PROBABILISTIC SEISMIC HAZARD ANALYSIS WITH NONLINEAR SITE RESPONSE AND LIQUEFACTION POTENTIAL EVALUATION FOR DEEP SEDIMENTARY DEPOSITS

by

Md. Zillur Rahman
M. Sc., ITC, University of Twente, The Netherlands, 2005
M. Sc., University of Dhaka, Bangladesh, 1993

A THESIS SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR THE DEGREE OF

DOCTOR OF PHILOSOPHY

in

THE COLLEGE OF GRADUATE STUDIES
(Civil Engineering)

THE UNIVERSITY OF BRITISH COLUMBIA
(Okanagan)

January 2019

© Md. Zillur Rahman, 2019
The following individuals certify that they have read, and recommend to the College of Graduate Studies for acceptance, a thesis/dissertation entitled:

PROBABILISTIC SEISMIC HAZARD ANALYSIS WITH NONLINEAR SITE RESPONSE AND LIQUEFACTION POTENTIAL EVALUATION FOR DEEP SEDIMENTARY DEPOSITS

submitted by  Md. Zillur Rahman  in partial fulfillment of the requirements of the degree of  Doctor of Philosophy (PhD)

Dr. Sumi Siddiqua, School of Engineering

Supervisor
Dr. Ahmad Rteil, School of Engineering

Supervisory Committee Member
Dr. A. S. M. Maksud Kamal, University of Dhaka

Supervisory Committee Member
Dr. Rudolf Seethaler, School of Engineering

University Examiner
Dr. Nicholas Vlachopoulos, Royal Military College of Canada

External Examiner
Abstract

Seismic hazard analysis includes deterministic or probabilistic seismic hazard analysis (DSHA or PSHA), site response analysis, and liquefaction potential evaluation. In the present study, probabilistic ground motion maps were prepared at bedrock condition for Bangladesh by using ground motion prediction equations (GMPEs) as a function of earthquake magnitudes and distances from the sources. The seismic sources have been modeled as background seismicity, crustal fault, and subduction zone. The uncertainties in the source parameters and GMPEs were accounted for using the logic tree approach. The activity rates of the background and regional seismicity sources were estimated using a declustered and complete earthquake catalog. The activity rates of the crustal fault and subduction zone sources were predicted from the geodetic strain rates using the well-accepted relationships.

The $V_s^{30}$ (time-averaged shear wave velocity in the top 30 m) map for Dhaka City was prepared using the relationship between the $V_s^{30}$ and Holocene soil thickness to estimate site coefficients. The $V_s^{30}$-based site coefficients and one-dimensional linear, equivalent-linear, and nonlinear site response analysis approaches have been used to evaluate the site effects of the deep sedimentary deposits in Dhaka City. The one-dimensional nonlinear site response analysis improves the accuracy of the ground motion. The site response analysis has to be performed using the soil profile down to the depth of the bedrock. In areas of thick and soft Holocene deposits, the long period seismic waves of the far-field earthquakes will amplify and the resonance will occur with the natural periods of the high-rise buildings, and consequently, the damage to high-rise buildings will be increased in these areas. The short period seismic waves of the near-field earthquakes will
amplify in the artificially filled shallow valleys of the Pleistocene Terrace and the resonance will occur with the natural periods of the low-rise buildings, and consequently, the damage to the low-rise buildings will be increased in these areas.

In this study, liquefaction hazard map for Dhaka City was prepared using liquefaction potential index (LPI) and cumulative frequency distribution of LPI of different surface geological units, which is an excellent approach to evaluate the liquefaction hazard quantitatively and spatially. The LPI values were calculated using the results of liquefaction potential evaluation that were estimated using Simplified Procedure from standard penetration test blow count (SPT-N).
Earthquake is a common natural disaster in earthquake-prone areas. Ground failure, ground breaking, structural damage, liquefaction, and landslide may occur as direct effects of earthquake and tsunami, flood, and fire may also occur as indirect effects of earthquake. During an earthquake, most of the casualties occur in urban areas due to building collapse. To reduce loss of life and property damage, a risk-based approach should be applied to design structures, prepare regional seismic hazard maps to use in building codes, and develop emergency response system. In this study, the ground shaking of the study area that may occur during an earthquake was estimated more accurately using the possible earthquake sources, distance from the sources, and properties of the soils of the study area. The loss of life and damage to properties can be reduced significantly by designing structures and developing emergency response system using the estimated ground shaking.
Preface

