1	2
USGS233-GZ-15	C2	G	N	Y	0.44	Average	7	1960	2	1	0.00	7	20
USC058-MB-11	C2	S	N	N	0.35	Good	2	1994	1	1	0.06	7	7
CDMG463-AC-07	C2	W	N	Y	1.08	Good	1	1940	2	2	0.05	1	19
CDMG303-JH-07	C2	RS	N	N	0.44	Good	1	1960	1	1	0.00	7	20
USC021-GTZ-02	C2	S	N	Y	0.23	Average	2	1930	2	1	0.06	7	7
CDMG231-GZ-14	C2	S	N	N	0.89	Good	4	1965	1	1	0.06	7	7
CDMG231-GZ-15	C2	S	N	Y	0.66	Good	5	1961	1	2	0.22	1	2
CDMG231-GZ-02	C2	S	Y	Y	0.37	Good	4	1957	2	2	0.225	1	2
CDMG231-GZ-16	C1	S	N	N	0.27	Good	6	1966	2	1	0.06	7	7

Similar to the previous case study, the FSE is used to compute the building damageability index I^{BD} plotted in Figure 3.8a-c. Figure 3.8a-c show I^{BD} values for frame RC building (*C1*), shear wall RC building (*C2*) and masonry infill frame building (*C3*), respectively. It should be mentioned that for the Northridge earthquake, the histogram based membership function generation is biased towards the lower damage states. However, from these figures, it can be seen that there is an increasing trend of the I^{BD} with increasing damage states.

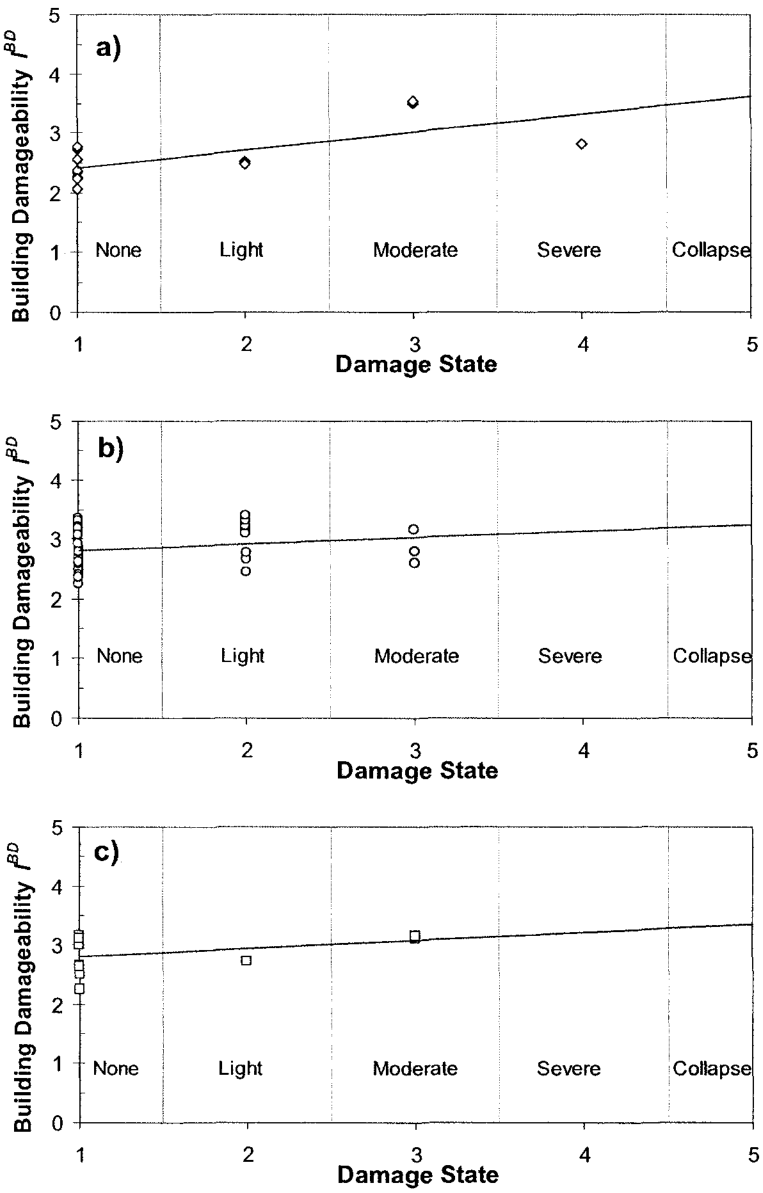


Figure 3.8: Building damageability for 1994 Northridge Earthquake. a) Frame RC building b) Shear wall RC building. c) masonry infill frame building.

3.7 Conclusions

Weighted based models are discussed, and utility of fuzzy synthetic evaluation technique illustrated through the 2003 Bingöl earthquake and 1994 Northridge Earthquake. The FSE modeling of both 2003 Bingöl earthquake and 1994 Northridge earthquake show good correlation with observed damage. The hierarchical structure and performance modifiers identified are intended to capture the structural deficiencies identified in FEMA 154 (ATC

2002). However, for specific regional estimation of risk, other performance modifiers need to be incorporated, e.g. heavy over hang (Yücemen *et al.* 2004; Sucuoğlu *et al.* 2007). The hazard considered for the 2003 Bingöl earthquake is represented by a single response spectrum. In further implementation of the proposed risk analysis, the proposed method has to be implemented in a GIS based platform to capture spatial variability. Furthermore, sensitivity of different hazard quantification procedure should be considered. In spite of the limited data used in it validation, the proposed heuristic method of estimating seismic risk appears to be promising.

As was reasoned earlier, the linear aggregation techniques can not handle complex problems, for example, when it is required to model intricate interaction of site seismic hazard and building vulnerability. The fuzzy rule base (FRB) modelling utilizes a fuzzy logic that offers the option of modeling highly non-linear model of the human brain. In the subsequent two chapters, Chapter 4 and Chapter 5, the FRB modeling is applied for Tier 1 and Tier 2 models, respectively.

Chapter 4
Tier 1 Model using Fuzzy Rule Base

"Everything is vague to a degree you do not realize till you have tried to make it precise."

Bertrand Russell

4.1 Theory of fuzzy-based technique

Fuzzy logic provides a language with semantics to translate qualitative knowledge into numerical reasoning. The strength of fuzzy logic is that it can integrate descriptive (linguistic) knowledge and numerical data into a fuzzy model and use approximate reasoning algorithms to propagate the uncertainties throughout the decision process. The fuzzy inference system (FIS) contains three basic features (Zadeh 1973):

- linguistic variables instead of, or in addition to, numerical variables;
- relationships between the variables in terms of IF-THEN rules (rule-base); and
- an inference mechanism that uses approximate reasoning algorithms to formulate relationships.

In recent years, fuzzy logic, or more generally, fuzzy set theory, has been applied extensively in earthquake engineering. Reported applications include seismic prediction, structural analysis and design, evaluation of existing buildings, and post-disaster assessment, as itemized below:

- Fuzzy risk model: Huang and Moraga (2002); Chongfu (1996); Shah *et al.* (1990).

- Earthquake damage assessment: Karimi and Hüllermeier (2007); Demartinos and Dritsos (2006); Carreño *et al.* (2004); Sánchez-Silva and Garcia (2001); Hadipriono and Ross (1991); Souflis and Grivas (1986).
- Determining seismic hazard and decision making: Karimi and Hüllermeier (2007); Dong *et al.* (1987).
- Interpretation of seismic design code: Alim and Smith (1989).
- Seismic inelastic analysis and design: Mistakidis and Georgiou (2003).
- Earthquake signal analysis in time series: Furuta and Nomura (2003); and, short-range seismic prediction: Klose (2002).

4.1.1 Fuzzification

In this and next chapter, the heuristic based method of fuzzification is utilized. At the decision making level (i.e. building damageability index and risk index) five granules are used, however, three granules are used for the basic performance modifiers. The three granules are used in order to reduce the required number of rules. The fuzzification process transforms the performance modifiers into a *cost criterion*, where a membership in "high", H, represents high risk and a membership in "low", L, represents low risk. For example, construction quality of "poor" will have higher membership value in H, and the converse is true.

Heuristic method

The heuristic method of generating membership functions entails use of our intelligence and understanding of the general system. A membership function could have any shape, but its selected shape should be justified by expert opinion or consensus. Figure 4.1 shows the commonly used fuzzy numbers including triangular, trapezoidal, and Gaussian shape fuzzy numbers. In general, triangular fuzzy numbers (TFNs) are used for representing linguistic variables. The TFN is expressed by three vertices, TFN(a_L, b_M, c_H), where a_L is *minimum*,

b_M is *most likely* and c_H is *maximum* values, respectively. The triangular membership values are represented by the following set of equations:

$$\mu_{A_i}(x) = \begin{cases} 0, & 0 \le x \le a_L \\ \dfrac{x - a_L}{b_M - a_L}, & a_L \le x \le b_M \\ \dfrac{c_H - x}{c_H - b_M}, & b_M \le x \le c_H \\ 0, & c_H \le x \end{cases} \tag{4.1}$$

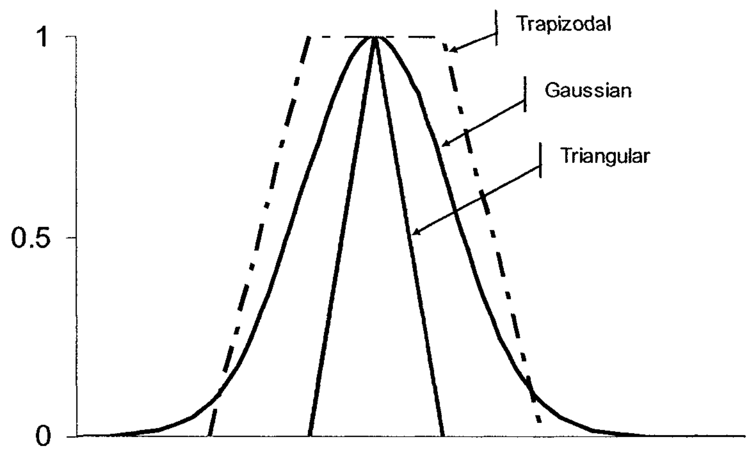


Figure 4.1: Typical Fuzzy Membership Functions.

The transformation and fuzzification is illustrated with the year of construction (YC). The transformation for YC is provided in Eq. (3.2). Equation (3.2) clusters the YC into three groups, *low code*, *moderate code*, and *high code*. The transformed value is furnished in the interval of [0, 1], which should further be fuzzified. The *cost criterion* fuzzification of the YC is provided as a three-tuple fuzzy granules; L, M and H. Each granule of the L, M and H has corresponding TFNs: TFN(0, 0, 0.4), TFN(0, 0.4, 1) and TFN(0.4, 1, 1), respectively. The three granules of the YC are summarized in column 3 of Table 4.1 as [TFN(0, 0, 0.4); TFN(0, 0.4, 1); TFN(0.4, 1, 1)]. The transformation values provided using Eq. (3.2) and the three-tuple fuzzy granules are plotted in Figure 4.2.

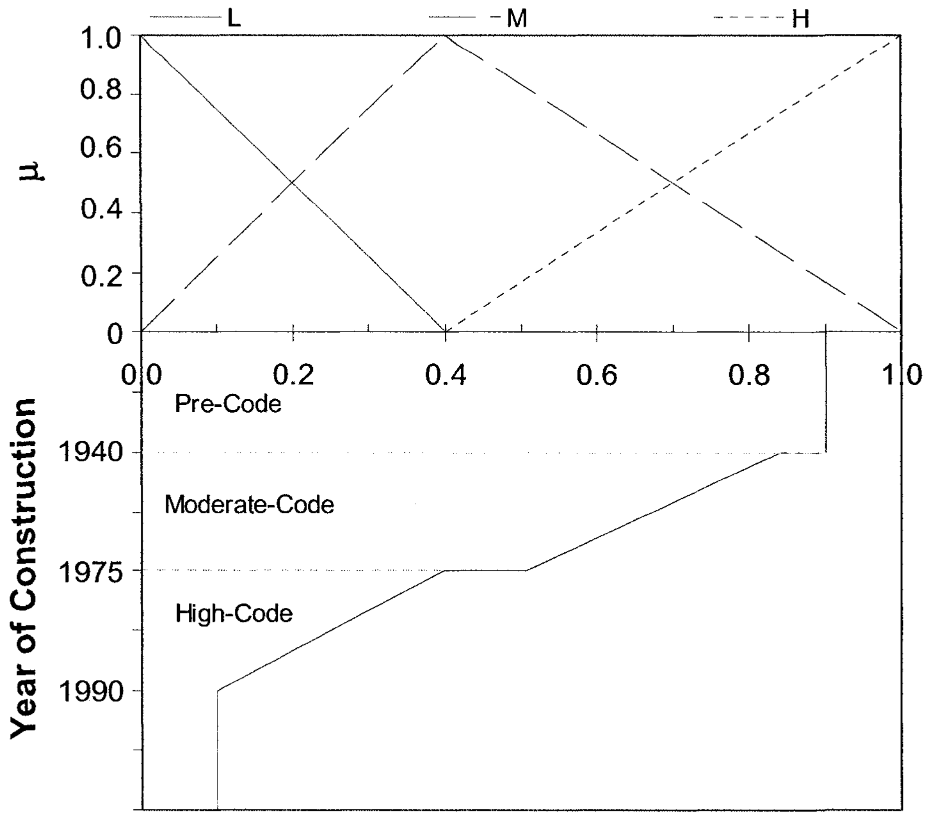


Figure 4.2: Transformation and fuzzification of the year of construction.

For each performance modifier associated with building vulnerability index, the transformation values and three-tuple fuzzy granules, are summarized in Table 4.1. Column 1 of Table 4.1 presents the performance modifier under consideration (e.g. vertical irregularity). The corresponding linguistic quantifier and transformation values are provided in column 2 of Table 4.1. The fuzzfication, corresponding FRB and defuzzification are provided in columns 3, 4, and 5, respectively. Also the three-tuple fuzzy granules are plotted in Figure 4.3.

The transformation values of vertical irregularity are 0.80 for *yes*, and 0.10 for *no*. The three-tuple fuzzy granules are summarized in Table 4.1 and plotted in Figure 4.3a. The transformation values assigned to plan irregularity are; 0.80 for *yes*, and 0.20 for *no*. The three-tuple fuzzy granules are summarized in Table 4.1 and plotted in Figure 4.3b.

Table 4.1: Transformation, fuzzfication, aggregation and defuzzification of performance modifiers.

Parameter	Transformation§	Fuzzification	Rule base	Defuzzification§
Vertical irregularity	(*Yes*, *No*) (0.80, 0.10)	[L; M; H] [TFN(0, 0, 0.5); TFN(0, 0.5, 1); TFN(0.5, 1, 1)]	R_1	(L, M, H) (0.25, 0.5, 1)
Plan irregularity	(*Yes*, *No*) (0.80, 0.20)	[L; M; H] [TFN(0, 0, 0.5); TFN(0, 0.5, 1); TFN(0.5, 1, 1)]		
Construction quality	(*Good*, *Average*, *Poor*) (0.01, 0.70, 0.99)	[L; M; H] [TFN(0, 0, 0.5); TFN(0, 0.5, 1); TFN(0.5, 1, 1)]	R_2	(L, M, H) (0.1, 0.6, 0.7)
Year of construction	Eq. (3.2)	[*L*; *M*; *H*] [TFN(0, 0, 0.4); TFN(0, 0.4, 1); TFN(0.5, 1, 1)]		
Increase in demand		[L; M; H] [TFN(0, 0.1, 0.4); TFN(0.1, 0.4, 0.8); TFN(0.4, 0.8, 1)]	R_3	(L, M, H) (0.1, 0.6, 0.9)
Decrease in resistance		[*L*; *M*; *H*] [TFN(0, 0.5, 0.7); TFN(0.5, 0.7, 1); TFN(0.5, 1, 1)]		
Structural deficiency		[L; M; H] [TFN(0, 0.1, 0.4); TFN(0.1, 0.4, 0.8); TFN(0.4, 0.8, 1)]	R_4	(L, M, H) (0.01, 0.5, 0.9)
Building structural system	(*C1*, *C2*, *C3*) (0.70, 0.25, 0.35)	[L; M; H] [TFN(0, 0, 0.5); TFN(0, 0.5, 1); TFN(0.5, 1, 1)]		

L = Low; M = Medium; H = High

§Calibration of the transformation and defuzzification values are discussed in Section 4.2: model calibration

The qualitative evaluation of construction quality is transformed as: 0.99 for *poor*, 0.7 for *average* and 0.01 for *good*. The three-tuple fuzzy granules are summarized in Table 4.1 and plotted in Figure 4.3c. The transformation values used for building structural system are; 0.7 for *moment resisting frames (C1)*, 0.35 for *moment resisting frames with masonry infills (C3)*, and 0.25 for *shear walls (C2)*. The TFN of the fuzzification are summarized in Table 4.1 and plotted in Figure 4.3h.

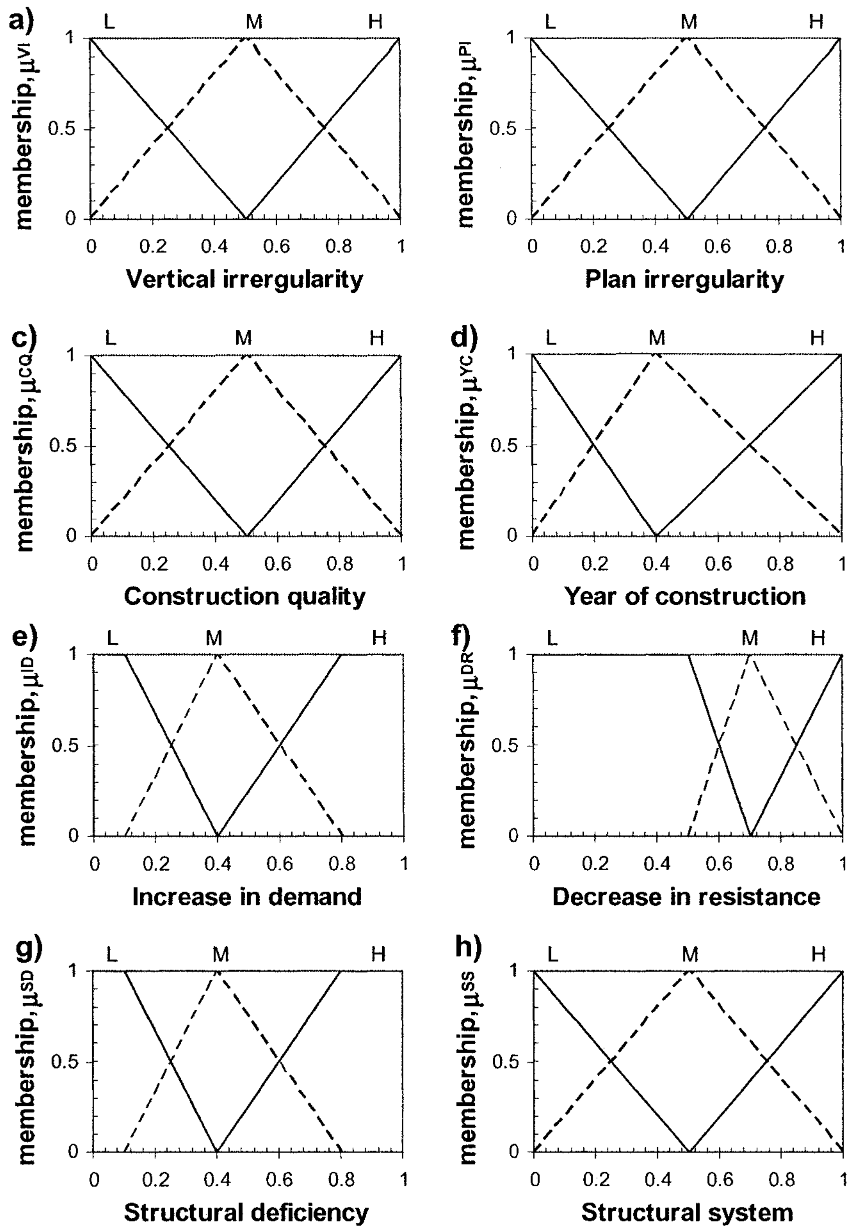


Figure 4.3: Fuzzification of performance modifiers and intermediate parameters: three granules a) vertical irregularity, b) plan irregularity, c) construction quality d) year of construction, e) increase in demand, f) decrease in resistance, g) structural deficiency, and h) structural system

For the building importance factor, the transformation values, three-tuple fuzzy granules and defuzzification parameters are summarized in Table 4.2. Also the three-tuple fuzzy granules are plotted in Figure 4.4.

Table 4.2: Building importance factor.

Parameter	Quantifier	Transfo-rmation	Fuzzification	Rule base
Building use	Neither LS nor IO	0.10	[L; M; H] [TFN(0, 0, 0.5); TFN(0, 0.5, 1); TFN(0.5, 1, 1)]	R_5
	LS	0.50		
	IO	0.90		
Occupancy	0-10	0.10	[L; M; H] [TFN(0, 0, 0.5); TFN(0, 0.5, 1); TFN(0.5, 1, 1)]	
	11-100	0.40		
	101-1000	0.75		
	> 1000	0.90		
Economic impact	Negligible	0.10	[L; M; H] [TFN(0, 0, 0.5); TFN(0, 0.5, 1); TFN(0.5, 1, 1)]	
	Average	0.50		
	Significant	0.90		

VL = Very low; L = Low; M = Medium; H = High; VH = Very high

The transformation values for building use are 0.90 for IO, 0.50 for LS, and 0.10 for neither LS nor IO. The three-tuple fuzzy granules are summarized in Table 4.2 and plotted in Figure 4.4a. The transformation values for occupancy are 0.90 for occupancy of >1000, 0.75 for occupancy of 101-1000, 0.40 for occupancy of 11-100, and 0.10 for occupancy of 0-10. The three-tuple fuzzy granules are summarized in Table 4.2 and plotted in Figure 4.4b. The transformation values for economic impact are 0.90 for significant, 0.50 for average, and 0.10 for negligible. The three-tuple fuzzy granules are summarized in Table 4.2 and plotted in Figure 4.4c. The building importance/ exposure five-tuple fuzzy granules are summarized in Table 4.2 and plotted in Figure 4.4d.

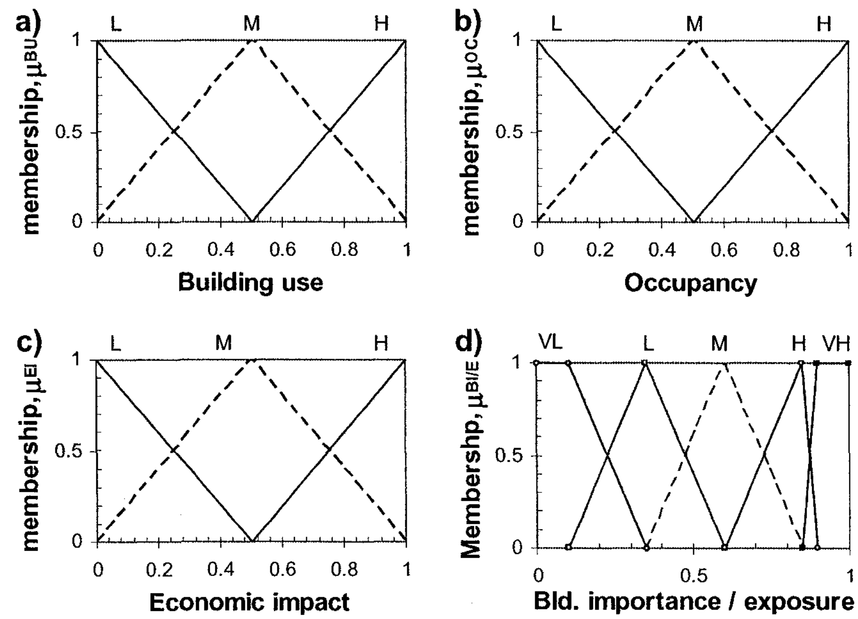


Figure 4.4: Fuzzification of building importance / exposure and corresponding performance modifiers a) building use, b) occupancy, c) economic impact d) building importance and exposure

The five-tuple granulation of I^{SH} and I^{BV} are summarized Table 4.3 and plotted in Figure 4.5a and b, respectively.