I, Md. Zillur Rahman, prepared all the contents of this dissertation including literature review, seismic source models, probabilistic seismic hazard and site response analyses, site characterization, seismic and liquefaction hazard maps, interpretation of the results and writing the manuscript under the supervision of Dr. Sumi Siddiqua. The co-authors of the published and submitted articles are Dr. Sumi Siddiqua and Dr. A. S. M. Maksud Kamal. They reviewed the manuscripts of the articles and dissertation and provided necessary comments to improve the manuscripts of the articles and dissertation. Most of the contents of this dissertation have been published in five peer-reviewed journal papers and four peer-reviewed conference papers, and two journal papers have been submitted for possible publication. The peer-reviewed journal papers, conference papers and submitted manuscripts are listed below:

- A version of Chapter 3 has been submitted to the Engineering Geology (Elsevier) journal for possible publication titled as “Probabilistic seismic hazard analysis for Bangladesh”(Rahman et al. 2018b).
- A portion of Chapter 3 has been published in the conference proceedings of the 70th Canadian Geotechnical Conference titled as “Probabilistic seismic hazard analysis for Dhaka City, Bangladesh”. Canadian Geotechnical Society (CGS), October 01-04, 2017, City of Ottawa (Rahman et al. 2017b).
- A version of Chapter 4 has been published in the Natural Hazards (Springer Nature) journal titled as “Near-surface shear wave velocity estimation and $V_{s30}$ mapping for Dhaka City, Bangladesh”(Rahman et al. 2018a).
• A portion of Chapter 4 has been published in the conference proceedings of the 11th Canadian Conference on Earthquake Engineering titled as “Shear wave velocity mapping of Dhaka City for seismic hazard assessment”. Canadian Association of Earthquake Engineering, July 21-24, 2015, Victoria (Rahman et al. 2015a).

• A portion of Chapter 4 has been published in the Journal of Applied Geophysics (Elsevier) titled as “Shear wave velocity estimation of the near-surface materials of Chittagong City, Bangladesh for seismic site characterization” (Rahman et al. 2016).

• A portion of Chapter 4 has been published in the conference proceedings of the 68th Canadian Geotechnical Conference titled as “Shear wave velocity estimation using multichannel analysis of surface wave and small scale microtremor measurement for seismic site characterization”. Canadian Geotechnical Society (CGS), September 20-23, 2015, Quebec (Rahman et al. 2015b).

• A portion of Chapter 4 has been published in the Bulletin of Engineering Geology and the Environment (Springer Nature) titled as “Geology and topography based $V_{s}^{30}$ map for Sylhet City of Bangladesh” (Rahman et al. 2018c).

• A version of Chapter 5 has been submitted to the Soil Dynamics and Earthquake Engineering (Elsevier) journal for possible publication titled as “Evaluation of seismic site effects for the deep sedimentary deposits in Dhaka City, Bangladesh” (Rahman and Siddiqua 2018).

• A version of Chapter 6 has been published in the Engineering Geology (Elsevier) journal titled as “Liquefaction hazard mapping by liquefaction potential index for Dhaka city, Bangladesh” (Rahman et al. 2015c).

• A portion of Chapter 6 has been published in the Environmental Earth Sciences (Springer Nature) journal titled as “Evaluation of liquefaction-resistance of soils using standard
penetration test, cone penetration test, and shear-wave velocity data for Dhaka, Chittagong, and Sylhet cities in Bangladesh” (Rahman and Siddiqua 2017a).

- A portion of Chapter 6 has been published in the conference proceedings of the 69th Canadian Geotechnical Conference titled as “Liquefaction resistance evaluation of soils using standard penetration test blow count and shear wave velocity”. Canadian Geotechnical Society (CGS), October 02-04, 2016, City of Vancouver (Rahman and Siddiqua 2016).