Table 4.3: Building damageability.

Parameter	Fuzzification	Rule base	Defuzzification
Building vulnerability index	[VL; L; M; H; VH] [TFN(0, 0.1, 0.4); TFN(0.1, 0.4, 0.6); TFN(0.4, 0.6, 0.75); TFN(0.6, 0.75, 0.85); TFN(0.75, 0.85, 1)]	R_6	Maximum membership value is used to estimate the corresponding damage state.
Site seismic hazard index	[VL; L; M; H; VH] [TFN(0, 0, 0.4); TFN(0.1, 0.4, 0.7); TFN(0.4, 0.7, 1.2); TFN(0.7, 1.2, 1.25); TFN(1.2, 1.25, 2)]		
Building damageability index	Table 4.4	R_7	(VL; L; M; H; VH) (0, 0.20, 0.40, 0.80, 1)
Building importance/ exposure	[VL; L; M; H; VH] [TFN(0, 0, 0.4); TFN(0.1, 0.4, 0.6); TFN(0.4, 0.6, 1.2); TFN(0.6, 0.9, 1); TFN(0.9, 1, 1)]		

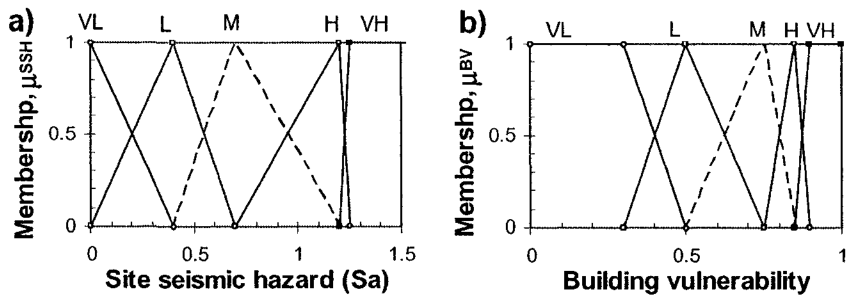
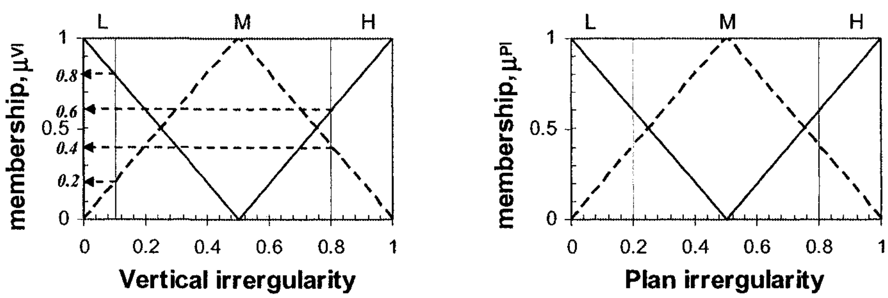


Figure 4.5: Fuzzification of hazard and building vulnerability a) site seismic hazard, b) building vulnerability

The process of fuzzification is illustrated with an example.

Example. The process of granulation and fuzzification is illustrated for vertical irregularity (VI) and plan irregularity (PI). From a *walk down* survey, the VI and PI are linguistically obtained as *yes* for present and *no* for not present. The VI and PI are transformed between 0 to 1, [0, 1]. The transformations provided in column 2 of Table 4.1 are: VI {*yes* = 0.80, *no* = 0.10}, and PI {*yes* = 0.80, *no* = 0.20}. The three-tuple fuzzy granule, L, M, and H, for VI and PI are column 3 of Table 4.1, are [TFN(0, 0, 0.5); TFN(0, 0.5, 1); TFN(0.5, 1, 1)], which are also plotted below.



With the VI = *yes*, the transformed value of 0.8 intercepts the TFN of membership value of M and H at μ_M = 0.40 and μ_H = 0.60, respectively. For all transformation values, the overall fuzzfication of VI and PI can be shown as:

$$\text{VI} = Yes \rightarrow \left(\mu_L^{VI}, \mu_M^{VI}, \mu_H^{VI}\right) = (0, 0.40, 0.60),$$

$$\text{VI} = No \rightarrow \left(\mu_L^{VI}, \mu_M^{VI}, \mu_H^{VI}\right) = (0.80, 0.20, 0),$$

$$\text{PI} = Yes \rightarrow \left(\mu_L^{PI}, \mu_M^{PI}, \mu_H^{PI}\right) = (0, 0.\ 40, 0.60),$$

$$\text{PI} = No \rightarrow \left(\mu_L^{VI}, \mu_M^{VI}, \mu_H^{VI}\right) = (0.60, 0.40, 0).$$

Each performance modifier can be fuzzified in a similar way. The building damageability is classified into five discrete states, *none-slight* ($D_{N\text{-}S}$), *light* (D_L), *moderate* (D_M), *heavy* (D_H) and *major-destroyed* ($D_{M\text{-}D}$). Thus, once the damage state is estimated, for risk index calculation, the fuzzification for building damageability is provided in Table 4.4.

Table 4.4: Fuzzification of building damageability

			μ_i^{BD}		
Damage state	*VL*	*L*	*M*	*H*	*VH*
$D_{N\text{-}S}$	0.7	0.3	0	0	0
D_L	0	0.7	0.3	0	0
D_M	0	0	0.7	0.3	0
D_H	0	0	0	0.7	0.3
$D_{M\text{-}D}$	0	0	0	0.3	0.7

4.1.2 Fuzzy inference system (FIS)

Information of the FIS is encapsulated into two modules; i) a fuzzy knowledge base and ii) an inference mechanism. The former is a model developed based on expert knowledge and/or input-output data[4]. The inference mechanism then uses the knowledge base to estimate the output of the system for a given input. The use of fuzzy rule based technique allows decision makers to express their preferences in a modular fashion (Yager and Filev 1994). A modularized design of the FIS enables it to maintain a generic processing structure that is

[4] Different methods of generating FIS are discussed in Appendix A, under section of fuzzy modeling.

capable of dealing with various systems in different application. Also, the FIS can readily be updated by modifying the knowledge base using new information as it becomes available.

4.1.3 Knowledge base

The knowledge base defines the relationships between the input and output parameters of a system. The most commonly used representation of the input-output relationships is Mamdani type fuzzy models (after Mamdani 1977). In this type of fuzzy models, linguistic propositions are used both in *antecedent* and *consequent* parts of IF-THEN rules. The fuzzy rule base (FRB) consists of a collection of rules, which can express the decision maker's opinion valuation for a particular uncertain environment. The IF-THEN rules can be established as:

$$R_i : \textbf{IF } x_1 \text{ is } A_{i1} \textbf{ AND } x_2 \text{ is } A_{i2} \textbf{ THEN } y \text{ is } B_i \text{ , } i = 1,\ldots,n \tag{4.2}$$

where R_i represents the i^{th} rule, x_1 and x_2 are input (antecedent) linguistic variables, n is the total number of rules, A_{i1} and A_{i2} are input fuzzy sets, y is output (consequent) linguistic variable and B_i is the consequent fuzzy set.

The rule base of a complex system usually requires a large number of rules to describe the behaviour of a system for all possible values of the input variables. This is referred to as the "completeness" of a fuzzy model. The aggregation of the rules described in Eq. (4.2) forms a rule base that is valid over the entire application domain. The aggregation is obtained using the union of the rules or subsystems as,

$$R = \bigcup_{i=1}^{n} R_i = \ R_1 \textbf{ ALSO } R_2 \textbf{ ALSO} \ldots \textbf{ ALSO } R_n \tag{4.3}$$

The number of inputs and granules are used to determine the required number of rule bases. For example, for two inputs and three granules, the total number rule is $3^2 = 9$.

Example. The fuzzification of VI and PI discussed earlier are used to calculate increase in demand (ID). A heuristic based method is used to define the relationships between the VI and PI, and ID. As shown in Eq. (4.2), each rule base expresses the decision maker's opinion for a particular uncertain environment as follows:

R_1: **IF** VI = L **AND** PI = L **THEN** ID = L

R_2: **IF** VI = L **AND** PI = M **THEN** ID = L

...

R_6: **IF** VI = M **AND** PI = H **THEN** ID = H

...

R_9: **IF** VI = H **AND** PI = H **THEN** ID = H

These rule bases can be presented in a tabular form as follows:

Rule *i*	**VI**	**PI**	**ID**
1	L	L	L
2	L	M	L
3	L	H	M
4	M	L	L
5	M	M	M
6	M	H	H
7	H	L	M
8	H	M	H
9	H	H	H

VI = vertical irregularity; PI = plan irregularity, ID = Increase in demand

The building vulnerability inferencing is performed through four fuzzy rule bases (FRBs) as identified in Figure 3.2, R_i (i = 1,2,3,4). The four FRBs are summarized in Table 4.5 to

Table 4.8.

Table 4.5: Fuzzy rule base for increase in demand (ID).

R_1	**Rule *i***	**VI**	**PI**	**ID**
	1	L	L	L
	2	L	M	L
	3	L	H	M
	4	M	L	L
	5	M	M	M
	6	M	H	H
	7	H	L	M
	8	H	M	H
	9	H	H	H

VI = vertical irregularity; PI = plan irregularity

Table 4.6: Fuzzy rule base for decrease in resistance (DR).

R_2	Rule *i*	CQ	YC	DR
	1	L	L	L
	2	L	M	L
	3	L	H	M
	4	M	L	L
	5	M	M	M
	6	M	H	H
	7	H	L	M
	8	H	M	H
	9	H	H	H

CQ = Construction quality; YC = Year of Construction

Table 4.7: Fuzzy rule base for structural deficiency (SD).

R_3	Rule i	ID	DR	SD
	1	L	L	L
	2	L	M	M
	3	L	H	M
	4	M	L	L
	5	M	M	M
	6	M	H	H
	7	H	L	M
	8	H	M	H
	9	H	H	H

ID = Increase in demand; DR = Decrease in resistance

Table 4.8: Fuzzy rule base for building vulnerability (BV).

R_4	Rule *i*	SD	BSS	BV
	1	L	L	L
	2	L	M	L
	3	L	H	M
	4	M	L	L
	5	M	M	M
	6	M	H	H
	7	H	L	M
	8	H	M	H
	9	H	H	H

SD = Structural deficiency; BSS = Building structural system

The FRB for I^{IE}, I^{BD}, and I^{R} are indicated in Figure 3.2 as $\mathbf{R}_i$ (i = 5,6,7), and corresponding rule base are summarized in Table 4.9, Table 4.10 and Table 4.11, respectively.

Table 4.9: FRB for Building importance/exposure factor (I^{IE}).

$\mathbf{R}_5$	Rule i	BU_{IE}	O_{IE}	E_{IE}	I^{IE}
	1	L	L	L	VL
	2	L	L	M	VL
	3	L	L	H	L
	4	L	M	L	L
	5	L	M	M	M
	6	L	M	H	M
	7	L	H	L	H
	8	L	H	M	H
	9	L	H	H	H
	10	M	L	L	VL
	11	M	L	M	L
	12	M	L	H	M
	13	M	M	L	M
	14	M	M	M	H
	15	M	M	H	H
	16	M	H	L	H
	17	M	H	M	VH
	18	M	H	H	VH
	19	H	L	L	M
	20	H	L	M	M
	21	H	L	H	H
	22	H	M	L	H
	23	H	M	M	H
	24	H	M	H	VH
	25	H	H	L	VH
	26	H	H	M	VH
	27	H	H	H	VH

BU_{IE} = Building use; O_{IE} = Occupancy; E_{IE} = Economy

Table 4.10: Fuzzy rule base for building damageability index I^{BD}.

$\mathbf{R_6}$	**Rule *i***	I^{BV}	I^{SH}	I^{BD}
	1	VL	VL	VL
	2	VL	L	VL
	3	VL	M	VL
	4	VL	H	L
	5	VL	VH	L
	6	L	VL	VL
	7	L	L	VL
	8	L	M	L
	9	L	H	L
	10	L	VH	M
	11	M	VL	VL
	12	M	L	L
	13	M	M	L
	14	M	H	M
	15	M	VH	H
	16	H	VL	L
	17	H	L	L
	18	H	M	M
	19	H	H	H
	20	H	VH	VH
	21	VH	VL	L
	22	VH	L	L
	23	VH	M	H
	24	VH	H	VH
	25	VH	VH	VH

I^{BV} = Building vulnerability index; I^{SH} = site seismic hazard index

Table 4.11: Fuzzy rule base for risk index I^R.

R_7	Rule i	Building damageability index	Building importance/ exposure index	Risk index
	1	VL	VL	VL
	2	VL	L	VL
	3	VL	M	VL
	4	VL	H	L
	5	VL	VH	L
	6	L	VL	VL
	7	L	L	VL
	8	L	M	L
	9	L	H	M
	10	L	VH	M
	11	M	VL	L
	12	M	L	L
	13	M	M	M
	14	M	H	M
	15	M	VH	H
	16	H	VL	M
	17	H	L	M
	18	H	M	H
	19	H	H	VH
	20	H	VH	VH
	21	VH	VL	M
	22	VH	L	H
	23	VH	M	VH
	24	VH	H	VH
	25	VH	VH	VH

4.1.4 Inference mechanism

For linguistic consequent parameters, Mamdani type inferencing can be used (Mamdani, 1977). Mamdani's inference mechanism consists of three connectives: the aggregation of antecedents in each rule (AND connectives), implication (i.e., IF-THEN connectives), and aggregation of the rules (ALSO connectives). The operators performing the connectives distinguish the type of fuzzy inferencing. The AND and ALSO connectives are chosen from a family of *t-norm* and *t-conorm* operators, respectively (Klir and Yuan 1995)[5]. In this thesis, the *minimum* and *maximum* operators are used for *t-norm* and *t-conorm*, respectively.

[5] Different aggregators used in fuzzy inferencing are summarized in Appendix A, under section of Aggregation.

Example. From the *walk down* survey, the presence of VI and PI are identified as *yes* and *no*, respectively. Aggregation of the VI and PI furnishes result for increase in demand (ID). Given the fuzzification of VI and PI $\left(\mu_L^{VI}, \mu_M^{VI}, \mu_H^{VI}\right)$ = (0, 0.40, 0.60) and $\left(\mu_L^{PI}, \mu_M^{PI}, \mu_H^{PI}\right)$ = (0.60, 0. 40, 0), respectively, the fuzzy inferencing, Eq. (4.2), to compute ID is:

R_1: **IF** VI $\mu_L^{VI} = 0$ **AND** PI $\mu_L^{PI} = 0.60$ **THEN** ID $\mu_L^{ID} = \min(0, 0.60) = 0$
R_2: **IF** VI $\mu_L^{VI} = 0$ **AND** PI $\mu_M^{PI} = 0.40$ **THEN** ID $\mu_L^{ID} = \min(0, 0.40) = 0$
R_3: **IF** VI $\mu_L^{VI} = 0$ **AND** PI $\mu_H^{PI} = 0$ **THEN** ID $\mu_M^{ID} = \min(0, 0) = 0$
R_4: **IF** VI $\mu_M^{VI} = 0.40$ **AND** PI $\mu_L^{PI} = 0.60$ **THEN** ID $\mu_L^{ID} = \min(0.40, 0.60) = 0.40$
R_5: **IF** VI $\mu_M^{VI} = 0.40$ **AND** PI $\mu_M^{PI} = 0.40$ **THEN** ID $\mu_M^{ID} = \min(0.40, 0.40) = 0.40$
R_6: **IF** VI $\mu_M^{VI} = 0.40$ **AND** PI $\mu_H^{PI} = 0$ **THEN** ID $\mu_H^{ID} = \min(0.40, 0) = 0$
R_7: **IF** VI $\mu_H^{VI} = 0.60$ **AND** PI $\mu_L^{PI} = 0.60$ **THEN** ID $\mu_M^{ID} = \min(0.60, 0.60) = 0.60$
R_8: **IF** VI $\mu_H^{VI} = 0.60$ **AND** PI $\mu_M^{PI} = 0.40$ **THEN** ID $\mu_H^{ID} = \min(0.60, 0.40) = 0.40$
R_9: **IF** VI $\mu_H^{VI} = 0.60$ **AND** PI $\mu_H^{PI} = 0$ **THEN** ID $\mu_H^{ID} = \min(0.60, 0) = 0$

Using the *maximum* operator and aggregating of the above rules, Eq. (4.3):

μ_L^{ID}=max(0, 0, 0.40) = 0.40;

μ_M^{ID}=max(0, 0.40, 0.60) = 0.60; and

μ_H^{ID}=max(0, 0.40, 0) = 0.40.

Thus, results of the increase demand (ID) is $\left(\mu_L^{ID}, \mu_M^{ID}, \mu_H^{ID}\right)$ = (0.40, 0.60, 0.40).

4.1.5 Defuzzification

The defuzzification process of the fuzzy membership values are provided in Eq. (3.7). The defuzzification quality factors q_i of the FRB are bounded between 0 and 1, $q_i \in [0, 1]$. The q_i used in the building vulnerability assessment are provided in Table 4.1. From column 5 of Table 4.1, the q_i (i = 1,2,3) values for defuzzifying outputs of $\mathbf{R_1}$, $\mathbf{R_2}$, $\mathbf{R_3}$ and $\mathbf{R_4}$ are (0.25, 0.50, 1), (0.1, 0.6, 0.7), (0.1, 0.6, 0.9) and (0.01, 0.5, 0.9), respectively. The defuzzification quality factors q_i used in the risk index is provided in Table 4.3. From column 4 of Table 4.3, the q_i values used for defuzzifying output of **R**$_7$ is q_i (i = 1,…,5) = (0, 0.20, 0.40, 0.80, 1). For the damage assessment, however, the maximum membership value is used to determine the damage state.

Example. From the previous example, inferencing of the FRB for increase demand (ID) resulted in, $\left(\mu_L^{ID}, \mu_M^{ID}, \mu_H^{ID}\right)$ = (0.40, 0.60, 0.40). Given the ID defuzzification quality factors q_i = (0.25, 0.5, 1), the ID is defuzzified using Eq. (3.7),

ID = 0.25* 0.40 + 0.50 * 0.60 + 1 * 0.40 = 0.80.

The fuzzification, inferencing and defuzzification process can conveniently be encapsulated in a two- or three-dimensional plot. For example, Quantification of I^{BD} using I^{SH} and I^{BV} values can be represented in a three-dimensional plot as depicted in Figure 4.6. Similarly, quantification of risk index I^R values using I^{BD} and I^{IE} values can be expressed in the form of risk contours, as illustrated in Figure 4.7.

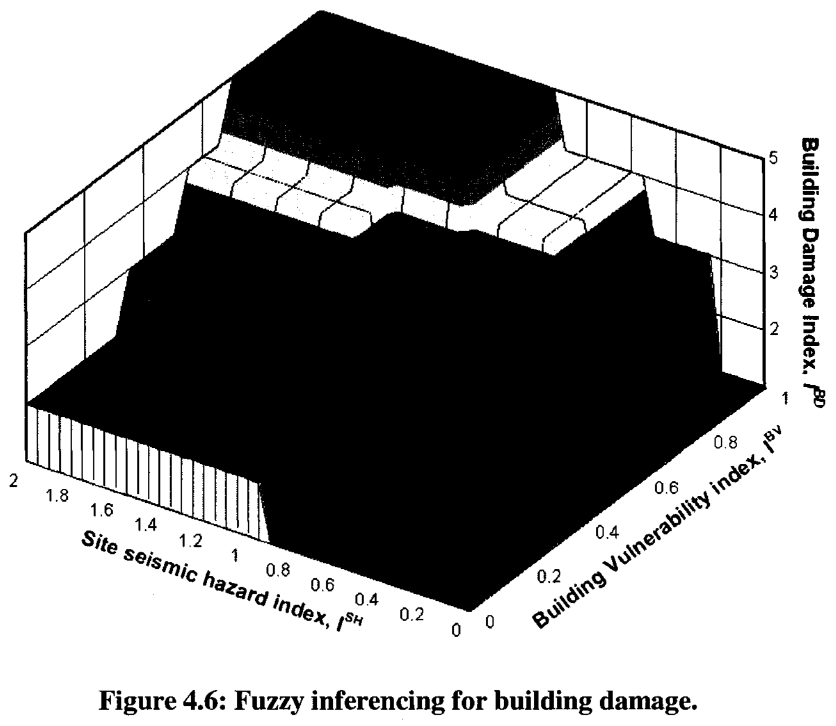


Figure 4.6: Fuzzy inferencing for building damage.

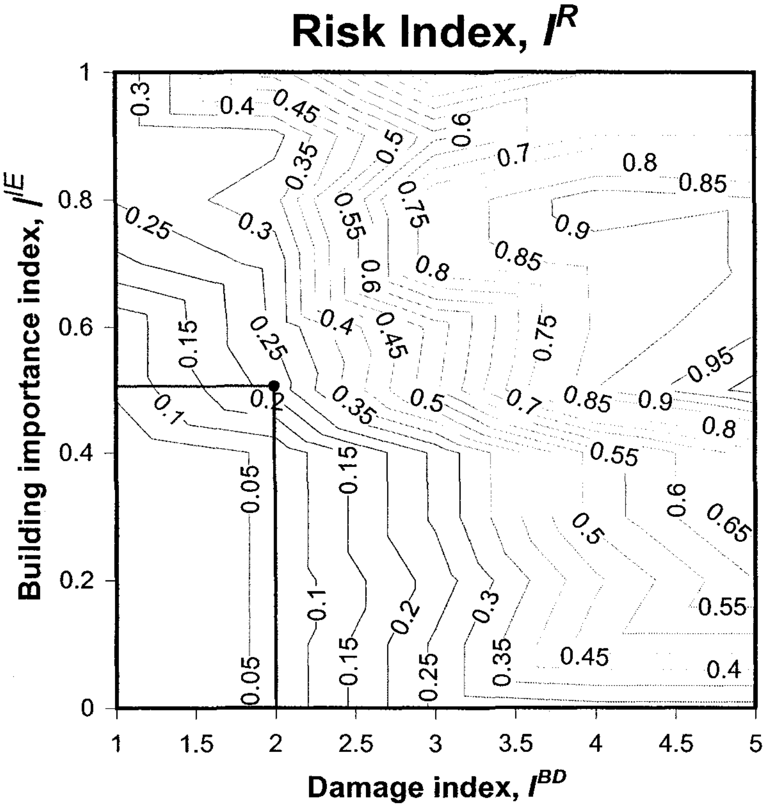


Figure 4.7: Risk contour index I^R.

4.1.6 Hierarchical fuzzy rule base modeling

The FRB modelling discussed so far presumes that at each node, indvidual input parameters are defined at the same level. With increasing number of inputs, however, the number of rules to be defined increases exponentially. This is referred toas the *curse of dimensionality,* and causes problems related to computational effort, real time performance, and system definition (Torra 2002).

To deal with the curse of dimensionality, several alternatives have been presented; such as identification of functional relationships, sensory fusion, rule hierarchy, and interpolation (Torra 2002). The rule hierarchy decomposes at the level of the rules, whereas Magdalena (2002) has shown decomposition at the level of the variables. Let us assume that, for example, there are four inputs V_1, V_2, V_3, and V_4, each of which is defined through three-tuples fuzzy sets. Therefore the total number of rules that should be defined is $3^4 = 81$. Using the concept of rule hierarchy depicted in Figure 4.8, the rule base required can be minimized. From Figure 4.8a for example, the number of rules requires are RB11 $= 3^2 = 9$, RB12 $= 3^3 = 27$. Hence, in total, 36 rules are defined and this is a significant reduction from the flat FRB of 81 rules. Figure 4.8b and Figure 4.8c show the use of a total of 27 rules.

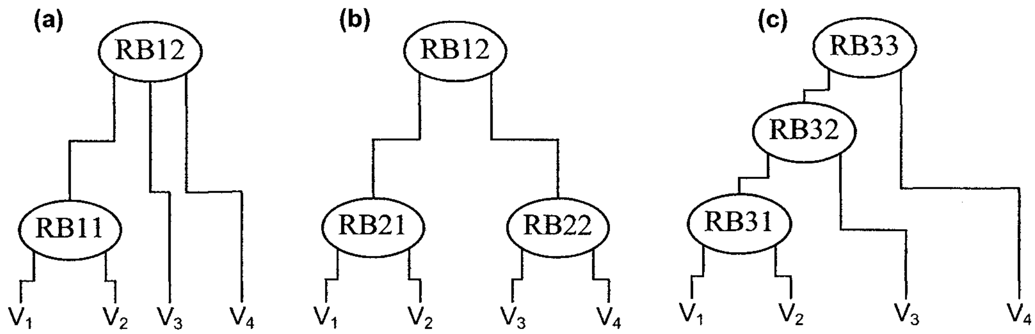


Figure 4.8: Hierarchical fuzzy rule base decomposition.

With the hierarchical rule base structure, the concept of fuzzfication and defuzzification is of importance. At each level of the hierarchical FRB inferencing, the output is furnished in three-tuple fuzzy sets. The three-tuple fuzzy sets can be defuzzified and crisp singleton is obtained. In turn, this singleton is fuzzified into next level. With each defuzzification step, there is a potential for loss of information. Alternatively, the three-tuple fuzzy sets output can be used as an input to the next FRB. With this procedure, there is a potential for fuzziness explosion. In general the inclusion or exclusion of defuzzification step is the problem of finding a trade-off between loss and explosion of fuzziness (Torra 2002). In the hierarchical structure shown in Figure 3.2 and Figure 5.2, for each intermediate FRB formulation, a defuzzification step is introduced.

4.1.7 Fuzzy uncertainty quantification

In quantifying the building damageability index, for example, the maximum membership value is used as a surrogate indicator of prevalent damage. For example, for site seismic hazard index I^{SH}= 1, and building vulnerability index I^{BV} = 0.70 and 0.75, the corresponding building damageability membership values can be computed to be[6] $\left(\mu_{N-S}^{BD}, \mu_{L}^{BD}, \mu_{M}^{BD}, \mu_{H}^{BD}, \mu_{M-D}^{BD}\right)$: (0, 0.333, 0.333, 0.364, **0.636**) and (0, 0, 0, 0.364, **0.636**), respectively. For both cases, μ_{M-D}^{BD} has the highest value, consequently, the damage is classified as *Major-Destroyed.* However, the crisp determination of damage doesn't consider the spread of the memberships, which can easily be discerned from the two values. The spread of fuzzy membership can be quantified through a *measure of fuzziness*, which estimates the average ambiguity in fuzzy sets (Pal and Bezdek 1994).

Pal and Bezdek (1994) outlined five criteria that need to be satisfied in order to qualify as a measure of fuzziness: i) sharpness, ii) maximality, iii) resolution, iv) symmetry, and v) valuation. For brevity, discussions on these conditions are not repeated here. A concept of Shanon's Entropy H based method can be used to quantify the fuzziness measures (Deluca and Termini 1972):

$$H = -K\sum_{i=1}^{n} \mu_i Log(\mu_i) + (1-\mu_i)Log(1-\mu_i) \qquad \textbf{(4.4)}$$

where K is a normalization factor. The Shanon's Entropy H is bounded between 0 and 1, $H \in$ [0, 1] that corresponds with the *minimum* and *maximum* spread of fuzziness, respectively. The minimum obtainable K value is = 1/1.51, which is obtained by considering μ_i = 0.5 (i =1,2,...,5). Using the building damageability membership values, e.g. (0, 0.333, 0.333, 0.364, **0.636**), and Eq. (4.4), the H can be calculated as:

---

[6]Note that the overall aggregation process will be discussed in the illustrative example section.

H = 1/1.51 [(0 log(0) + (1-0) log(1-0))+ (0.333 log(0.333) + (1-0.333) log(1-0.333))+…+ (0. 636 log(0. 636) + (1-0. 636) log(1-0.636))] = 0.75.

Similarly, for the second building damageability membership values, (0, 0, 0, 0.364, **0.636**), H = 0.38. Indeed, the spread of membership values quantified through H follows our intuitive observation of the given results, where the first case shows more spread than the latter. In final linguistic determination of the risk index, threshold values should be specified in order to determine acceptability of the building damageability index. This process is already illustrated in Section 3.1.5, under risk index.

4.2 Model calibration

The transformation values for VI, PI, and CQ, and q_i values for $\mathbf{R_1}$, $\mathbf{R_2}$, $\mathbf{R_3}$ and $\mathbf{R_4}$ are calibrated for the 1994 Northridge Earthquake damage database. The calibration is done by comparing observed ($I_i^{BD'}$) and estimated or computed (I_i^{BD}) building damage indices. For N_B = 73 training datasets, the optimization is carried out by minimizing the root mean squared error (RMSE):

$$RMSE = \sqrt{\frac{1}{N_B}\sum_{i=1}^{N_B}\left(I_i^{BD'} - I_i^{BD}\right)^2} \tag{4.5}$$

subject to:

VI: $0.01 < no < 0.51$; $0.49 < yes < 0.99$;

PI: $0.01 < no < 0.51$; $0.49 < yes < 0.99$;

CQ: $0.51 < poor < 0.99$; $0.25 < average < 0.75$; $0.01 < good < 0.50$.

q_i (values): $0 < q_1 < 0.25$; $0.25 < q_2 < 0.7$; $0.7 < q_3 < 1$

The optimization problem is performed through Microsoft Excel solver. After the training, the RMSE obtained is 0.752. The final transformation values are:

VI→ (*yes* = 0.80, *no* = 0.10),

PI→ (*yes* = 0.80, *no* = 0.20),

CQ→ (*poor* = 0.99, *average* = 0.70 and *good* = 0.01),

and corresponding q_i values are

q_i for $\mathbf{R_1} \rightarrow q_i$ (i = 1,2,3) = (0.25, 0.5, 1),
q_i for $\mathbf{R_2} \rightarrow q_i$ (i = 1,2,3) = (0.1, 0.6, 0.7),
q_i for $\mathbf{R_3} \rightarrow q_i$ (i = 1,2,3) = (0.1, 0.6, 0.9) and
q_i for $\mathbf{R_4} \rightarrow q_i$ (i = 1,2,3) = (0.01, 0.5, 0.9).

The proposed risk analysis procedure is generic; however, in a case where there is historical damage database, the Tier 1 model can be calibrated for the specific database.

4.3 Illustrative example

A step by step aggregation through the seismic risk analysis hierarchical structure is illustrated below. The illustration is carried out for Building ID = CDMG231-GZ-02 (Table 3.10). From the 1994 Northridge Earthquake, the building is identified to be in a damage state of *Light*. From the *walk down* survey, the performance modifiers (as indicated in the hierarchical structure of Figure 3.2) are:

Building Structural system (BSS) = Shear wall building (*C2*),
Building use (BU) = School,
Vertical irregularity (VI) = *yes*,
Plan irregularity (PI) = *yes*,
Construction quality (CQ) = *Good*,
Year of construction (YC) = 1957,
Number of stories (N) = 4.

Step 1: The first step of the evaluation process is transforming the performance modifiers into a commensurable unit, which can be shown to be:

BSS = 0.25;
BU = 0.5;
VI = 0.80;
PI = 0.80;
CQ = 0.01;
YC = (-0.01*1957 + 20.25) = 0.68, Eq. (3.2);
T_1 = 0.40, Eq. (3.1b), for N = 4;
$S_a(T_1)$ = 0.37, from the 1994 Northridge Earthquake response spectrum.

Step 2: Fuzzify the transformed values using corresponding granules:

$$\left(\mu_L^{VI}, \mu_M^{VI}, \mu_H^{VI}\right) = (0, 0.40, 0.60);$$

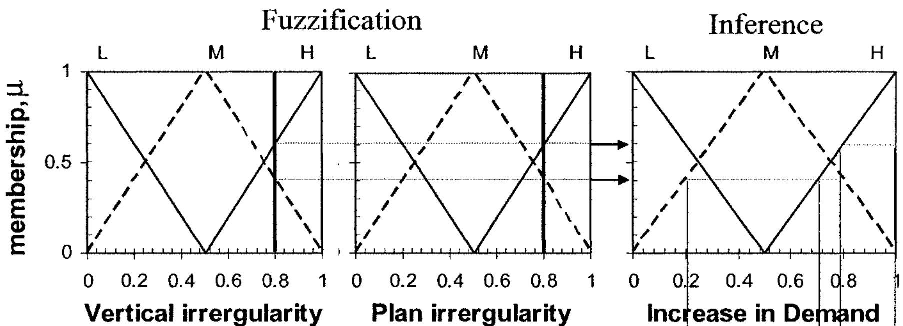
$$\left(\mu_L^{PI}, \mu_M^{PI}, \mu_H^{PI}\right) = (0, 0.40, 0.60);$$

$$\left(\mu_L^{CQ}, \mu_M^{CQ}, \mu_H^{CQ}\right) = (0.98, 0.02, 0);$$

$$\left(\mu_L^{YC}, \mu_M^{YC}, \mu_H^{YC}\right) = (0, 0.54, 0.46);$$

$$\left(\mu_L^{BSS}, \mu_M^{BSS}, \mu_H^{BSS}\right) = (0.50, 0.50, 0).$$

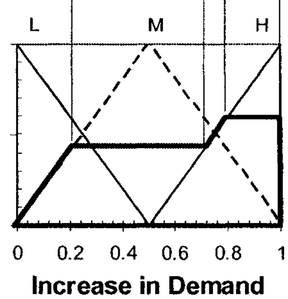
Step 3: The hierarchical fuzzy inferencing is performed using $\mathbf{R}_1$ to $\mathbf{R}_7$. The inferencing is performed as a bottom up, which starts with $\mathbf{R}_1$ and $\mathbf{R}_2$. A detailed inferencing procedure is previously illustrated in Section 4.1.4: inference mechanism. The FRB aggregation of $\mathbf{R}_1$ is schematically illustrated below. As shown in this figure, inferencing of $\mathbf{R}_1$ for VI and PI furnishes the ID = 0.80.



Fuzzy rule base $\mathbf{R}_1$ for ID

Rule *i*	VI	PI	ID
1	L	L	L
2	L	M	L
3	L	H	M
4	M	L	L
5	M	M	M
6	M	H	H
7	H	L	M
8	H	M	H
9	H	H	H

VI = vertical irregularity; PI = plan irregularity; ID = Increase in demand



Defuzzification

ID = 0.25*0 + 0.5*0.4+1*0.6 = 0.80

Further illustration of quantifying DR using $\mathbf{R_2}$ is provided. For example, using the rule base provided in Table 4.6, the inferencing to quantify DR is illustrated. Using the fuzzy rule of composition (Eqs. 4.2 and 4.3) and $\mathbf{R_2}$ (Table 4.6), DR is computed to be:

$$DR = \begin{array}{ll} \mu_L^{DR} = max(min(0.98,0), min(0.98,0.53), min(0.02,0)) & = 0.54 \\ \mu_M^{DR} = max(min(0.98,0.46), min(0.02,0.54), min(0,0)) & = 0.46 \\ \mu_H^{DR} = max(min(0.02,0.46), min(0,0.54), min(0,0.46)) & = 0.02 \end{array}$$

Step 4: Using the quality-ordered weights factors, q_i (i = 1,2,3) = (0.1, 0.6, 0.7), the DR is defuzzified as:

$$DR = \sum_{i=1}^{n} q_i * \mu_i = 0.1 \times 0.53 + 0.6 \times 0.47 + 0.7 \times 0.02 = 0.35$$

The ID and DR are fuzzified and inferencing through $\mathbf{R_3}$ results in the SD of 0.60. The I^{BV} = 0.71 is obtained through inferencing of $\mathbf{R_4}$. Inferencing through $\mathbf{R_1}$ to $\mathbf{R_5}$ and $\mathbf{R_7}$ requires defuzzification. Whereas, for $\mathbf{R_6}$, the building damageability, the maximum membership is used to assign the corresponding damage state, which will be illustrated below.

Fuzzification of the I^{BV} and I^{SH} are:

$\left(\mu_{VL}^{BV}, \mu_L^{BV}, \mu_M^{BV}, \mu_H^{BV}, \mu_{VH}^{BV}\right)$ = (0, 0, 0.30, 0.70, 0)

$\left(\mu_{VL}^{SSH}, \mu_L^{SSH}, \mu_M^{SSH}, \mu_H^{SSH}, \mu_{VH}^{SSH}\right)$ = (0.08, 0.93, 0, 0, 0)

and inferencing through $\mathbf{R_6}$, membership to BD is:

$\left(\mu_{N-S}^{BD}, \mu_L^{BD}, \mu_M^{BD}, \mu_H^{BD}, \mu_{M-D}^{BD}\right)$ = (0.08, **0.70**, 0, 0, 0).

The corresponding entropy H, Eq. (4.4), can be shown to be H = 0.19. Since the maximum membership is associated with μ_L^{BD}, the I^{BD} is classified as "Light." Given the I^{SH} and I^{BV} indices, from Figure 4.6, the same damage level can be inferred. If sufficient information is provided to compute the importance/exposure index, I^{IE}, the I^R can also be obtained from Figure 4.7. For example, if I^{IE} = 0.5, and I^{BD} = "Light" = 2, from Figure 4.7, the I^R = 0.225. In the final decision making process, it can be shown that for I^R = 0.225 and H = 0.19, the building is quantified as *negligible* (Table 3.3), which requires no further investigation.

4.4 Case study

The Northridge Earthquake with a moment magnitude M_w=6.7 struck the San Fernando Valley on January 17, 1994. Because of its proximity to communities in the Los Angeles basin, there was tremendous damage (EERI 1994). The Northridge earthquake has highlighted the importance of economic consequences of failure (Elms 2004). This earthquake and the ATC-38 (ATC 2001) building performance and strong motion data have been adopted for the case study presented in this section to demonstrate the application of the proposed risk-based analysis procedure.

4.4.1 Data structures

The three building type considered, *C1*, *C2* and *C3* are categorized into flexible and rigid diaphragms, however, for the purpose of this case study; all buildings are considered as rigid diaphragms (i.e., no distinction is made on the type diaphragms). The corresponding building modifiers (summarized in Figure 3.5) and strong-motion data are obtained from the ATC-38 database. "Discontinuous columns" and "plan setbacks" are used as a surrogate measure of the vertical irregularity, i.e. if either one of them is selected as *yes*, vertical irregularity = *yes*. Similarly, "open front plan," "other torsional imbalance," and "plan irregularities" are used as a surrogate measure of plan irregularity. Figure 3.5 also show histogram of performance modifiers and corresponding damage observed. The spectral acceleration $S_a(T_1)$ is computed using the period given in Eqs. (4a) and (4b) and corresponding response spectrum reported in the ATC-38 database.

4.4.2 Model development, validation and calculation of damage indices

The model calibration is performed as outlined in the previous section. Of the 93 available data sets, 73 and 20 randomly selected data sets are used for calibration (model development) and testing (model validation), respectively. Table 3.10 shows the data inputs used for model validation and the computed I^{BD} and I^R values. The computed I^{BD} and observed building damage of the model testing and validation are plotted in Figure 4.9 and Figure 4.10, respectively. Most of the observed damages are in the lower damage states, as such the model training and calibration will be slightly biased. However, there is good agreement between the observed and predicted damage states. Most of the miss classifications observed are over or under predicting by one state. In Figure 4.11 and Figure 4.12 the estimated damage and corresponding entropy are shown for model training and validation, respectively. In Figure 4.11 and Figure 4.12, the entropy is bounded between 0.1 and 0.4, however, in Figure 4.11, for three cases, the entropy is close to 0.7.

The risk index I^R value is computed through the fuzzy rule base modeling, and using the risk index I^R thresholds summarized in Table 3.3, the results can be categorized into *Negligible* and *Marginal*. It should be noted that the I^{IE} value is computed by considering only the *building use*, and the I^{IE} value may change when other modifiers are considered.

In a risk-based prioritization, the estimated damage states are used to compute I^R values and the uncertainty in the building damageability estimation are quantified through entropy. These coupled values are used for prioritization of repair and upgrade or to judge whether if further assessment is warranted. Two sets of prioritization, using I^{BD} and I^R, are summarized in Table 3.10. Table 3.10 shows that with the risk-based prioritization, the priorities changes and there is a rank reversal.

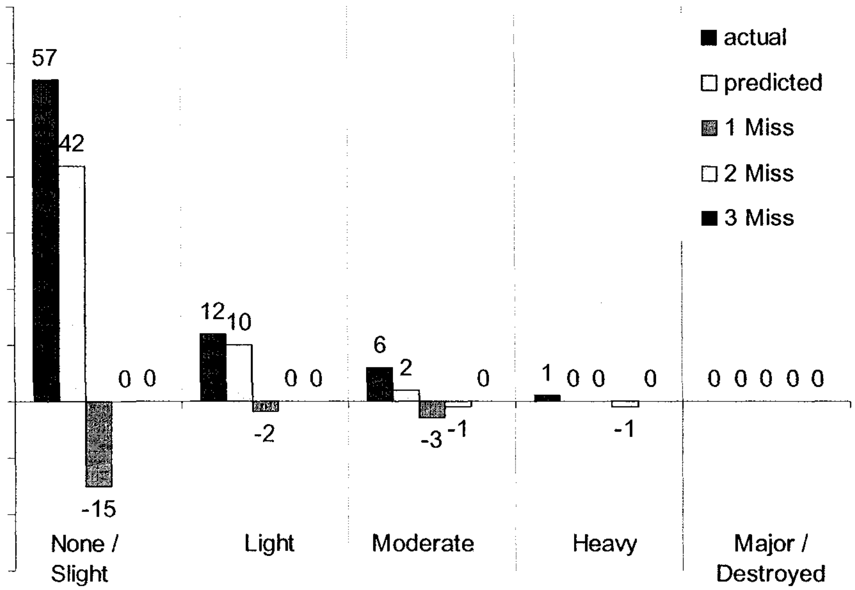


Figure 4.9: data building damageability index I^{BD} (model training).

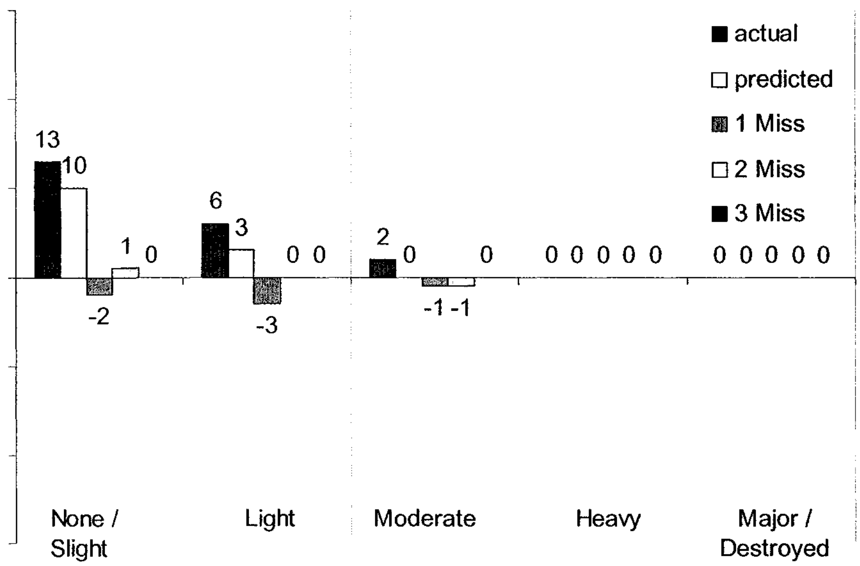


Figure 4.10: data building damageability index I^{BD} (model validation).

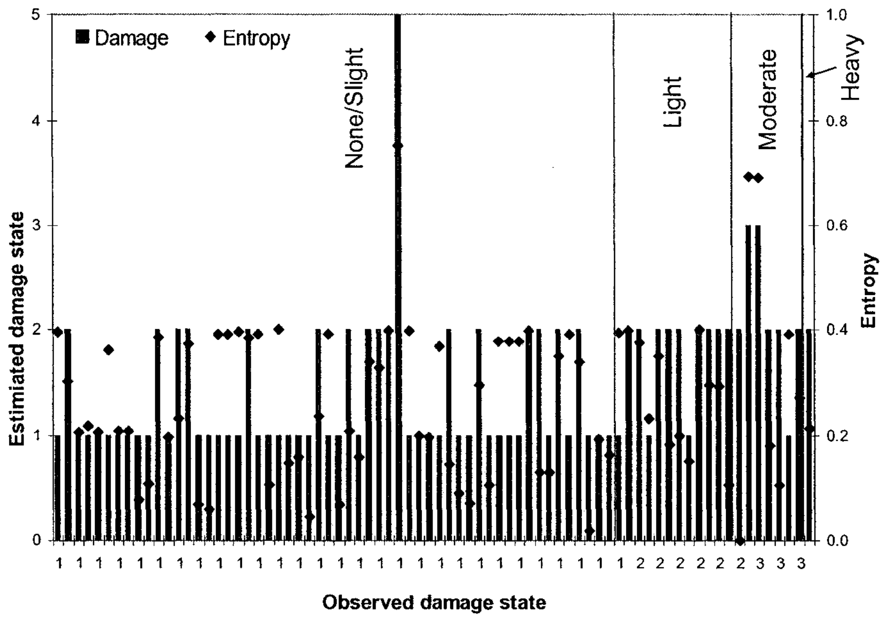


Figure 4.11: building damageability and entropy (model training).