- The images in Figures 3.1, 3.2, Figures from 4.1 to 4.16, and Tables 4.1, 4.2 are used with permission from Springer Nature as the author of the articles Rahman and Siddiqua (2017b), Rahman et al. (2018a, 2018c). The images in Figure 5.1, Figures from 6.1 to 6.4, and Tables 6.1, 6.2 are used from Elsevier. Don’t require permission as the author of the article Rahman et al. (2015c).
# Table of Contents

Abstract .................................................................................................................................................. iii  
Lay Summary ......................................................................................................................................... v  
Preface .................................................................................................................................................. vi  
Table of Contents ................................................................................................................................ ix  
List of Tables .......................................................................................................................................... xv  
List of Figures ......................................................................................................................................... xvii  
List of Abbreviations .......................................................................................................................... xxix  
List of Symbols ...................................................................................................................................... xxxi  
Acknowledgements ............................................................................................................................. xxxiv  
Dedication ............................................................................................................................................. xxxvi

**Chapter 1: Introduction** .................................................................................................................. 1  
1.1 Background ...................................................................................................................................... 1  
1.2 Seismic Hazard Analysis .................................................................................................................. 3  
1.3 Seismic Site Characterization ......................................................................................................... 4  
1.4 Site Response Analysis .................................................................................................................... 5  
1.5 Liquefaction Potential Evaluation .................................................................................................. 5  
1.6 Research Objectives ....................................................................................................................... 6  
1.7 Thesis Organization ......................................................................................................................... 6

**Chapter 2: Literature Review** ......................................................................................................... 8  
2.1 Background ...................................................................................................................................... 8  
2.2 Seismic Hazard Analysis ................................................................................................................ 8  
2.3 Seismic Site Characterization ....................................................................................................... 8  
2.4 Site Response Analysis .................................................................................................................. 8  
2.5 Liquefaction Potential Evaluation ................................................................................................. 8
Deterministic Seismic Hazard Analysis (DSHA) .................................................. 8

Probabilistic Seismic Hazard Analysis (PSHA) ...................................................... 9

Identification of Earthquake Sources ................................................................. 11

Identification of Earthquake Magnitudes .......................................................... 11

Identification of Earthquake Distances .............................................................. 13

Ground Motion Intensity Prediction ................................................................. 13

Combined All Information .................................................................................. 14

Deterministic Versus Probabilistic Seismic Hazard Analysis .............................. 16

Ground Motion Prediction Equations (GMPEs) .................................................. 18

Site Response Analysis ...................................................................................... 19

Site Characterization .......................................................................................... 21

Shear Wave Velocity Measurement .................................................................... 22

Modulus Reduction Factor and Damping Ratio .................................................. 24

Liquefaction Hazard Analysis ............................................................................ 25

Simplified Procedure .......................................................................................... 25

Evaluation of Cyclic Stress Ratio (CSR) ............................................................ 26

Evaluation of Cyclic Resistance Ratio (CRR) .................................................... 27

Determination of Factor of Safety ........................................................................ 29

Magnitude Scaling Factor (MSF) .......................................................................... 30

Seismic Factors .................................................................................................... 30

Liquefaction Potential Index (LPI) ........................................................................ 30

Summary ................................................................................................................ 32

Chapter 3: Seismic Source Models and Probabilistic Seismic Hazard Analysis .......... 34
### Chapter 3: Seismic Source Models