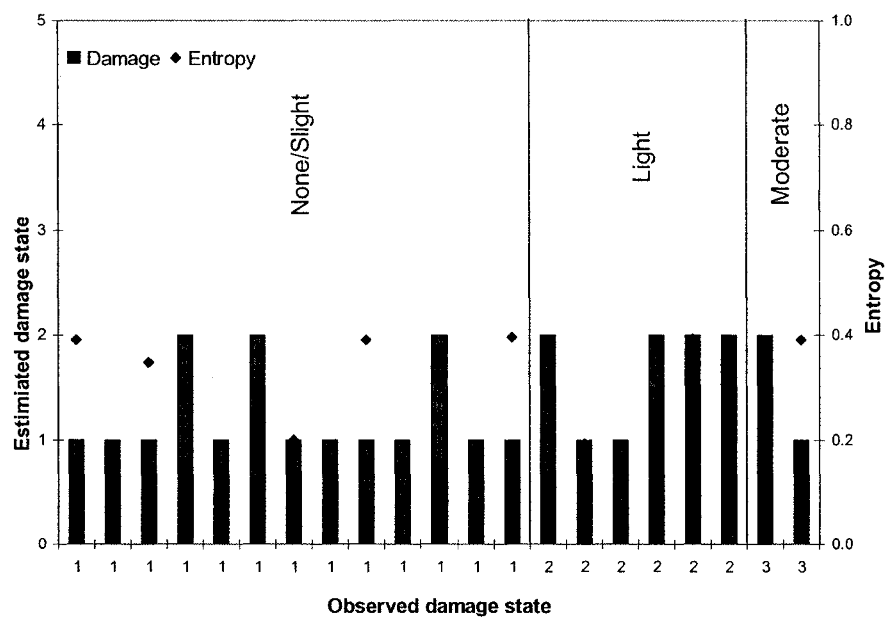


Figure 4.12: building damageability and entropy (model training).

4.5 Conclusion

Seismic risk analysis of RC buildings is used as an impetus for risk management framework. The seismic risk analysis should incorporate the engineering decision making aspects (e.g. damage quantification) and societal values (e.g. tolerance to the consequence of failure). In this chapter, a Tier 1 model for seismic risk analysis of RC building assessment is presented. The proposed risk-based prioritization is undertaken by integrating performance modifiers of site seismic hazard, building vulnerability, and importance/exposure factor. The performance modifiers are obtained form a *walk down* survey, which is prone to subjectivity, consequently, vagueness uncertainty is prevalent. The vagueness uncertainty is quantified through fuzzy set theory and the decision maker's attitude is incorporated in the aggregation process through the FRB. The proposed Tier 1 model is programmed into Microsoft Excel computer program called CanRisk. A screen snapshot and functionality of CanRisk is illustrated in Appendix B.

The proposed hierarchical seismic risk analysis procedure is validated using 1994 Northridge Earthquake observed damages. Results of the proposed risk-base prioritization method show good correlation with observed damage, albeit extracted from limited data sets. Most of the 1994 Northridge earthquake RC building damages are in the lower damage states, as such the model training and calibration can slightly be biased towards the lower damage states. However, for more extensive model calibration and validation, the data need to be augmented with analytical data to cover the gap on higher levels of damage. Furthermore, in order to generalize the proposed technique, further calibrations with different databases are warranted.

The uncertainty in the quantification of damage states is quantified through Shannon's Entropy concept. The final decision making is made in terms of four linguistic constants: *negligible*, *marginal*, *critical* and *catastrophic*. The risk index ($I^R \in [0, 1]$) coupled with Shannon's Entropy ($H \in [0, 1]$) are used to quantify these linguistic constants. A risk analysis result of *critical* and *catastrophic* can be used as a decision in favour of Tier 2 evaluation. The Tier 2 method is discussed in the next chapter.

Chapter 5
Tier 2 Model for Seismic Risk Analysis of RC Buildings

"Divide each difficulty into as many parts as is feasible and necessary to resolve it."

Rene Descartes

In the Tier 1 model (Chapter 3), a risk analysis framework was proposed that incorporates building performance modifiers in congruence with FEMA 154. Most of the information in the Tier 1 framework is solicited from a *walk down* survey. This chapter presents an extension of the building vulnerability module which incorporates detailed performance modifiers as specified in FEMA 310 (Table 2.4). Table 2.4 shows the information solicited in FEMA 310, which is categorized into basic structural checklist (column 2) and supplementary structural checklist (column 3). Henceforth the proposed seismic risk analysis of RC buildings will be referred to as Tier 2 model. The input information in the Tier 2 model entails values obtained from simple structural calculations and a *walk down* survey. For example, whether or not a soft story exists in inferred through computations involving relative stiffness of two adjacent floors. In turn, the acceptable soft story threshold limit can be obtained from the current building codes, FEMA 310 (FEMA 1998), NEHARP 2003 (FEMA 450-1 2003) and reported analytical works. The proposed Tier 2 model is depicted in Figure 5.1 and details of the building vulnerability are provided Figure 5.2. The $\mathbf{R_5}$, $\mathbf{R_6}$ and $\mathbf{R_7}$ shown in Figure 5.1 are as specified in the Tier 1 model, and are provided in Table 4.9, Table 4.10 and Table 4.11, respectively. The evaluation process of seismic risk analysis is illustrated inFigure 1.2 and Figure 3.1.

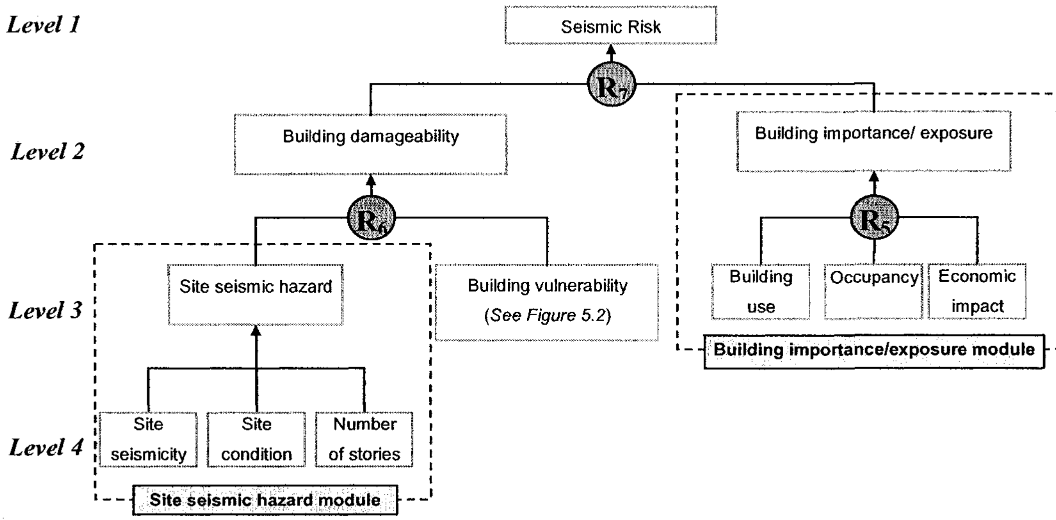


Figure 5.1: Seismic risk analysis of RC buildings.

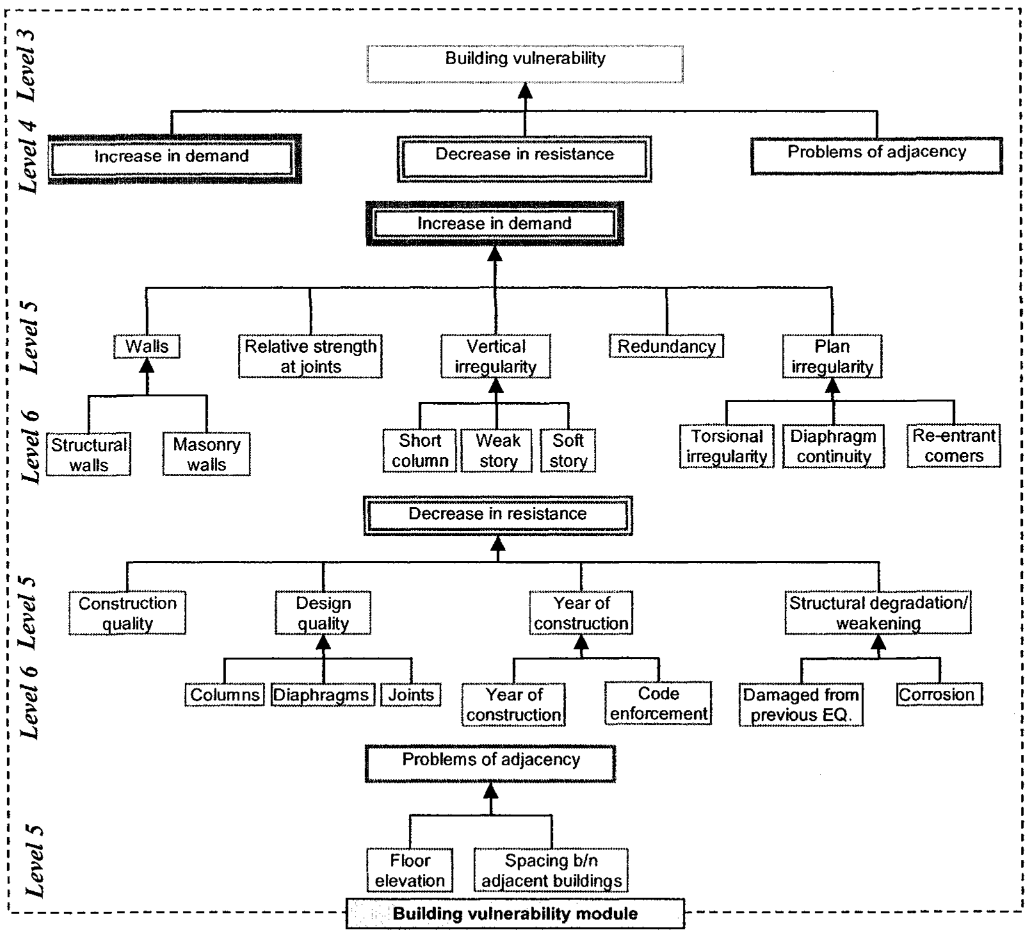


Figure 5.2: Detailed RC buildings vulnerability assessment.

5.1 Building vulnerability module

In the Tier 1 model (Chapter 3), building structural system is explicitly considered as a performance modifier. However, in the Tier 2 model, the lateral load resistance system is quantified through a parameter identified as *walls* and is integrated into the hierarchical structures of increase in demand.

Any structural deficiency compromises the seismic capacity of the RC buildings. The different performance modifiers that impact structural deficiency are incorporated in the hierarchal structures to quantify building vulnerability. Building vulnerability (shown at *level 3* in Figure 5.1 and Figure 5.2) to ground shaking and associated damage can be grouped into two categories (Saatcioglu *et al.* 2001); factors contributing to an increase in seismic demand (e.g., soft story frame, weak column-strong beam, vertical irregularities); and factors contributing to reduction in ductility and energy absorption capacity (e.g., construction quality, year of construction, structural degradation). The *increase in demand* and *decrease in resistance* are inherent system properties and can be described as *endogenous parameters*. Further, the building vulnerability can be affected through an exogenous parameter, such as *problem of adjacency*. The problem of adjacency can simultaneously contributes to an increase in demand and decrease in resistance of the building.

Modelling the interactions between these parameters is complex, and often full fledged nonlinear structural analysis is required. In the Tier 2 model, an expert and intuitive knowledge is used to develop the building vulnerability hierarchical structure shown in Figure 5.2, and the uncertainty propagation is performed through FRB. Aggregating through the hierarchical structures is finally used to compute the building vulnerability index I^{BV}. The hierarchical FRB used in the computation of building vulnerability is shown in Figure 5.3.

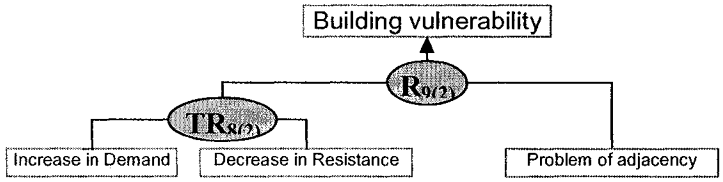


Figure 5.3: Hierarchical rule base for building vulnerability.

Each of the performance modifier used in the computation of I^{BV} are further expounded below. The **TR**$_{8(2)}$ and **R**$_{9(2)}$ FRB are summarized and Table 5.1 and Table 5.2, respectively. The two in bracket (2) is used to indicate FRB for the Tier 2 model and the prefix T stands for *temporary* FRB.

Table 5.1: Fuzzy rule base for endogenous effects on building vulnerability

$\mathbf{TR_{8(2)}}$	Rule i	ID	DR	Endogenous parameter
	1	L	L	L
	2	L	M	L
	3	L	H	M
	4	M	L	L
	5	M	M	M
	6	M	H	H
	7	H	L	M
	8	H	M	H
	9	H	H	H

ID = Increase in demand; DR = Decease in resistance

Table 5.2: Fuzzy rule base for building vulnerability

$\mathbf{R_{9(2)}}$	Rule i	Endogenous parameter	Problem of adjacency	Building vulnerability
	1	L	L	L
	2	L	M	L
	3	L	H	M
	4	M	L	L
	5	M	M	M
	6	M	H	H
	7	H	L	M
	8	H	M	H
	9	H	H	H

5.1.1 Increase in demand

The fuzzy hierarchical rule base used to compute increase in demand is shown in Figure 5.4. The rule hierarchies shown in Figure 5.4 are developed by considering the functional relation of each performance modifier. The structural weakness in the vertical elements, *weak story* (WS) and *soft story* (SS) are aggregated through FRB $\mathbf{TR_{1(2)}}$ (Table 5.3). Output of $\mathbf{TR_{1(2)}}$ is aggregated with *short column effects* (SCE) through $\mathbf{R_{1(2)}}$ (Table 5.4) to obtain *vertical irregularity*. Response of the lateral load resistance is affected by the *relative strength of joints*. Thus, results of the vertical irregularity are aggregated with relative strength at the joints through $\mathbf{TR_{4(2)}}$ (Table 5.5). The response of the vertical element is modified by contribution of *walls* and *redundancy*. These effects are aggregated through $\mathbf{TR_{3(2)}}$ (Table 5.6), and results of $\mathbf{TR_{3(2)}}$ and $\mathbf{TR_{4(2)}}$ are aggregated through $\mathbf{TR_{5(2)}}$ (Table 5.7).

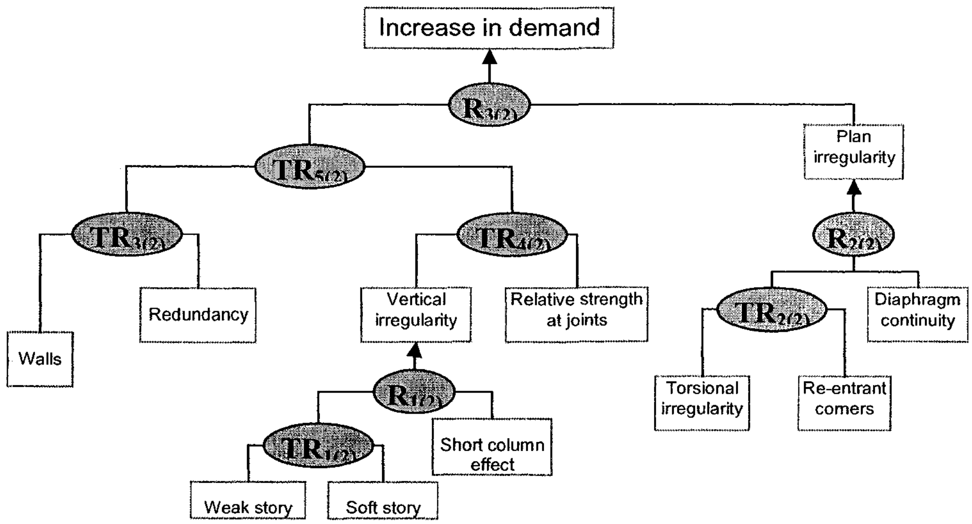


Figure 5.4: Hierarchical rule base for increase in demand.

Table 5.3: Fuzzy rule base for temporary node of vertical irregularity

$TR_{1(2)}$	Rule i	WS	SS	Temp node 1 f(WS, SS)
	1	L	L	L
	2	L	M	L
	3	L	H	M
	4	M	L	L
	5	M	M	M
	6	M	H	H
	7	H	L	M
	8	H	M	M
	9	H	H	H

WS = Weak story; SS = Soft story

Table 5.4: Fuzzy rule base for vertical irregularity

$R_{1(2)}$	Rule i	$TR_{1(2)}$	SCE	Vertical irregularity
	1	L	L	L
	2	L	M	L
	3	L	H	M
	4	M	L	M
	5	M	M	M
	6	M	H	H
	7	H	L	H
	8	H	M	H
	9	H	H	H

SCE = Short column effects

Table 5.5: Fuzzy rule base for $TR_{4(2)}$

$TR_{4(2)}$	Rule i	VI	RSJ	Temp node 4 f(VI, RSJ)
	1	L	L	L
	2	L	M	M
	3	L	H	M
	4	M	L	L
	5	M	M	M
	6	M	H	H
	7	H	L	M
	8	H	M	H
	9	H	H	H

VI = Vertical irregularity; RSJ = Relative strength at the joints

Table 5.6: Fuzzy rule base for $TR_{3(2)}$

$TR_{3(2)}$	Rule i	Walls	Red.	Temp node 3 f(Walls, Red.)
	1	L	L	L
	2	L	M	L
	3	L	H	M
	4	M	L	L
	5	M	M	M
	6	M	H	H
	7	H	L	M
	8	H	M	H
	9	H	H	H

Red. = Redundancy

Table 5.7: Fuzzy rule base for $TR_{5(2)}$

$TR_{5(2)}$	Rule i	$TR_{3(2)}$	$TR_{4(2)}$	Temp node 5 $f(TR_{3(2)}, TR_{4(2)})$
	1	L	L	L
	2	L	M	L
	3	L	H	M
	4	M	L	L
	5	M	M	M
	6	M	H	H
	7	H	L	M
	8	H	M	H
	9	H	H	H

The building performance is also modified by plan irregularity. The plan irregularity arises from *torsional irregularity*, *re-entrant corners* and *diaphragm continuity*. The *torsional irregularity* and *re-*entrant *corners* are aggregated through $\mathbf{TR_{2(2)}}$ (Table 5.8). The results of $\mathbf{TR_{2(2)}}$ are aggregated with *diaphragm continuity* through $\mathbf{R_{2(2)}}$ (Table 5.9) to obtain *plan irregularity.* Finally, the plan irregularity and results of $\mathbf{TR_{5(2)}}$ are aggregated through $\mathbf{R_{3(2)}}$ (Table 5.10) to obtain *increase in demand.*

Table 5.8: Fuzzy rule base for $TR_{2(2)}$

$TR_{2(2)}$	Rule *i*	TI	REC	Temp node 2 *f*(TI, REC)
	1	L	L	L
	2	L	M	M
	3	L	H	M
	4	M	L	L
	5	M	M	M
	6	M	H	H
	7	H	L	M
	8	H	M	H
	9	H	H	H

TI = Torsional irregularity; REC = Re-entrant corners

Table 5.9: Fuzzy rule base for plan irregularity

$R_{2(2)}$	Rule *i*	$TR_{2(2)}$	DC	Plan irregularity
	1	L	L	L
	2	L	M	L
	3	L	H	M
	4	M	L	M
	5	M	M	M
	6	M	H	H
	7	H	L	H
	8	H	M	H
	9	H	H	H

DC = Diaphragm continuity

Table 5.10: Fuzzy rule base for increase in demand

$R_{3(2)}$	Rule i	$TR_{5(2)}$	PI	Increase in Demand
	1	L	L	L
	2	L	M	M
	3	L	H	M
	4	M	L	L
	5	M	M	M
	6	M	H	H
	7	H	L	M
	8	H	M	H
	9	H	H	H

PI = Plan irregularity

The description of building performance modifiers and corresponding fuzzification that contribute to an increase in demand are discussed below.

Vertical irregularity (VI)

The earthquake induced inertia load is transferred from the floors, where most of the mass is concentrated, to the foundation through the lateral load resisting system. A good design practice avoids discontinuities and/or abrupt changes in this load path so that localized stress concentrations are avoided. The vertical irregularity parameter reflects the presence of discontinuity and/or abrupt change in strength and stiffness along the building height. The vertical irregularities considered in this thesis are *soft story* (SS), *weak story* (WS), and *short column effect* (SCE). Often, discontinuous column/or shear walls are classified as vertical irregularity, however, their effects manifest as soft story or weak story. Hence, in this thesis, discontinuous columns/ walls are not explicitly considered as a basic risk item of VI. Granulations of the SS, WS, and SCE are shown in Figure 5.5a-c, respectively. Derivation of these granulations is discussed within each subsection.

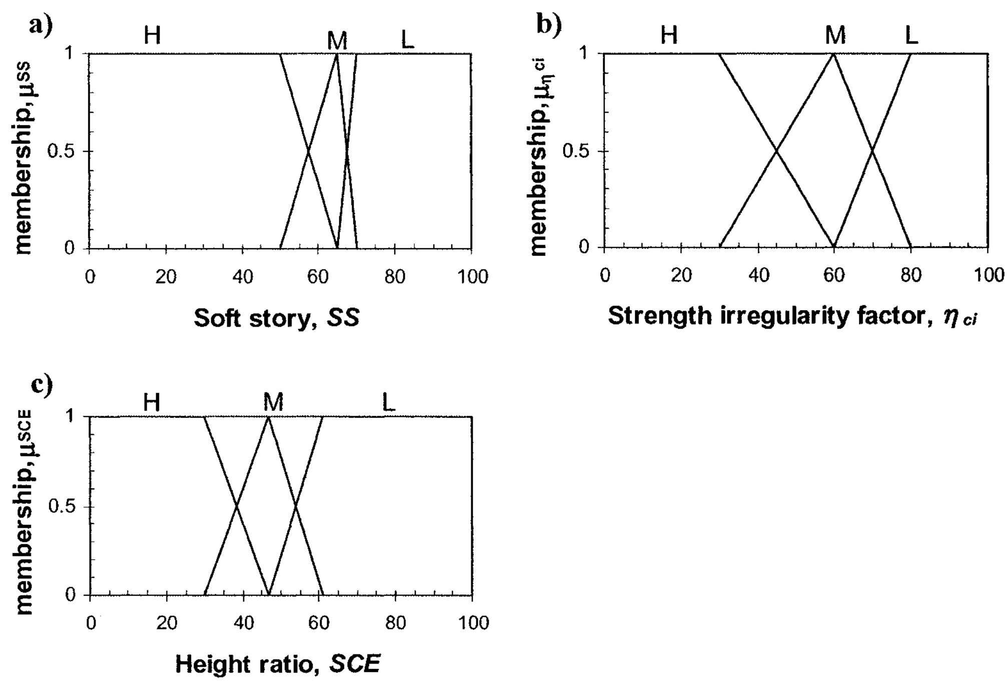
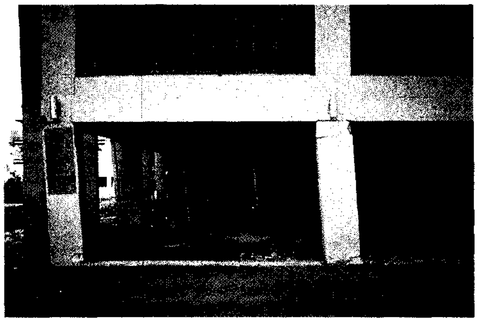


Figure 5.5: Granulation of the basic risk items used in a vertical irregularity, a) soft story, b) weak story, and d) short column effect.