3.1 Background .................................................................................................................. 34
3.2 Seismotectonics ........................................................................................................... 35
3.3 Seismic Source Models ............................................................................................... 37
  3.3.1 Background Seismicity Model ............................................................................. 38
    3.3.1.1 Earthquake Catalog .................................................................................. 40
    3.3.1.2 Magnitude Conversion ............................................................................. 40
    3.3.1.3 Declustering ............................................................................................ 41
    3.3.1.4 Catalog Completeness ............................................................................. 42
    3.3.1.5 Gutenberg-Richter Model ....................................................................... 43
  3.3.2 Regional shallow crustal seismicity model ....................................................... 46
  3.3.3 Crustal Fault Source Model ............................................................................... 49
  3.3.4 Crustal Fault Recurrence Models ...................................................................... 53
  3.3.5 Subduction Zone Source Model ........................................................................ 55
  3.3.6 Subduction Zone Recurrence Models .............................................................. 59
3.4 Ground Motion Prediction Equations (GMPEs) .................................................... 61
3.5 Probabilistic Seismic Hazard Analysis (PSHA) ...................................................... 68
  3.5.1 EZ-FRISK software ......................................................................................... 68
  3.5.2 Probabilistic Seismic Hazard Maps ................................................................. 69
3.6 Discussions ................................................................................................................. 79
3.7 Summary ..................................................................................................................... 90

### Chapter 4: Seismic Site Characterization

4.1 Background ................................................................................................................. 91
4.2 Study Area ..................................................................................................................... 93
Chapter 4: Site Response Analysis

4.3 Geology of Dhaka City ................................................................. 94
4.4 Methodology ............................................................................... 98

4.4.1 Downhole Seismic (DS) Method ....................................................... 98
4.4.2 Surface Wave Methods .................................................................... 99
  4.4.2.1 Multichannel Analysis of Surface Waves (MASW) ......................... 100
  4.4.2.2 Small Scale Microtremor Measurement (SSMM) ......................... 104

4.4.3 Empirical Correlations between the $V_s$ and SPT-N ............................ 107

4.5 Results ............................................................................................ 111

4.6 Relationship between $V_s^{30}$ and Holocene Soil Thickness .................... 114

4.7 Preparation of $V_s^{30}$ Map ............................................................... 114

4.8 Discussions ..................................................................................... 119

4.9 Summary ........................................................................................ 123

Chapter 5: Site Response Analysis ............................................................... 124

5.1 Background ..................................................................................... 124

5.2 Mechanical and Dynamic Properties of the Near-surface Soils .............. 126
  5.2.1 Shear Wave Velocity ($V_s$) ............................................................... 128
  5.2.2 Modulus Reduction and Material Damping Curves ......................... 134

5.3 Probabilistic Seismic Hazard Analysis .................................................. 135

5.4 Spectral Matching ............................................................................ 138

5.5 Site Response Analysis ...................................................................... 145
  5.5.1 Site Response Analysis using $V_s^{30}$-based Site Coefficients ............ 145
  5.5.2 One-dimensional Site Response Analysis ......................................... 146
    5.5.2.1 DEEPSOIL Software for Site Response Analysis ....................... 147
5.5.2.2 Linear Site Response Analysis ................................................................. 147
5.5.2.3 Equivalent-linear Site Response Analysis .................................................. 148
5.5.2.4 Nonlinear Site Response Analysis ............................................................ 149
5.5.3 Comparisons of the UHS of Different Site Response Models ......................... 149
5.5.4 Evaluation of Surface UHS using Nonlinear Response Analysis ..................... 150
5.5.5 Nonlinear Analysis using the Acceleration Time History Recorded at the Station of 
KiK-net .................................................................................................................. 155
5.5.6 Developing Site Amplification Function ......................................................... 157
5.5.7 Interpolation of Site coefficients for a Range of Periods ................................. 158
5.5.8 Simplified Approaches to Merge Site effects with PSHA ............................... 162
5.6 Discussions ....................................................................................................... 164
5.7 Summary ........................................................................................................... 167

Chapter 6: Liquefaction Potential Evaluation ....................................................... 169
6.1 Background ....................................................................................................... 169
6.2 Geomorphology and Geology .......................................................................... 171
6.3 Seismotectonics ............................................................................................... 173
6.4 Evaluation of Liquefaction Potential of Geological Units ................................. 175
6.4.1 Establishment of Database ........................................................................... 176
6.4.2 Simplified Procedure for Determination of \( F_L \) against Liquefaction .......... 178
6.4.3 Liquefaction Potential Index (LPI) ............................................................... 179
6.5 Liquefaction Hazard Map ................................................................................ 182
6.6 Discussions ....................................................................................................... 183
6.7 Summary ........................................................................................................... 186
Chapter 7: Conclusions and Recommendations ......................................................... 188