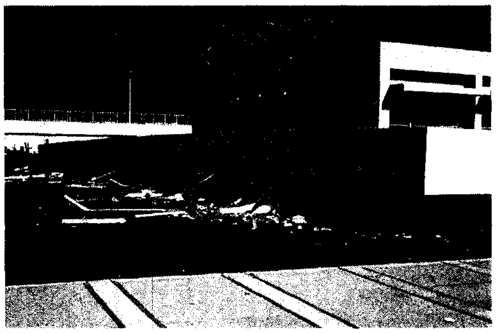
Soft story (SS)

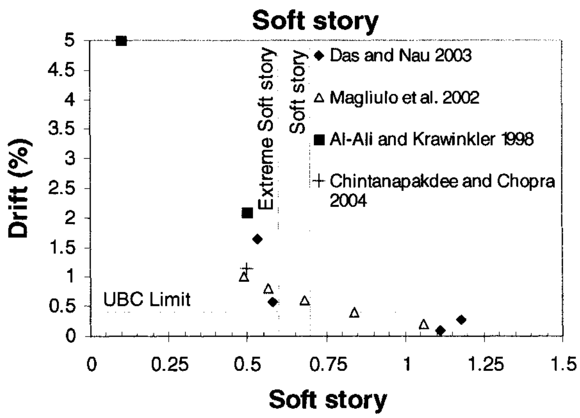
Soft story effect is introduced due to functional reasons that require opening in the first story or an intermediate story, Figure 5.6a-c (e.g., shops, restaurants, hotel lobbies, parking space). The soft story will deform more significantly than other floors, and if the demand exceeds the lateral load resisting system capacity, overall structural integrity of the building is undermined with ensuing localized damage or overall collapse of the building (Figure 5.6c). This excessive deformation could be a source of secondary failure, which may introduce a P-Δ effect. For extreme soft story case, the horizontal inertial force can induce rigid body type movement of the upper stories and shear off the lower floor columns.



a) Commercial Building Casa Micasa S.A., Managua, Nicaragua. This 2-story reinforced concrete frame building suffered significant lateral displacement at the second floor level during the 1972 Managua Earthquake.[§]

b) Olive View Hospital, San Fernando, California. Illustrating the damage that these buildings suffered during the 1971 San Fernando Earthquake. [§]





c) Olive View Hospital, Psychiatric Unit, San Fernando, California. 1971 San Fernando Earthquake. [§]

d) Soft story and corresponding drift.

([§]Photo credit: Godden Collection, Earthquake Engineering Library, University of California, Berkeley)

Figure 5.6: Soft story

A *SS* is present when the stiffness of the lateral force resisting system in any story is less than 70% of the stiffness in an adjacent story (above or below) or less than 80% of the average stiffness of the three stories (above or below) (FEMA 310 1998; FEMA 450-1 2003). Further, FEMA 450-1 (2003) specifies that an *extreme soft story* exists when the lateral force resisting system in any story has a stiffness that is less than 60% of the stiffness in an adjacent story (above or below). Reported parametric analyses, (Chintanapakdee and Chopra 2004, Das and Nau 2003, Magliulo *et al.* 2002 and Al-Ali and Krawinkler 1998), for example, have quantified the variation of SS on the drift demand of the ground floor (Figure 5.6d). The variation in the reported results of the drift is due to the different magnitude of earthquake loads considered, and differences in the column-beam stiffness model. For example, Chintanapakdee and Chopra (2004) and Das and Nau (2003) used strong-column-weak-beam configuration, whereas, Al-Ali and Krawinkler (1998) used weak-column-strong-beam configuration. However, Al-Ali and Krawinkler (1998) highlighted that the weak-column-strong-beam configuration is an upper bound (worst case scenario) solution. Figure 5.6d shows that the extreme soft story and soft story limits correspond with the UBC drift limit of 0.004 (0.4%). The percentage of soft story shown in Figure 5.6d is used for fuzzification of soft story (Figure 5.5a).

Weak story

Weak story is determined by the strength of the lateral force resisting system in any two adjacent stories. According to FEMA 310 a weak story exists when the strength of lateral force resisting system any story is less than 80% of the strength in an adjacent story (above or below). A weak story structure with story strength less than 65% of the story above is prohibited (Al-Ali and Krawinkler 1998). The aforementioned limits are assigned based on judgment. The potential for weak story can be computed through "strength irregularity factor" η_{ci} (Ministry of Public Works and Settlement 1998):

$$\eta_{ci} = (\sum A_e)_i \,/\, (\sum A_e)_{i+1} < 0.80 \tag{5.1}$$

where $\sum A_e$ = effective shear area of any storey, and the indices i and i+1 show two adjacent floors. The effective shear area is computed as (Ministry of Public Works and Settlement 1998):

$$\sum A_e = \sum A_w + \sum A_g + 0.15 \sum A_k \tag{5.2}$$

where $\sum A_w$ = Sum of effective web areas of column cross sections; $\sum A_g$ = Sum of section areas of structural elements at any storey behaving as structural walls in the direction parallel to the earthquake direction considered; and $\sum A_k$ = Sum of masonry infill wall areas (excluding door and window openings) at any storey in the direction parallel to the earthquake direction considered.

Valmundsson and Nau (1997) and Al-Ali and Krawinkler (1998), among others, have carried out parametric analysis on the impact of strength irregularity factor on ductility demand, the results obtained are summarized in Figure 5.7. Figure 5.7 comparison of the ductility demand in a weak first story building and that in a regular building. Both authors have used the weak-column-strong-beam configuration. From Figure 5.7, it can be discerned that, for a small strength reduction, the increase in ductility demand is significant. The weak story is far more damaging than soft story (Al-Ali and Krawinkler 1998). However, combination of weak and soft story has synergetic negative effect. Figure 5.7 shows that the FEMA 310 weak story ratio 80% cut off is not conservative. The strength irregularity factor shown in Figure 5.7 is used for the fuzzification of weak story (Figure 5.5b).

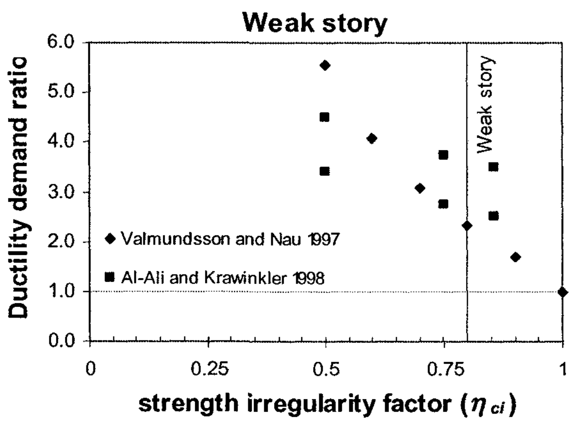


Figure 5.7: Weak story

Short column effect / captive columns (SCE)

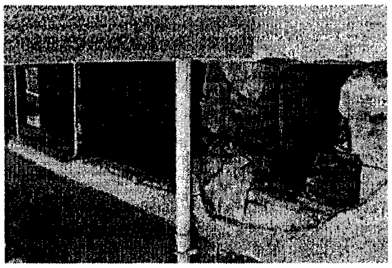
Under current seismic design practice, columns in earthquakes prone areas are designed to have high ductility. The ductility of a structure ensures there are enough distortions or deformations before severe damage or failure occurs. Addition of non structural components (such as partial-height frame infill; balcony parapets) inadvertently captivate the column and create short column effect. Consequently, the column becomes stiff, looses its ductility and attracts more loads. If the design has not considered this effect, shear demand tends to exceed the shear capacity of the column with ensuing damage (Figure 5.8a). However, the impact of short column effects often manifests as local column damage that may not necessarily be associated with global building damage. Prudence in the impact assessment of short column effect is essential. The relative location and number of short columns need to be assessed with respect to the overall lateral resisting elements.

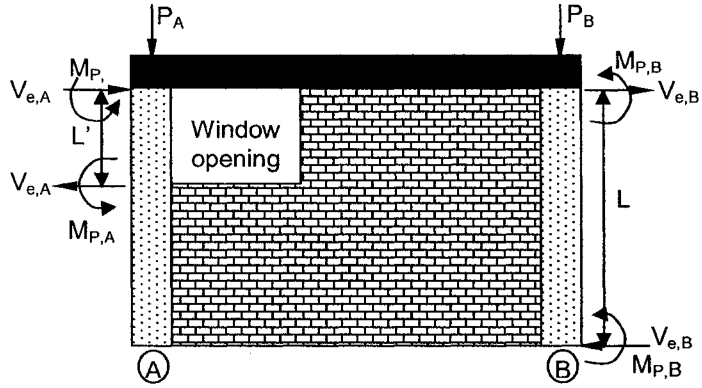
The captive column effect is schematically illustrated in Figure 5.8b. This figure depicts two columns, captivated column A and regular column B, with the corresponding length of L' and L, respectively. If we assume the probable plastic moment of the two columns being equal, i.e., $(M_{P,A} = M_{P,B} = M_P)$, the corresponding shear demand, V_e, is computed as

$$V_{e,A} = 2\frac{M_P}{L'},\ V_{e,B} = 2\frac{M_P}{L} \qquad (5.3)$$

The M_P in Eq. (5.3) is computed based on two assumptions: 1) the tension bar yield strength is assumed to be 1.25 times the specified minimum yield; 2) $\phi = 1.0$, i.e., there is no capacity reduction as required for design. From Eq. (5.3), it can be discerned that, with increasing captivated column effect, i.e. decreasing L', the shear demand increases.

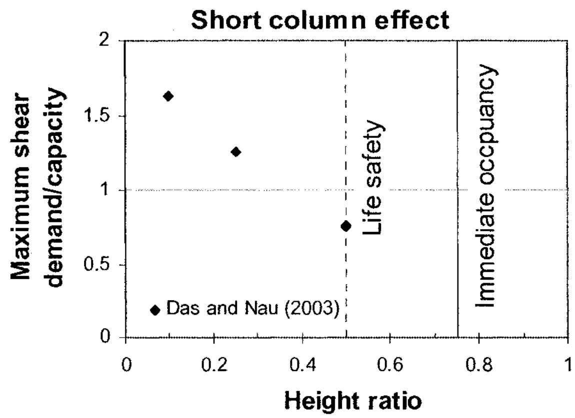
FEMA 310 specifies that "there shall be no columns at a level with height/depth ratios less than 50% of the nominal height/depth ratio of the typical columns at that level for life safety and 75% for immediate occupancy". Das and Nau (2003) have performed parametric analysis on the impact of short column effect on the shear demand/capacity ratio, and the results are illustrated in Figure 5.8c. The Das and Nau (2003) results corroborate the FEMA 310 Life safety and Immediate Occupancy performance limits. The nominal height/depth ratio is used in the fuzzification of *short column effect* (Figure 5.5c).





a) Shear failure at short column[§]

b) Schematic illustration of short column effect



c) Parametric study on the impact of short column effect

([§]Photo credit: Godden Collection, Earthquake Engineering Library, University of California, Berkeley)

Figure 5.8: Quantification of short column effect

Relative strength at joints "weak column-strong beam" (RSJ)

When buildings are designed, the loads are transferred from the diaphragm/roofs to the vertical structural component, such as columns. Prudent structural design dictates the use of strong column-weak beam design in order to initiate the yielding in the beam section. With yielding the beam/girder section deforms from elastic to plastic range, and a plastic hinge is formed. This limits the demand, and minimizes the load transferred to the columns. However, if the yielding initiates in the columns, the lateral deformation of the columns is enhanced and this may introduce P-Δ effect. Consequently, the overall structural integrity of the building is undermined with consequent significant damage (Figure 5.9). Hence, as reasoned throughout the thesis, the columns are of paramount importance in safeguarding the structural integrity of the building and saving lives. The desired values of relative beam and column strength at joints can be obtained from (Mitchell *et al.* 1995):

$$\sum M_{rc} \geq 1.1 \sum M_{nb} \tag{5.4}$$

where M_{rc} is the factored flexural resistance of the column and M_{nb} is the nominal flexural resistance of the beam.

(§Photo credit: Godden Collection, Earthquake Engineering Library, University of California, Berkeley)

Figure 5.9: Damage due to strong beam and weak column.

The ratio of $\sum M_{rc} / \sum M_{nb}$ can be used in the fuzzification of relative strengths at joints. Alternatively, from field visual observation, the impact of relative strength at joints can be linguistically assessed as *negligible*, *low*, *moderate*, *high*, and *significantly high*, the corresponding fuzzification is shown in Table 5.11.

Table 5.11: Fuzzification of relative strength at joints

relative strength at joints	L	M	H
Negligible	0.9	0.1	0
Low	0.7	0.3	0
Moderate	0	0.5	0.5
High	0	0.3	0.7
Significantly high	0	0.1	0.9

Assignment of these linguistic quantifiers necessitates prudence on the part of decision maker. The overall column design and column dimensions relative to those of the beam can be used to provide guidance in this assessment.

Redundancy

Structural redundancy of a building can be defined as the number plastic hinges in the structural system that must be formed or fail to produce collapse (Bertero and Bertero 1999). Bertero and Bertero have highlighted that there are various definition of *structural redundancy*, each of which is context dependent. There are various reported redundancy measure of buildings (e.g., Liao *et al.* 2007; Husain and Tsopelas 2004; Özcebe *et al.* 2003; Wen and Song 2003).

Structural systems that have many lateral load-resisting elements are observed to perform well during earthquakes. Redundancy in the structure ensures redistribution of the earthquake-induced lateral forces within the structural system. Redundancy of a structure can be defined as the indication of the degree of continuity of multiple frame lines to distribute lateral forces throughout the structural system. Özcebe *et al.* (2003) have quantified measure of redundancy through a normalized redundancy ratio (nrr):

$$nrr = \frac{A_{tr}(f_x - 1)(f_y - 1)}{A_{gf}} \tag{5.5}$$

where f_x, f_y = number of continuous frame lines in the critical story in x and y directions, respectively; A_{tr} = the tributary area for a typical column; A_{tr} is taken as 25 m^2 if f_x and f_y

are both greater than or equal to 3. In all other cases, A_{tr} is taken as 12.5 m^2. The reason for this additional penalty on such buildings is that buildings having significant irregularities in plan and/or buildings having just one frame in either direction are considered more vulnerable than the others. A_{gf} = the area of the critical story (usually the ground story).

The *nrr* values computed from the Düzce Database[7] and are plotted in Figure 5.10a. Figure 5.10a show that with increase in observed damage, the *nrr* values decrease. Özcebe *et al.* (2003) have further provided the transformation of *nrr* into a crisp normalized redundancy score value. However, in this thesis, the *nrr* value will be directly used for the fuzzification of Redundancy (Figure 5.10b).

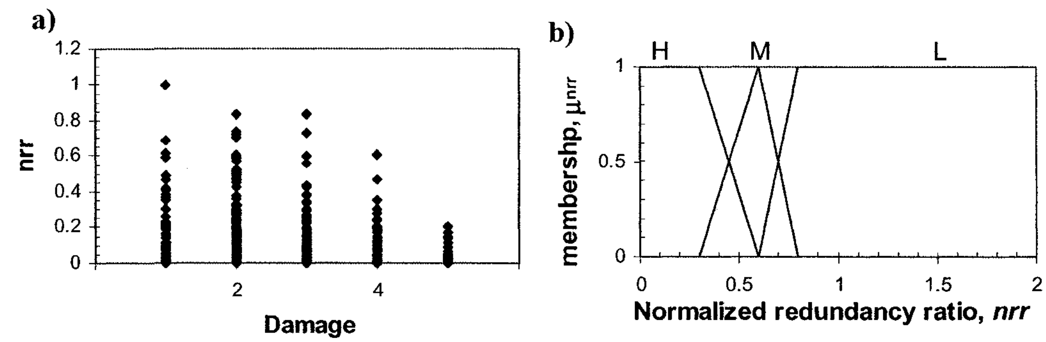


Figure 5.10: Quantification of redundancy a) Normalized redundancy ratio, b) Granulation of Normalized redundancy ratio.

5.1.2 Plan irregularity (PI)

The plan irregularity is used to determine building vulnerability to torsion and potential area of high stress concentration. Forces due to torsion are accentuated more during dynamic response, leading to the overloading of some structural elements. A good design practice is to have a symmetrical plan layout. In this chapter, three basic risk items are considered to quantify PI: *torsional irregularity*, *diaphragm continuity* and *plan building shape*.

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[7] SERU, Middle East Technical University, Ankara, Turkey; Archival Material from Düzce Database located at website http://www.seru.metu.edu.tr.

Torsional irregularity (TI)

Torsional irregularity is introduced due to the lack of symmetry in plan, e.g., variation in perimeter strength-stiffness, false symmetry and mass eccentricities (Figure 5.12), all of which will induce torsion induced forces. The torsional forces are accentuated during dynamic response, leading to the overloading of some structural elements which eventually may cause local damage of structural component or collapse of the overall building. NEHRP 2003 indicates that torsional irregularity and extreme torsional irregularity exist when the computed maximum story drift (including accidental torsion) at one end of the structure is more than 1.2 times and 1.4 times, respectively, of the average of the story drifts at the two edges. These threshold values are used in the fuzzification of torsional irregularity (Figure 5.11a).

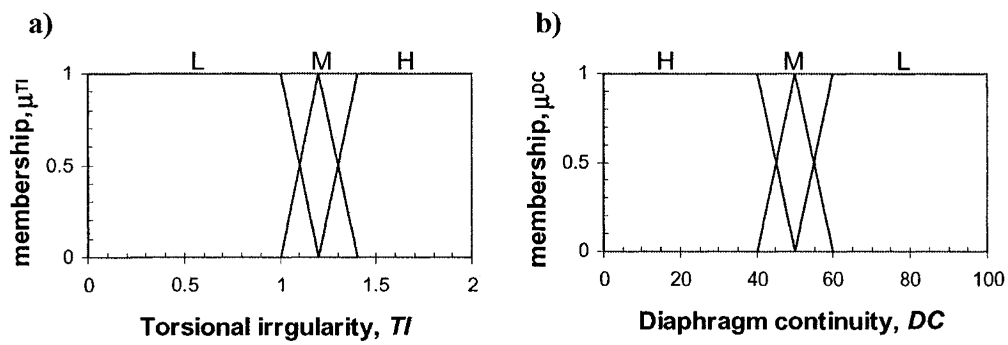


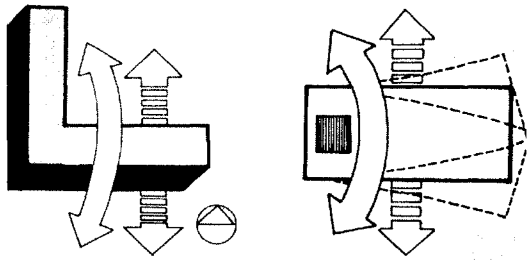
Figure 5.11: Granulation of the basic risk items used in a plan irregularity, a) torsional irregularity and b) diaphragm continuity.

Diaphragm discontinuity (DC)

During earthquake induced lateral loads, the integrity and continuity of lateral load resisting elements to act in unison is ensured through what is called diaphragm actions. Any discontinuity in the diaphragm can compromise the lateral load resisting capacity. According to NEHRP 2003 indicated that diaphragm discontinuity is said to exist when diaphragms have abrupt discontinuities or variations in stiffness. Abrupt discontinuity is said to exist with the presence of a cut-out or open areas that is > 50% of the gross enclosed diaphragm area and variation in stiffness is said to exist when there is a changes in effective diaphragm stiffness of more than 50% from one story to the next. This problem is more severe with flexible diaphragms (Masi *et al.* 1997). Thus, in this thesis, if the diaphragms are rigid, impact of diaphragm discontinuity is neglected. The cut-out or open areas are used in the fuzzification of diaphragm discontinuity (Figure 5.11b).

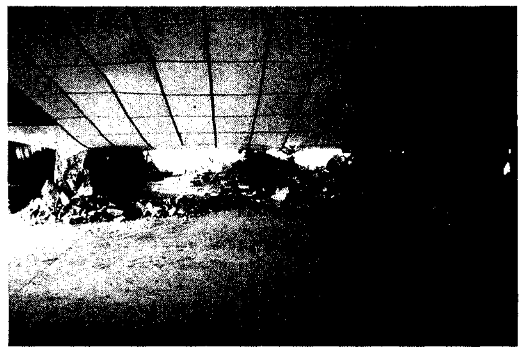
Re-entrant corners (REC)

The shape of the building has an effect on its vulnerability to seismic loads. Different symmetrical shapes (both in x and y direction) of buildings can be used (○, □). However, re-entrant corner buildings having irregular shapes (L, T, U, H, +), may be prone to torsion and stress concentration. NEHRP 2003 indicates that irregularity due to re-entrant corners is said to exist when projection of the structure beyond a re-entrant corner is greater than 15 % of the plan dimension of the structure in the given direction. A basic guideline for quantifying re-entrant corners is illustrated in Figure 5.13, where re-entrant corners is said to exist when $a_x > 0.15L_x$.



a) Torsional effects created by irregular shape of building plan (L configuration) and by a very stiff off-center core area in a rectangular (regular) plan building.

b) Hotel Terminal, Guatemala City. Overall view of 6-story hotel, illustrating the torsional failure of the second story during the 1976 Guatemala Earthquake.



c) Hotel view inside the building showing the collapse of the second story due to shear failure of the second-floor columns.

(Photo credit: Godden Collection, Earthquake Engineering Library, University of California, Berkeley)

Figure 5.12: Plan irregularity and torsion damage.

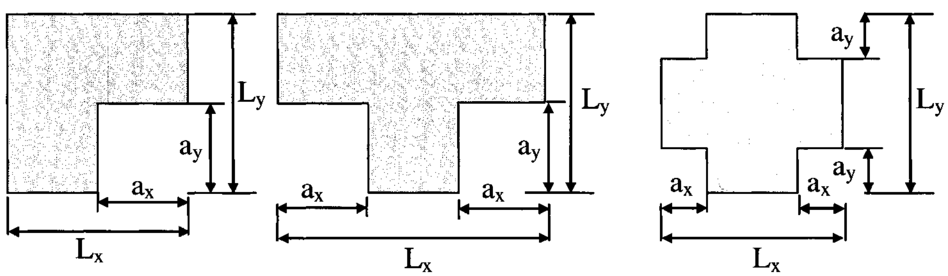


Figure 5.13: Re-entrant corners irregularity.

Through a *walk down* visual observation survey or aerial photograph, the severity of re-entrant corners is assessed linguistically as *negligible*, *low*, *moderate*, *high* and *significantly high*, and corresponding fuzzification is summarized in Table 5.12. The subjective assessment needs prudence to quantify the overall impact of re-entrant corners on critical lateral load resisting elements.

Table 5.12: Fuzzification of Re-entrant corners

Re-entrant corners	L	M	H
Negligible	0.9	0.1	0
Low	0.7	0.3	0
Moderate	0	0.5	0.5
High	0	0.3	0.7
Significantly high	0	0.1	0.9

Walls

The moment-resisting frames resist lateral forces by bending and shearing of columns and beams, which are connected by moment connections. The columns are responsible for overall strength and stability and hence are critical elements. Their strength relative to the adjoining beams plays an important role in seismic resistance in terms of dictating the sequence of plastification among the framing elements. Their inelastic deformability, as governed by concrete confinement and shear capacity are critical. The detailing of beam-column connections is also important for seismic performance. Frames are susceptible to excessive lateral drifts and associated secondary (P-Δ) moments. Performance of bare frame RC buildings is often modified through the use of structural walls. These structural walls can be categorized under structural (or shear) walls and masonry walls.

Shear walls resist almost all the seismic forces when used in buildings. Although the term "shear wall" is well accepted and used within the engineering community, their predominant mode of behaviour is usually flexural when used in medium to high-rise construction. They typically act as vertical cantilevers and provide lateral bracing to the system, while receiving lateral forces from diaphragms and transmitting them to the foundation. The size and location are critical for seismic resistance. Shear walls in buildings constructed prior to the enactment of modern seismic design codes are lightly reinforced flexible elements and tend to extend throughout the building height. In recent years, shear walls occur in isolated locations and are more heavily reinforced. Shear wall structures have been reported to behave well under moderate to strong earthquake excitations (Saatcioglu *et al.* 2001, Mitchell *et al.* 1995).

Many older frame buildings include masonry infill panels. Although unreinforced masonry behaves in a brittle manner and is regarded as undesirable construction material for seismically active regions, walls of unreinforced masonry may act as shear walls in controlling deformations, and may save non-ductile concrete frames until the elastic limit of the former is exceeded. There have been many cases of non-ductile frames that have survived strong earthquakes due to the participation of masonry infill walls, especially when the wall-to-floor area ratio is high.

Penelis and Kappos (1997) have performed analytical work to identify the structural resistance of RC buildings and corresponding contributors (Figure 5.14). Figure 5.14 shows the results for: BF = bare frame; IF1 = fully infilled frame, low strength masonry (τ_u = 0.27 MPa); IF2 = fully infilled frame, high strength masonry (τ_u = 0.38 MPa); IF1P = infilled frame with open ground storey, low-strength masonry; IF2P = infilled frame with open ground storey, high-strength masonry.

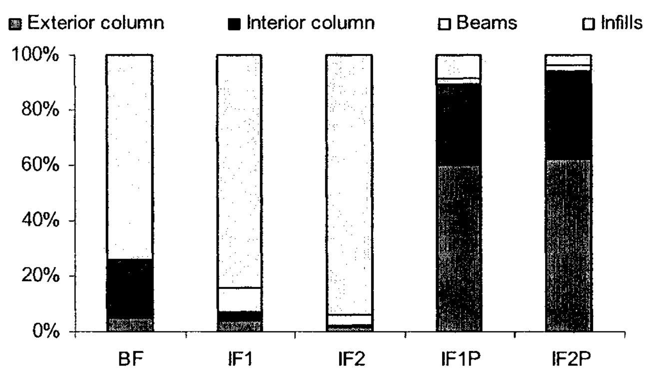


Figure 5.14: Structural wall resistance (after Penelis and Kappos 1997).

Figure 5.14 shows that, for the BF building, the beams and interior columns contribute 90% of the lateral load resistance. However, with the incorporation of infill walls, the lateral load is resisted more by the infills. Comparison of IF1 and IF2 show only a small impact of the strength of the infill material. Further, when the infill material is used along with open ground floors, IF1P and IF2P, there is higher contribution of the exterior and interior columns in resisting the lateral force.

The walk down structural wall identification survey is linguistically evaluated as b*are frame, lightly reinforced masonry walls, heavily reinforced masonry walls, lightly reinforced shear walls* and *heavily reinforced shear walls*. The corresponding fuzzification of structural walls is shown in Table 5.13.

Table 5.13: Fuzzification of structural walls

Structural walls	L	M	H
Bare frame	0	0.1	0.9
Lightly reinforced masonry walls	0	0.3	0.7
Heavily reinforced masonry walls	0.7	0.3	0
Lightly reinforced shear walls	0.5	0.5	0
Heavily reinforced shear walls	0.9	0.1	0

5.1.3 Decrease in resistance (DR)

The fuzzy hierarchical rule base generated for computing decrease in resistance is shown in Figure 5.15. The rule hierarchies shown in Figure 5.15 are developed by considering the functional relation of each performance modifier. The *year of construction* is aggregated through $\mathbf{R}_{5(2)}$ (Table 5.14).by considering the year the building is constructed and prevalent *code enforcement*. The year of construction is aggregated with *design quality* (columns, diaphragms, and joints) through $\mathbf{TR}_{6(2)}$ (Table 5.15). *Structural degradation/ weakening* is computing by considering *damaged from previous earthquakes* and deterioration due to *corrosion* through $\mathbf{R}_{6(2)}$ (Table 5.16). Structural degradation/ weakening are aggregated with prevalent *construction quality* through $\mathrm{TR}_{7(2)}$ (Table 5.17). Finally, the *decrease in resistance* is aggregated by aggregating output of $\mathrm{TR}_{6(2)}$ and $\mathrm{TR}_{7(2)}$ through $\mathbf{R}_{7(2)}$ (Table 5.18).

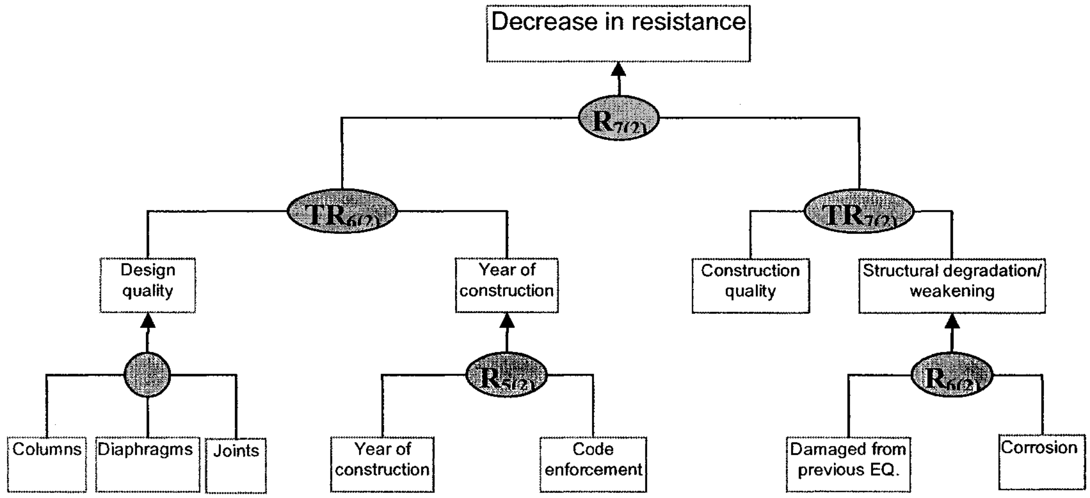


Figure 5.15: Hierarchical rule base for decrease in resistance.

Table 5.14: Fuzzy rule base for year of construction

$R_{5(2)}$	Rule *i*	YC	CE	**Year of construction**
	1	L	L	L
	2	L	M	M
	3	L	H	M
	4	M	L	L
	5	M	M	M
	6	M	H	H
	7	H	L	M
	8	H	M	H
	9	H	H	H

YC = Year of construction; CE = code enforcement

Table 5.15: Fuzzy rule base for temporary node 6

$TR_{6(2)}$	Rule *i*	DQ	YC	**Temp node 6 *f*(DQ, YC)**
	1	L	L	L
	2	L	M	L
	3	L	H	M
	4	M	L	M
	5	M	M	M
	6	M	H	H
	7	H	L	M
	8	H	M	H
	9	H	H	H

DQ = Design quality

Table 5.16: Fuzzy rule base for structural degradation/weakening

$R_{6(2)}$	Rule *i*	DEQ	Corrosion	**SDW**
	1	L	L	L
	2	L	M	M
	3	L	H	M
	4	M	L	L
	5	M	M	M
	6	M	H	H
	7	H	L	M
	8	H	M	H
	9	H	H	H

DEQ = Damage from previous earthquake; SDW = Structural degradation/weakening

Table 5.17: Fuzzy rule base for temporary node 7

$TR_{7(2)}$	Rule *i*	CQ	SDW	**Temp node 7 *f*(CQ, SDW)**
	1	L	L	L
	2	L	M	L
	3	L	H	M
	4	M	L	L
	5	M	M	M
	6	M	H	H
	7	H	L	M
	8	H	M	H
	9	H	H	H

CQ = Construction quality; SDW = Structural degradation/weakening

Table 5.18: Fuzzy rule base for decrease in resistance

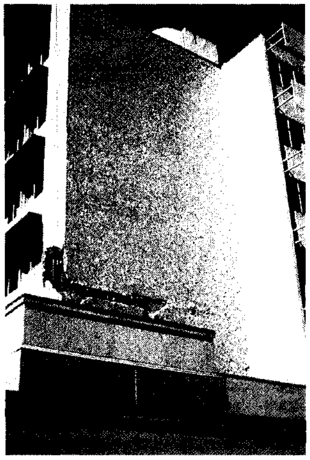
$R_{7(2)}$	Rule *i*	$TR_{6(2)}$	$TR_{7(2)}$	**Decrease in resistance**
	1	L	L	L
	2	L	M	L
	3	L	H	M
	4	M	L	M
	5	M	M	M
	6	M	H	H
	7	H	L	M
	8	H	M	H
	9	H	H	H

Construction quality (CQ)

Resilience of buildings to seismic loading is determined by its seismic design detailing and quality of material and construction. Poor quality of construction and material used are detrimental to ensuring that the intended design protection is in place (Figure 5.16). Causes of poor construction quality are:

- construction error (erroneous placement of reinforcement bars; non uniform spacing of ties, etc.);
- improper construction procedures (creating unwanted construction joint and weak zone; honey combing; poor concrete placement, etc.);
- lack of anchorage of beam and column reinforcement; use of plain bars for reinforcement, without proper hooks; insufficient splice length; use of non-seismic hook (90^{o} instead of 135^{o});
- poor concrete quality (lacking of adequate cement; poor gradation of aggregate; lack of fine aggregate; too coarse aggregate; impurities in concrete mix, etc.).

Dimova and Negro (2005) have performed experimental work to quantify the influence of construction deficiencies on the seismic response of structures. Dimova and Negro constructed two identical buildings where one was built with proper construction design and detailing, and other building with construction deficiencies. The deficiencies were inaccurate spacing of the stirrups, wrong anchoring of longitudinal reinforcement of the columns into the beams, and inaccurate placement or lack of stirrups in the beam-column joints. Dimova and Negro have reported that as a result of these deficiencies, the yield displacement and corresponding base shear force decreased by 27% and 12%, respectively; and the ultimate story displacement, and the maximum base shear force reduced by 34% and 22%, respectively.



a) Holy Cross Building, Los Angeles, California, 1971 San Fernando Earthquake. Failure of this wall occurred at the fourth floor level during the earthquake due to poor workmanship.

b) Illustrating clearly how the concrete in this critical region has been disrupted (broken off) as a consequence of the premature (early) ending of the spiral reinforcement.

c) Honeycombing with complete lack of mortar in the column.

d) Unanchored longitudinal reinforcing bars on the roof level.

(Photo credit: Godden Collection, Earthquake Engineering Library, University of California, Berkeley)

Figure 5.16: Different construction quality defects.

The construction quality is qualitatively determined linguistically as: *extremely poor*, *poor*, *moderate*, *good* and *extremely good.* A number of different construction related errors may be identified, and they may have compounding effect so that construction should be assigned values in the lower end of the scale. Intermediate scales are selected based on the judgment of the field evaluator. The corresponding fuzzification is shown in Table 5.19.

Table 5.19: Fuzzification of Construction quality

Construction quality	L	M	H
Extremely poor	0	0.1	0.9
Poor	0	0.3	0.7
Moderate	0.5	0.5	0
Good	0.7	0.3	0
Extremely good	0.9	0.1	0

Design quality (DQ)

Design quality is introduced to encapsulate the level of detailing considered in seismic design. Irrespective of the year of construction and *in situ* construction practice, the level of design quality may be detrimental to the building survivability under earthquake. As alluded throughout the thesis, the building resistance to lateral loading is ensured through the effective interaction of columns and/or shear walls, joints, and diaphragms. Proper design quality of the three elements ensures better resistance to seismic loads. Poor design and detailing of any of the three elements would jeopardize the lateral load resistance of buildings.

In the building evaluation, the design quality is qualitatively quantified through five linguistic descriptors (Table 5.20): *extremely poor*, *poor*, *moderate*, *good* and *extremely good*. Specification of design quality through linguistic descriptors requires prudence in the evaluation process. When two or more elements are deficient the design quality can be evaluated as extremely poor, for example. The column design quality is good and the poor quality is only associated with the diaphragm, moderate building quality can be selected.

Table 5.20: Fuzzification of design quality

Design quality	L	M	H
Extremely poor	0	0.1	0.9
Poor	0	0.3	0.7
Moderate	0.5	0.5	0
Good	0.7	0.3	0
Extremely good	0.9	0.1	0

Year of construction (YC) and code enforcement (CE)

During the initial site investigation, the original design drawings may not be readily available. The YC can be used to infer important information about the seismic design code provision and consequently about the ductility, strength and detailing. Reported causes of failure that can be related to the year of construction are lack of column confinement and poor detailing; and the use of non ductile material. The transformation and fuzzification of YC is discussed in Chapter 3.

The YC discussed in Chapter 3 is based on the presumption that stringent code enforcement is ensured. However, as observed during current reconnaissance reports, this is not the case and hence consideration of code enforcement is paramount. The resilience of a building is compromised even if up-to-date code is used but code enforcement is not ensured. Therefore, code enforcement is incorporated as part of basic risk items. The code enforcement is linguistically evaluated as lenient, moderate, and stringent. The corresponding fuzzification is shown in Table 5.21.

Table 5.21: Fuzzification of Code enforcement

Code enforcement	L	M	H
Lenient	0	0.1	0.9
Moderate	0.3	0.5	0.2
Stringent	0.9	0.1	0

Structural degradation and weakening

<u>Damage from previous earthquake (DPE)</u>

Buildings damaged during previous earthquakes and subsequently repaired are prone to the same type of damage as other buildings (Penelis and Kappos 1997). The problem commonly encountered in such repaired structures is that the repaired work may be substandard. This was observed during the 1985 Mexico earthquake, where substandard repair or un-repaired structures failed catastrophically (Popov 1987). Thus, in the *walk down* survey and previously logged information, one of the information collected is the level of damage incurred during previous earthquakes and the corresponding repair and upgrade performed. The linguistic parameters used to quantify the severity of damage are *extremely severe*, *severe*, *moderate* and *negligible.* The corresponding description and fuzzification are summarized in Table 5.22.

Table 5.22: Damage during previous earthquake

Damage from previous earthquake	Description	L	M	H
Extremely sever	Severely damaged and sub standard repair	0	0.1	0.9
Sever	Severely damaged and standard repair	0	0.3	0.7
Moderate	Moderate damage and standard repair	0.3	0.5	0.2
Negligible	Any type damage and high standard repair, or no damage	0.9	0.1	0

Deterioration (corrosion)

Seismic evaluation is carried out under the assumption that the structural components are sound and that attention has been given to detailing and other structural parameters. Nevertheless, structures exposed to deleterious environmental conditions, e.g. parking garages, where de-icing salt is used, will be subject to deterioration with time (e.g. Bertero and Shah 1983). With increasing deterioration, the structural capacity of different structural elements, such as beams, columns and beam-column joints may be compromised and the seismic resilience is reduced. Consequently, under any seismic loading, the deteriorated location may fail prematurely with catastrophic consequences. For seismic resistance, the criticality of each element may be ranked as beams/girders, columns and joints, in an increasing order. The linguistic parameters used to quantify the severity of damage are *extremely severe*, *severe*, *moderate* and *good,* and *extremely good.* The corresponding fuzzification is summarized in Table 5.23.

Table 5.23: Damage due to deterioration

Deterioration (corrosion)	L	M	H
Extremely sever	0	0.1	0.9
Sever	0	0.3	0.7
Moderate	0.3	0.5	0.2
Good	0.7	0.3	0
Extremely good	0.9	0.1	0

5.1.4 Problem of adjacency

Adjacency building problems is observed when two buildings with different floor levels and close separation distances are subjected to seismic induced inertia loads. If the separation distance is close, there is a potential for pounding. Furthermore, the pounding may induce additional inertia loads and damage the adjacent building (e.g., Figure 5.17). Buildings that are the same height and matching floors exhibit similar dynamic behaviour. As a result, pounding is between the corresponding floor levels, and the damage will be limited to non-structural components. However, when the floor heights do not coincide, the difference in dynamic response coupled with the level of slab differences induce impact loads on the adjacent building columns. Consequently, discontinuities of the lateral force resisting element are introduced, and both buildings may be severely damaged, with potentials for collapse.

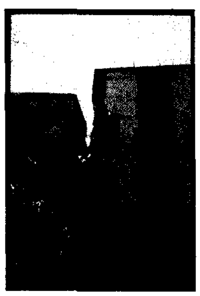


Figure 5.17: Pounding of adjacent building and the corresponding damage.

FEMA 310 specifies that buildings of same height, with matching floor levels are exempt from this analysis. A general 4% of the building height threshold value is specified to limit the spacing between two adjacent buildings for Life Safety and Immediate Occupancy. Thus, the potential for pounding two adjacent building is determined by considering floor elevation level and spacing between adjacent buildings, and the fuzzy hierarchal rule base is shown in Figure 5.18. The FRB for problem adjacency, $\mathbf{R}_{4(2)}$, is provided in Table 5.24.

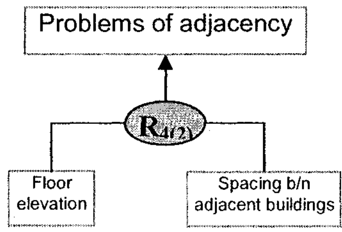


Figure 5.18: Hierarchical rule base for problem of adjacency

Table 5.24: Fuzzy rule base for problem of adjacency

$\mathbf{R}_{4(2)}$	**Rule *i***	**Floor elevation**	**SB**	**Problem of Adjacency**
	1	L	L	L
	2	L	M	L
	3	L	H	M
	4	M	L	L
	5	M	M	M
	6	M	H	H
	7	H	L	M
	8	H	M	H
	9	H	H	H

SB = Spacing between adjacent buildings

The floor level between two adjacent buildings is evaluated linguistically as (Table 5.25): *same level*, *slightly different*, and *mid height*. The spacing between two adjacent buildings is used in the fuzzification of spacing between two adjacent buildings (Figure 5.19).

Table 5.25: Fuzzification of relative height of slabs

Spacing between two corresponding slabs	Description	L	M	H
Same level	Slabs of two adjacent buildings are at the same level	0.9	0.1	0
Slightly different	Slabs are slightly off the same level, and not at mid height	0.2	0.3	0.5
Mid height	Slab of one building is at mid height of the adjacent building	0	0.1	0.9

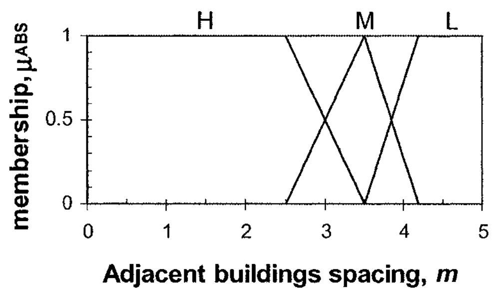


Figure 5.19: Granulation for spacing between two adjacent buildings.

5.2 Case study

On May 1, 2003, the city of Bingöl, Turkey, was struck with an earthquake moment of magnitude Mw = 6.4, which caused 168 casualties, 520 injuries and damage to several buildings. The total economic loss to the Turkish national economy was estimated to be over 400 million US dollars (Doğangün 2004). The damage is classified into five discrete stages: none (ND), light (LD), moderate (MD), severe (SD) and collapse (CD). The damage state definitions employed are summarized in Table 5.26. Summary of the Bingöl Database[8] is shown in Table 5.27.

The performance indicators shown in Figure 5.2, are collated from the Bingöl Database and other reported studies (Hakut 2004). The performance indicators are synthesized and the summaries are plotted in Figure 5.20 and Figure 5.21. The field reconnaissance reports for the

[8] SERU, Middle East Technical University, Ankara, Turkey; Archival Material from Bingöl Database located at website http://www.seru.metu.edu.tr.

performance indicators shown in Figure 5.20a to d are provided at five discrete scales, {1, 2, 3, 4, 5}, where 1 and 5 correspond to N_D and C_D, respectively. These values are associated with the linguistic performance indicators provided earlier.

Table 5.26: Damage state definitions employed

Damage state	column	Beam	Shear wall	Infill wall
Light/none	Visible flexural and inclined hairline cracks	Visible flexural and inclined hairline cracks	Visible flexural hairline cracks	Surface crack along the boundaries
Moderate	Clear flexural and shear cracks	Wide flexural and inclined cracks, spalling of concrete	Visible inclined hairline cracks and clear flexural cracks	Diagonal cross cracks, separation from the frame
Heavy/collapse	Slab of one building is at mid height of the adjacent building	Large cracks, plastic hinge formation, crushing of concrete	Complete diagonal cracks, spalling of concrete, exposure of reinforcement	Through cross cracks, rupture of bricks

Table 5.27: Summary of Bingöl database for Tier 2 evaluation

Building ID	Walls	BC-joint	Redundancy	SC	WS	SS	TI	DC	REC	CQ	YC	Corrosion	Pounding potential	R/C
BNG-10-3-10	RCF	2	0.536	No	Yes	Yes	Yes	No	Yes	Poor		0	5	M
BNG-10-3-3	RCF	5	0.408	No	No	No	No	No	No	Poor	1975	4	5	M
BNG-10-4-4	RCF	2	0.167	No	Yes	Yes	Yes	No	No	Average	1998	5	0	M
BNG-10-4-6	RCF	2	0.291	Yes	No	No	Yes	No	No	Average	1976	5	0	L
BNG-10-4-7	RCF	2	0.136	No	Yes	Yes	Yes	No	No	Average	1988	5	0	L
BNG-10-4-9	RCSW	5	0.227	No	Yes	Yes	Yes	Yes	No	Good	2002	3	0	N
BNG-10-5-1	RCSW	5	0.171	No	No	Yes	Yes	No	No	Average	1990	0	5	M
BNG-10-5-11	RCF	2	0.326	No	No	Yes	No	No	No	Average	1988	5	3	L
BNG-10-5-2	RCSW	5	0.134	No	No	No	Yes	No	Yes	Good	1990	5	5	L
BNG-11-2-3	RCF	2	0.770	No	Yes	Yes	No	No	Yes	Poor		5	5	M
BNG-11-4-1	RCSW	2	0.024	No	No	Yes	Yes	No	Yes	Poor	1999	5	3	S
BNG-11-4-2	RCF	0	0.172	No	Yes	Yes	Yes	No	Yes	Poor	1989	5	5	S
BNG-11-4-4	RCF	2	0.026	No	Yes	Yes	Yes	No	Yes	Poor	2000	5	5	M
BNG-11-4-5	RCF	5	0.343	No	Yes	No	Yes	No	Yes	Average	1997	5	5	L
BNG-3-4-1	RCF	2	0.154	No	Yes	No	No	No	No	Poor	1998	5	5	L
BNG-3-4-2	RCF	5	0.161	No	No	No	No	No	No	Average	1996	5	5	N
BNG-3-4-4	RCF	2	0.196	No	Yes	No	No	No	No	Average		5	5	N
BNG-5-5-1	RCF	2	0.101	No	Yes	Yes	Yes	No	Yes	Average	1990	5	3	L
BNG-6-2-8	RCF	5	0.214	No	No	No	Yes	No	Yes	Poor	1992	5	5	S
BNG-6-3-1	RCF	5	0.183	Yes	No	Yes	No	No	Yes	Average	1991	5	0	M
BNG-6-3-10	RCF	2	0.615	No	Yes	Yes	Yes	No	Yes	Good	1995	5	5	N
BNG-6-3-11	RCF	2	0.319	No	Yes	Yes	Yes	No	Yes	Average		5	5	N
BNG-6-3-12	RCF	0	0.225	No	Yes	Yes	Yes	No	Yes	Average		5	0	L
BNG-6-3-4	RCF	5	0.436	No	No	No	No	No	Yes	Average	2003	5	5	L
BNG-6-4-2	RCF	0	0.025	Yes	Yes	Yes	Yes	Yes	Yes	Poor	2001	0	3	S
BNG-6-4-3	RCF	0	0.057	No	No	Yes	Yes	No	Yes	Poor	2003	5	5	C
BNG-6-4-5	RCF	5	0.377	No	No	No	Yes	No	Yes	Good	1996	5	5	N
BNG-6-4-7	RCSW	5	0.026	No	No	No	No	No	Yes	Poor	1996	5	5	S

Figure 5.20a, b, and c show the building performance modifiers for NRR, WCSB and presence of corrosion, respectively. With increasing level of observed damage the three modifiers show a decreasing trend, as expected. Figure 5.20d shows the potential for

pounding. However, the observed damage does not seem to be affected by the potential for pounding.

Figure 5.21a, b show presence of torsional irregularity and short column effect against observed damage states, respectively. Figure 5.21c shows negligible impact of diaphragm discontinuities. Of all the factors, the lack of construction quality as illustrated in Figure 5.21d has the highest impact on the prevalent damage states, as intuitively expected.

Initial screening of the data shows that the year of construction has a counter intuitive result such that the newer structures are showing more damages. One possible explanation is that newer buildings are designed with higher ductility, however, due to poor construction practice and lenient code enforcement, the expected ductility capacity is undermined. Although the older buildings lack ductility, the stronger infill material helps in resisting earthquake induced load and prevents severe damages. Thus, the basic risk item identified under year of construction, *code enforcement*, is specified as "lenient".

Inputs to the performance modifiers obtained from the Bingöl Database are furnished as linguistic and numeric values (Table 5.27). These inputs values are not as defined in the previous section. As a result, initial transformations into commensurable units are made. The WS, SS, SC, TI, REC, and DC are provided linguistically as *yes* for *present* and *no* for *not present*. The transformation values selected are in congruence with the universe of discourse for each basic risk item. Thus, the transformation values used for WS, SS, and SC are: *yes* = 32.5 and *no* = 55; for TI: *yes* = 1.35 and *no* = 1.05; for DC and REC: *yes* = "Significantly high" and *no* = "Negligible". The transformation values for WCSB and corrosion are shown in Figure 5.22a and Figure 5.22b, respectively. In Figure 5.23 the transformation for potential for pounding is provided. The ensuing FRB modeling is limited to quantifying building damageability index.

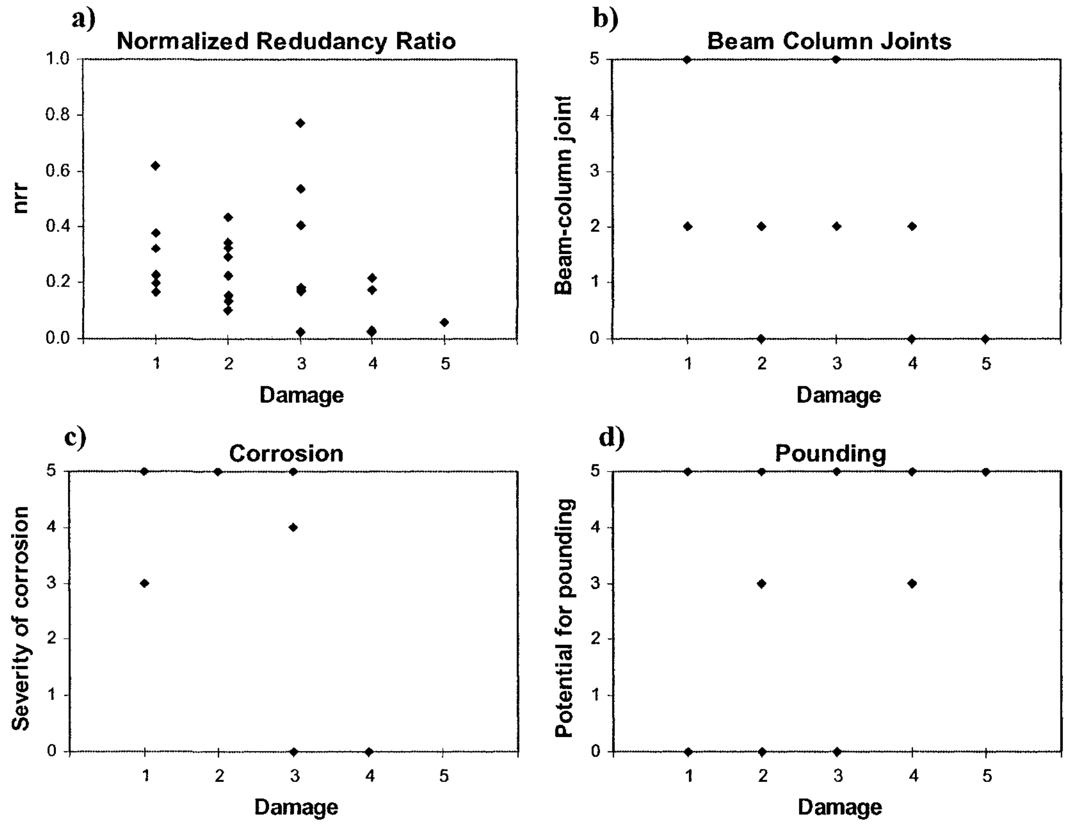


Figure 5.20: Building performance modifiers a) Normalized redundancy ratio, b) Beam column joint quality, c) Prevalent Corrosion and d) Potential for pounding.

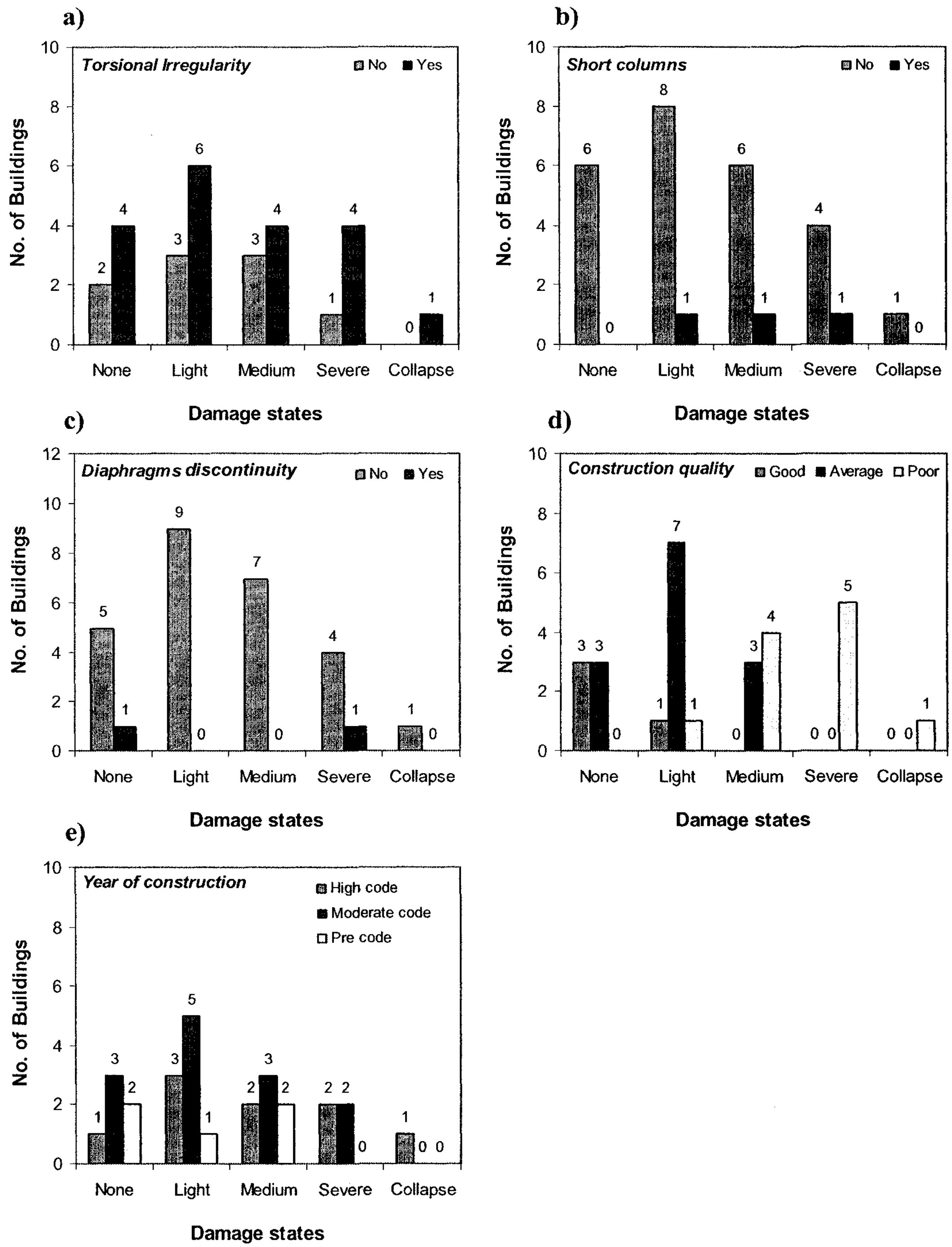


Figure 5.21: Building performance modifiers a) Torsional irregularity, b) Short columns effects, c) Diaphragm continuity, d) Construction quality and e) Year of construction.

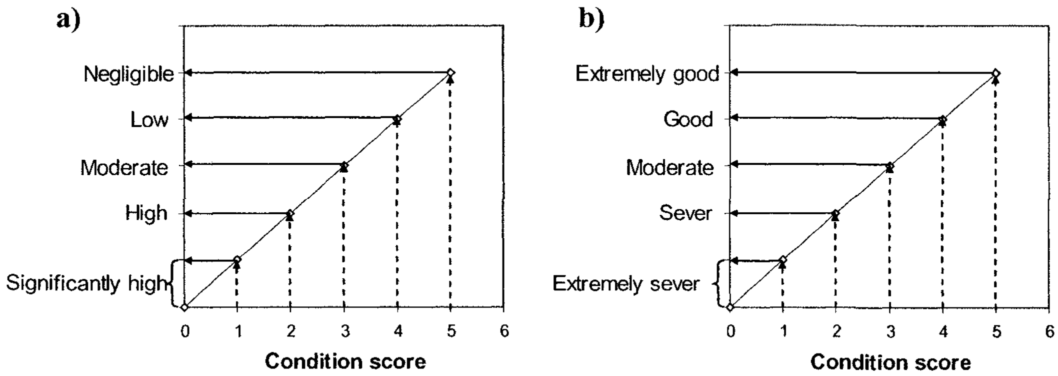


Figure 5.22: Transformation values a) relative strength at the joint b) corrosion

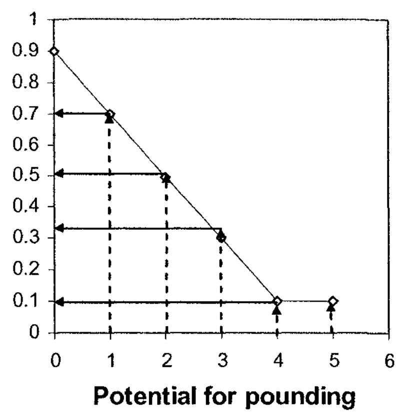


Figure 5.23: Transformation values for potential for pounding

Following the previously outlined procedure, for the Bingöl database (Table 5.27), the FRB is computed and the results of the I^{BV} are plotted in Figure 5.24 and Figure 5.25, without and with the problem of adjacency, respectively. Both figures show that with increasing observed damage states, the I^{BV} value increases.

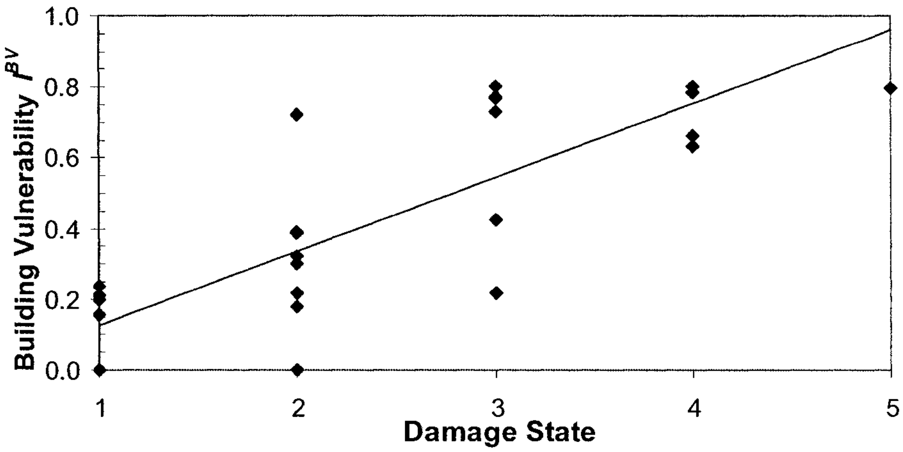


Figure 5.24: Building vulnerability for May 1, 2003 Bingöl Earthquake (without problem of adjacency).

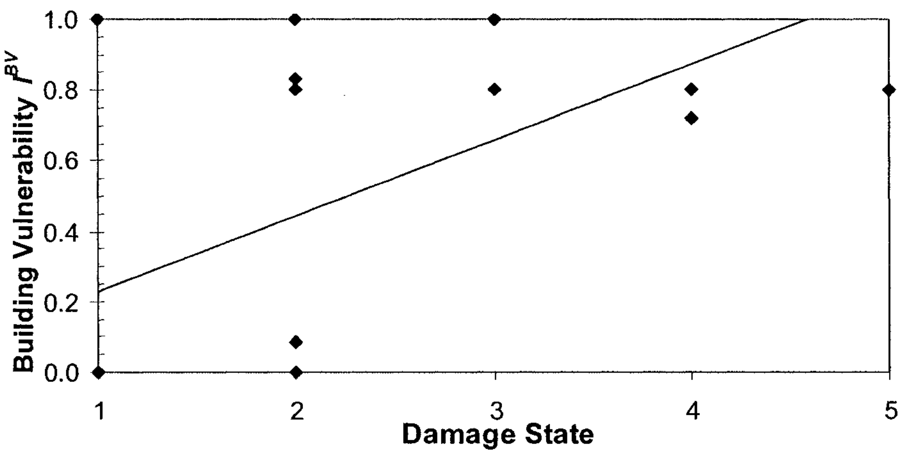


Figure 5.25: Building vulnerability for May 1, 2003 Bingöl Earthquake (with problem of adjacency).

5.3 Conclusions

Risk-based seismic analysis approach is proposed for prioritizing buildings for retrofit and repair. Risk-base prioritization incorporates engineering decision making aspects, such as damage estimation, and societal value, tolerance to the consequence of failure. The risk-based prioritization is undertaken by integrating site seismic hazard (SSH), building vulnerability, and importance/exposure factors. The complexity of building vulnerability assessment is handled through a systems theory, where the complex problem is managed by a simple hierarchical structure. The vagueness uncertainty encountered as a result of subjective walk down survey are handled through a fuzzy set theory, and fuzzy rule based modelling is used to incorporate decision maker's attitude and intuitive knowledge in the aggregation process using FRB modelling.

The proposed method is validated through the use of May 1, 2003 Bingöl Earthquake damage observations. Results of the proposed risk-base prioritization method show correlation with observed damage, albeit extracted from limited data sets.

Chapter 6
Summary, Conclusion and Future Recommendations

6.1 Summary

The vulnerability of existing RC buildings is apparent from past earthquake damages. The vulnerabilities stem from many factors, including older design codes and/or poor practices at the time of design and construction. Most of these older buildings are currently operational and are required to be further assessed and upgraded to minimize seismic damage and improve life safety. In this thesis, the potential for damage coupled with the consequence of failure are integrated into a rational risk-based prioritization. The risk-based prioritization entails consideration of site seismic hazard, building vulnerability and importance, and exposure of the occupants to the hazard.

The proposed techniques incorporate heuristics based hierarchal structures and incorporate wisdoms and intuitive knowledge obtained from practitioners. The building vulnerability assessment is modeled through two-tier hierarchical structures. In Tier 1 evaluation the parameters considered are in congruence with FEMA 154: i) building type, ii) vertical irregularity, iii) plan irregularity, iv) year of construction, and v) construction quality. These parameters can easily be obtained from a walk down survey and engineering drawings. If results of the Tier 1 evaluation show high potential for damage or risk, Tier 2 evaluation can be performed. Tier 2 evaluations are developed by considering detailed building vulnerability parameters in congruence with FEMA 310 screening guideline. Once this information is obtained, a seven step aggregation scheme of Tier 1 and 2 evaluations are proposed.

These performance modifiers are can be obtained through a *walk down* survey. However, the walk down survey is subject to vagueness type uncertainty, and it is modeled through fuzzy set theory. As well, the fuzzy modeling approach is utilized to incorporate intuitive engineering expertise in the model. The proposed techniques are illustrated with a case study. The two proposed methods are validated using the 1994 Northridge Earthquake, California and 2003 Bingöl Earthquake, Turkey data. The fuzzy rule based modeling and heuristic building vulnerability modules are implemented in a prototype Excel based program. Also, the Canadian site seismic hazard is incorporated into the program.

6.2 Conclusions

This thesis has highlighted the utility of risk-based prioritization of RC buildings situated in seismically active areas. A two tier heuristic evaluation technique is proposed and a prototype Excel based program is generated. From this study, the following particular points can be concluded:

- Risk analysis of existing structures is of paramount importance in the management and mitigation of risk,
- Risk-based prioritization incorporates the engineering decision making aspects, such as damage estimation, and societal values, such as tolerance for the consequence of failure,
- The two tier hierarchical risk analysis is a simple and intuitive technique for prioritization of RC building,
- Data for Tier 1 evaluation can readily be obtained from a walk down survey ,

- Ambiguity uncertainty involved in the subjective assessment of RC building is captured through the fuzzy set theory, and FRB modeling and FSE are used to aggregate through the hierarchical structure,
- Both aggregation techniques capture observed RC building damageability of the 1994 Northridge earthquake and 2003 Bingöl earthquake, albeit extracted from limited data sets,
- The Tier 2 evaluation is developed by considering detailed building vulnerability parameters as provided in FEMA 310 screening guideline, and validation performed through the 2003 Bingöl earthquake damage database shows good correlation.

6.3 Future recommendations

This dissertation covered broad research topics, such as hazard assessment, building vulnerability assessment and building importance / exposure assessment. Each is encapsulated in a modular fashion and will benefit from further research and refinement.

- The building hazard assessment considered in this thesis considers the response spectrum provided in the National Building Code of Canada. The median values provided with the building code are used. Thus, this thesis did not consider the uncertainty and variability of the hazard assessment. In future work, the stochastic variability should be taken into consideration in risk analysis.
- Ground motion is the only hazard assessment considered in this study. However, in future work, liquefaction and landslide hazard need to be considered.
- The building damage database used in this study lacked higher level damages. Thus, in future work, the calibration of proposed procedure should be done with different earthquake databases coupled with analytical work. As well, for Tier 2

evaluation, more detailed analytical work is required to delineate different performance indicators and to refine the fuzzification.

- The building importance / exposure module needs further refinement.
- Utility of the proposed technique should be measured with different pilot studies and calibration of proposed model should be done with different stakeholders.
- The proposed risk analysis technique can be integrated with a geographical information system (GIS).

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Appendix A

Uncertainty is an unavoidable and inevitable component of any risk analysis. Input parameters used in quantifying risk are subject to data scarcity and variability. As a result, often, risk analysis is performed with stochastic distribution. Typology and definition of uncertainty within engineering community is vast and often conflicting. In risk analysis, for example, the prevalent uncertainty can be categorized into aleatory and epistemic uncertainty. Aleatory (variability) uncertainty is due to the natural heterogeneity or stochasticity of the input parameter, and it cannot be reduced. Epistemic uncertainty is due to partial ignorance or subjectivity (vagueness), and can be reduced with the availability of more information. Klir and Yuan (1995) have broadly categorized uncertainty into vagueness and ambiguity (Figure A.1).

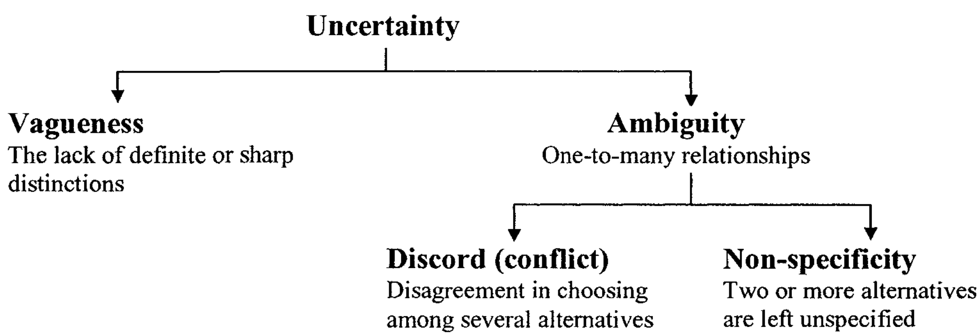

Figure A.1 Typology of uncertainty (modified after Klir and Yuan 1995)

The proposed seismic risk analysis may inherently involve vagueness uncertainty. Vagueness (imprecision) refers to lack of definite or sharp distinction, whereas ambiguity is due to unclear distinction of various alternatives, which is further divided into *discord* (conflict) and *non-specificity*. The taxonomy of uncertainty shown in Figure A.1, albeit to a different degree, is reflected in the seismic risk analysis. Indeed, the ignorance and variability uncertainty, requires different methods of uncertainty propagation (Ferson and Ginzburg 1996). Klir (2004, 1991) has proposed a 'generalized information theory' (GIT) to develop broader uncertainty quantification. The GIT framework for uncertainty theories are provided in Table A.1. The definition and discussion of the GIT framework are presented in Klir (2004). The difference between probability and possibility theories are further expanded in Table A.2.

Table A.1. Framework for uncertainty theories (Klir 2004)

Uncertainties theories			Formalized languages					
			Classical sets	Nonclassical sets				
				Standard fuzzy sets	Interval valued	Type 2	Type 3	...
Monotone measures	Additive	Classical numerical probability	1	8				
		Possibility necessity	2	9				
	Non additive	Sugeno λ-measure	3	10				
		Belief/ Plausibility (Capacity of order ∞)	4	11				
		Capacity of various finite orders	5					
		Interval-values probability distributions	6	12				
		:						
		General lower and upper probabilities	7					

Table A.2. Probability theory versus possibility theory: comparison of mathematical properties for finite sets (modified after Klir and Yuan 1995)

Probability theory	Possibility theory
Based on measures of one type: probability measures (Pro)	Based on measures of two types: possibility measures (Pos) and necessity measure (Nec)
Body of evidence consists of *singletons*	Body of evidence consists of a *family of nested subsets*
Unique representation of Pro by a *probability* distribution function	Unique representation of Pos by a possibility distribution function
p: X→[0,1]; via the formula: Pro(A) = $\sum_{x \in A} p(x)$	r: X→[0,1]; via the formula: Pos(A) = $\max_{x \in A} r(x)$
Normalization: $\sum_{x \in X} p(x) = 1$	Normalization: $\max_{x \in X} r(x) = 1$
Additivity: Pro(A U B) = Pro(A) + Pro(B) – Pro(A ∩ B)	*Max/Min rules*: Pos(A U B) = max[Pos(A) , Pos(B)] Pos(A ∩ B) = min[Nec(A) , Nec(B)]
Total ignorance: P(x) = 1/|X| for all x ∈ X	Total ignorance: r(x) = 1 for all $x \in X$

Fuzzy sets

An ordinary set (or *crisp set*), is defined by its sharp distinction between membership and non membership of that set. In classical set theory, membership of an element x in a set A, considered as some subset of a universe of discourse X, can be defined as:

$$\mu_A(x) = \begin{cases} 1, \text{ } if \text{ } x \in A \\ 0, \text{ } if \text{ } x \notin A \end{cases} \tag{A.1}$$

This can be interpreted as an element x is either a member of set A $(\mu_A(x) = 1)$ or not $(\mu_A(x) = 0)$. In this definition, μ_A is the characteristic function:

$$\mu_A : X \rightarrow \{0,1\} \tag{A.2}$$

For many classifications, however, it is not quite clear whether x belongs to a set A or not. A *fuzzy set*, introduced by (Zadeh 1965), is an extension in that elements are characterized by their grade of membership in the real interval: $\mu_A(x) \in [0,1]$. This generalizes the traditional membership of an element in a set $(x \in A)$ from being binary to being a value (typically) in the unit interval $I = [0,1]$.

A graphical illustration of a membership function for *soft storey* (SS) is illustrated as follows. In Chapter 5, for example, a three-tuple fuzzy set is used, *L*, *M*, *H*. A *soft storey* and *extreme soft story* are defined by the stiffness of the lateral force resisting system in any story being less than 70% and 60% of the stiffness in an adjacent story, respectively. Thus, it can be interpreted that a SS > 70% has a μ_L^{SS} = 1. Similar interpretation can be undertaken and the final three-tuple fuzzification is depicted in Figure A.2.

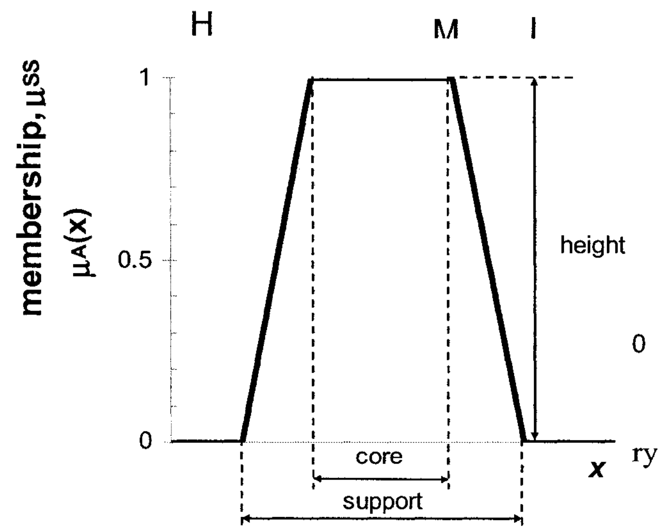

Figure A.3 Height, core and support of a fuzzy set.

Properties of fuzzy sets

The *height* of a fuzzy set A, $hgt(A)$, is defined by:

$$hgt(A) = \sup_{x \in X} \mu_A(x) \tag{A.4}$$

and fuzzy sets with a height equal to 1 are called *normal*. Fuzzy sets called *subnormal* are characterized by $hgt(A) < 1$. The core of a fuzzy set, also referred to as *kernel* or *nucleus*, is a crisp subset of *X*:

$$\text{core}(A) = \{x \in X \mid \mu_A(x) = 1\} \tag{A.5}$$

The *support* of a fuzzy set is also a crisp subset of *X*:

$$\text{supp}(A) = \{x \in X \mid \mu_A(x) > 0\} \qquad \text{(A.6}$$

If the support of a fuzzy set is finite, it is called *compact support*. Figure A.3 shows schematically the height, core and support of a fuzzy set.

Fuzzy modelling

There are two basic approaches for developing a fuzzy inference system (FIS): direct approach and system identification (Yager and Filev 1994). Direct approach is essentially simple and intuitive, but it has inherent limitations. The main limitation is due to the fact that quantitative observations provide an overview of the performance of the system, but do not explicitly determine the structure or parameters of the model. Also, it is often the case that an expert cannot tell linguistically what kind of outcome he expects or what kind of action he takes in a particular situation. As a result, the adequacy of the direct approach is restricted to the boundaries of the expert knowledge.

Another approach for developing a FIS is system identification. In this approach, the FIS is developed based on the input-output data (training data) obtained from the actual system. System identification is predominantly useful when a predetermined model structure based on characteristics of variables is not available. Different reported authors have used the system identification in earthquake damage assessment. Sánchez-Silva and Garcia (2001) used the fuzzy logic to translate uncertainty from the expert provided evaluation data and neural network approach for system identification. Demartinos and Dritsos (2006) used the adaptive network-based fuzzy inferencing system for system identification. Boissonnade and Shah (1985) have used fuzzy sets and Bayesian classifiers.

Aggregation Operations

Aggregation operations on fuzzy sets are operations by which several fuzzy sets are combined in a desirable way to produce a single fuzzy set. A large family of aggregation operators are discussed by different researchers, e.g., Detyniecki (2001), Grabisch et al. (1999), Dubois and Prade (1985), Klir and Yuan (1995). The different aggregators capture two extremes the *minimum* and *maximum* values and intermediate values. The two extreme values of maximum and minimum values are computed using the drastic intersection and drastic union discussed in the t-norm and t-conorm sections, respectively. The t-norm operator are categorized under union operators, the upper bound is $u_{\max}$ and lower bound is computed using $\max$. Similarly, The t-conorm operator are categorized under intersection operators, the lower bound is $i_{\min}$

and upper bound is computed using min. Intermediate values, between min and max are computed using averaging (compensatory) operators. The range of t-norms, averaging operators and t-conorms are shown in Figure A.4.

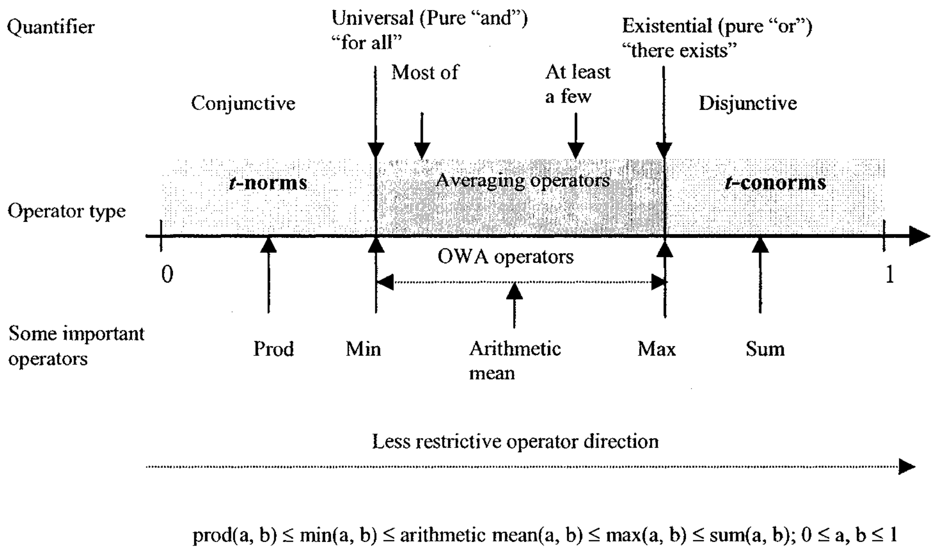

Figure A.4 Common aggregation operators (after Larsen, 2002)

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Appendix B

CanRisk

This appendix is a tutorial to illustrate the use of ***CanRisk*** software.

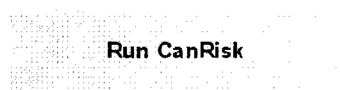
Consider a moment resisting frame (*C1*), located in the City of Vancouver, BC, and the soil type is "B". The building has 5 stories, and the construction date is indicated as 1975, and has a "poor" construction quality. From the *walk down* survey, vertical irregularity and plan irregularity are indicated as present "Yes" and not present "No", respectively. Through the next section, a step by step procedure of ***CanRisk*** will be illustrated. Assume that the building occupancy = 101-1000, Building use = School and Economic impact = Negligible.

Step 1: Start *CanRisk*

Start the ***CanRisk*** program with the option of enabling the Macro. Once the program is opened, the input data need to be inserted manually.

Step 2: Open *CanRisk* user interface

To open the CanRisk form, **Click** Run CanRisk

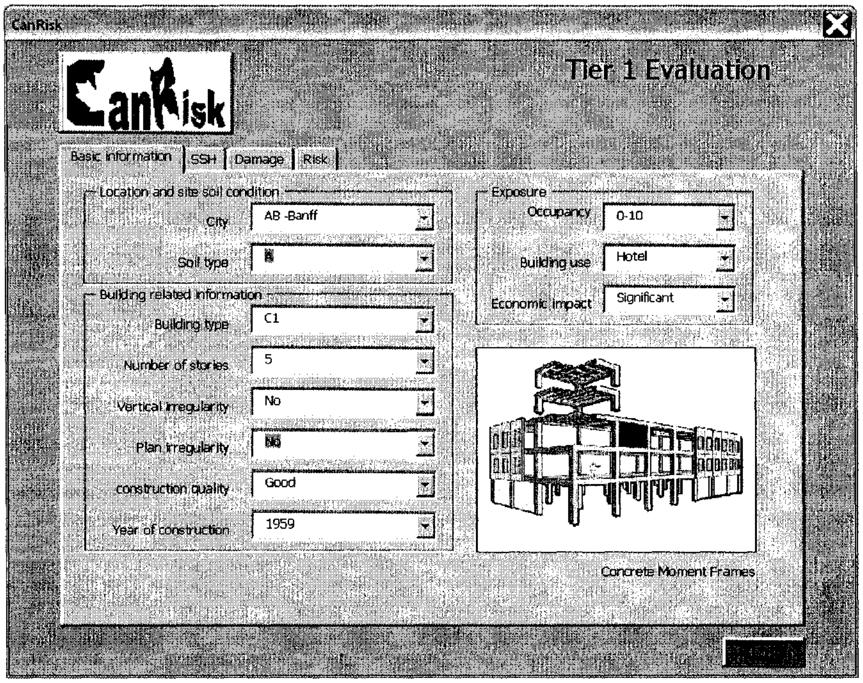


The CanRisk form consists of four tabs:

- Basic information
- SSH
- Damage
- Risk

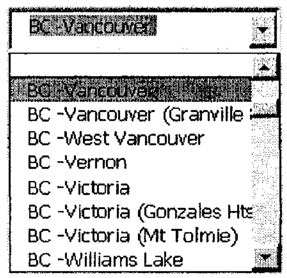
The basic information is put into the *Basic information* tab. And the corresponding SSH and estimated damage will be shown in the *SSH* and *Damage* tabs, respectively. Finally, the estimated risk will be presented in the *risk* tab. The ***CanRisk*** form has the end command button, where at any point during the analysis, it can be invoked to quit and go back the excel spreadsheet.

Given the previous information, adding the Basic information will be done as follows.



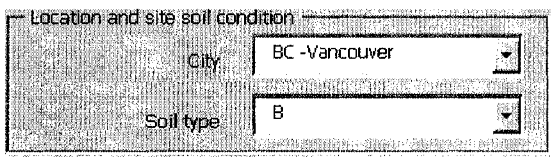
Step 3: Basic information

Locate and site condition



- For **City**, select **BC-Vancouver**
- For **Soil type**, select **B**

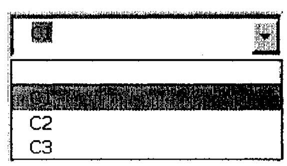
The populated **Locate and site condition** input are shown below.



Building related information

The **Building related information**, is used to collate information that can potentially affect the demand or capacity of the building and structural system.

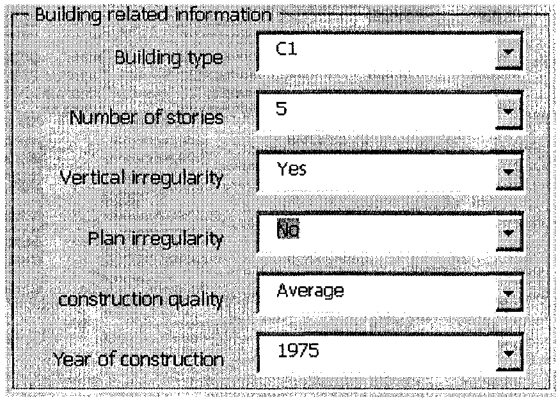
Given the previous information, the inputs for Building related information is:



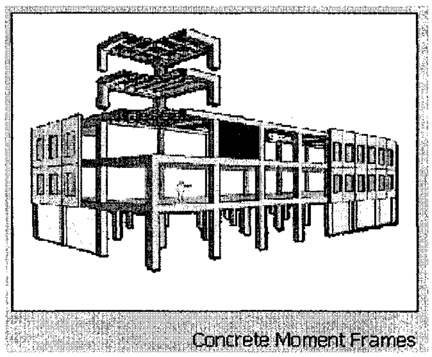
- For **Building type**, select **C1**
- For **Number of stories**, select **5**
- For **Vertical irregularity**, select **Yes**
- For **Plan irregularity**, select **No**
- For **Construction quality**, select **Average**

- For **Year of construction,** select **1975**

The populated **Building related information** inputs are shown below.



Note that, for the building type selected, a schematic of the building and building type labels are provided.

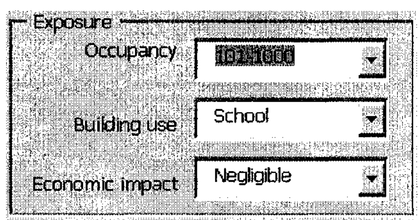


Exposure

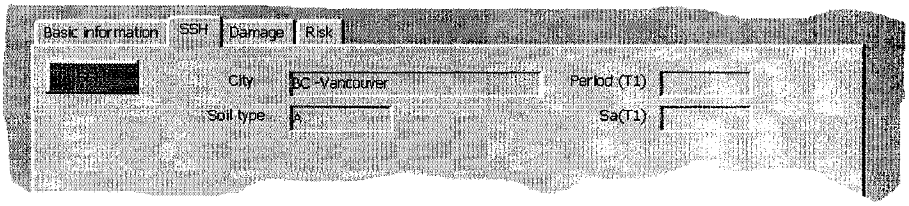
The **Exposure**, is used to collate parameters that are used to compute the consequence of failure.

- For **Occupancy,** select **101-1000**
- For **Building use,** select **School**
- For **Economic impact,** Negligible

The populated **Exposure factors** input are shown below.



To proceed to the next section, quantification of site seismic hazard, **Click** the *SSH* tab. The SSH tab is shown below.

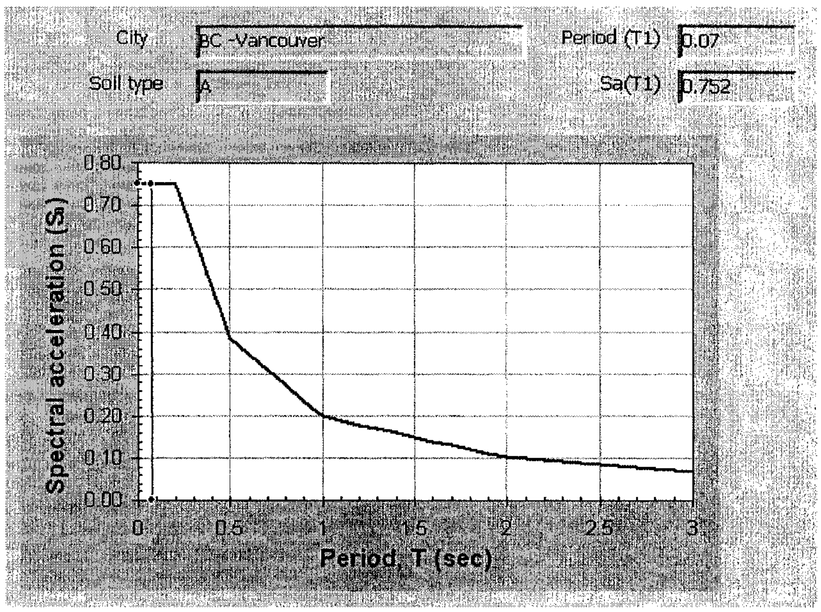


Step 4: Site seismic hazard (SSH)

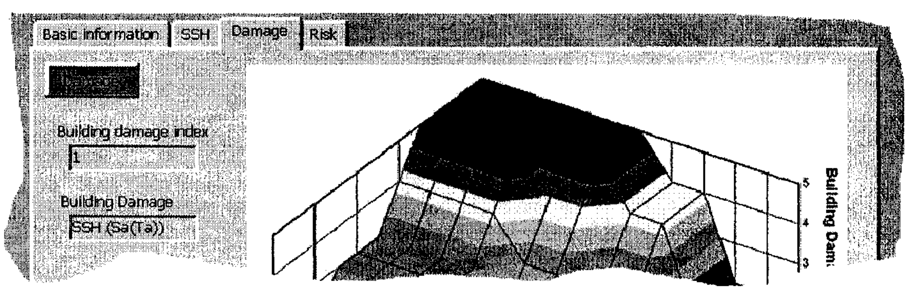
The **Site seismic hazard** is quantified through the Spectral acceleration.

- To compute the spectral acceleration, **Click** SSH 

The SSH tab shows the prevalent City, Soil type, computed period of the structure, $S_a(T_1)$, and a plot of the response spectrum. The fundamental period is computed to be 0.07, and corresponding response spectrum is $S_a(T_1)$ = 0.752.

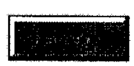


To proceed to the next section, quantification of damage, **Click** the *Damage* tab. The Damage tab is shown below.

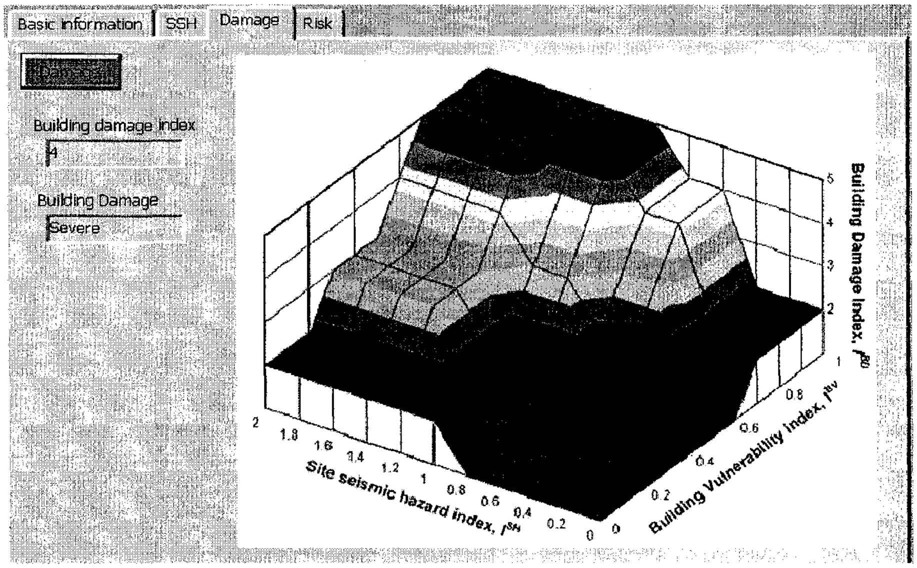


Step 5: Damage

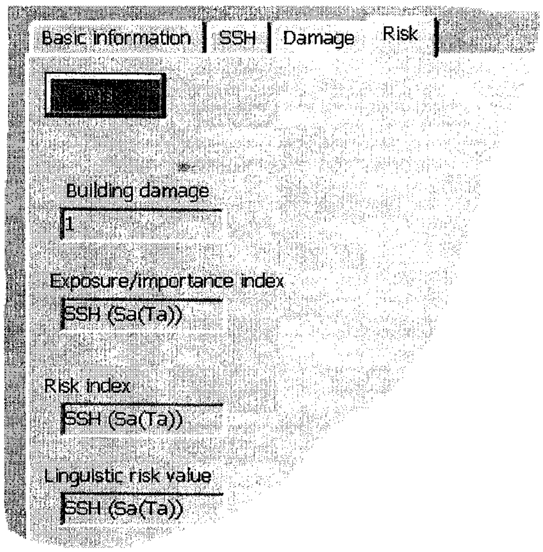
The **Damage** tab is used to quantify damage for the prevalent building vulnerability and SSH.

- To estimate Damage, **Click** Damage 

The estimated damage state is shown below. The building damage state index is 4, and the corresponding linguistic descriptor of the Building damage = "severe".



Form the estimated damage state, to compute the corresponding risk index, select the *Risk* tab.

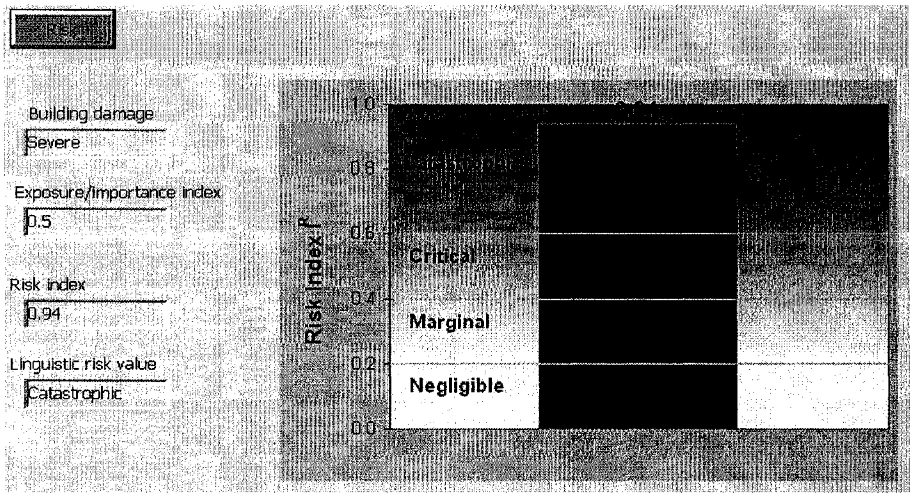


Step 6: Risk

The **Risk** tab is used to quantify risk for the prevalent building vulnerability and SSH.

- To quantify Risk, **Click** Risk 

The exposure/importance index is computed to be 0.5, and final risk index is = 0.94. The corresponding linguistic quantifier is Catastrophic. This building needs further analysis, and can be a candidate for risk management program.


Building damage
Severe
Exposure/Importance index
0.5
Risk index
0.94
Linguistic risk value
Catastrophic
Risk index
1.0
0.8
0.6
0.4
0.2
0.0
Critical
Marginal
Negligible