7.1 Summary and Conclusions ................................................................................. 188

7.2 Originality and Contributions ........................................................................... 192

7.3 Limitations and Recommendations ..................................................................... 193

Bibliography ........................................................................................................... 195
List of Tables

Table 3.1 Damage and casualty of Major earthquakes in Bangladesh during the last 256 years .......................................................... 38

Table 3.2 Seismicity parameters (a- and b-values) for seismic source models.............................. 44

Table 3.3 Parameters for crustal fault source models ........................................................................ 52

Table 3.4 Parameters for subduction source models........................................................................... 57

Table 4.1 Existing empirical correlations of different researchers between the $V_s$ and SPT-N (Rahman et al. 2018a). Reprinted with permission of Springer Nature................. 108

Table 4.2 Site classes of subsoils according to the National Earthquake Hazards Reduction Program (NEHRP), USA and Eurocode 8 (Rahman et al. 2018a). Reprinted with permission of Springer Nature............................... 121

Table 5.1 Classification of the soils in the study area................................................................. 129

Table 5.2 The response spectra of the time histories of the following earthquakes are used for spectral matching with the target response spectra for 10% probability of exceedance in 50 years................................................................. 141

Table 5.3 The response spectra of the time histories of the following earthquakes are used for spectral matching with the target response spectra for 2% probability of exceedance in 50 years.................................................................................... 142
Table 6.1 List of historical earthquakes occurred in Bangladesh and NE India (Rahman et al. 2015c). Don’t require permission of Elsevier as the author of this article........ 170

Table 6.2 Liquefaction potential index (LPI) calculated for each SPT profile for an earthquake scenario of $M_w = 7$ having peak horizontal ground acceleration $(a_{max})$ of 0.15g (Rahman et al. 2015c). Don’t require permission of Elsevier as the author of this article. ...................................................... 177
List of Figures

Figure 1.1 Components of seismic hazard analysis. .................................................................3

Figure 3.1 Historical and recent earthquakes (magnitude ≥ 6.5) from 1762 to 2016
(Rahman and Siddiqua 2017b). Reprinted with permission of Springer Nature. ......36

Figure 3.2 Seismotectonic map of Bangladesh and surrounding regions showing epicenters
of earthquakes (declustered catalogue) from 1762 to 2016 (Rahman et al.
2018c). Reprinted with permission of Springer Nature. ........................................................39

Figure 3.3 Background shallow crustal (focal depth ≤ 40 km) seismicity (gridded and
spatially smooth) rate (a-value). ..........................................................................................45

Figure 3.4 Background deep (focal depth > 40 km) seismicity (gridded and spatially
smooth) rate (a-value). .......................................................................................................46

Figure 3.5 Regional shallow crustal seismicity zones with earthquake epicenters of shallow
crustal (focal depth ≤ 40 km). .............................................................................................47

Figure 3.6 Logic tree for background seismicity-based source models of this study. The
assigned branch weights are shown in the parentheses. Shallow crustal ground
motion prediction equations (GMPEs): ASK14- Abrahamson et al. (2014),
BSSA14- Boore et al. (2014), CB14- Campbell and Bozorgnia (2014), CY14-
Chiou and Youngs (2014), and I14- Idriss (2014). Deep seismic GMPEs: BC
Hydro12- Abrahamson et al. (2016), ZH06- Zhao et al. (2006), and AB03-
Atkinson and Boore (2003). ..............................................................................................48
Figure 3.7 Crustal faults that are characterized to use in seismic hazard analysis..................50

Figure 3.8 Logic tree for crustal fault (Dauki fault) source models of this study. The assigned branch weights are shown in the parentheses. Shallow crustal ground motion prediction equations (GMPEs): ASK14- Abrahamson et al. (2014), BSSA14- Boore et al. (2014), CB14- Campbell and Bozorgnia (2014), CY14-Chiou and Youngs (2014), and I14- Idriss (2014). .................................................................51

Figure 3.9. Subduction zones to estimate the seismicity parameters (a- and b-values)............56

Figure 3.10 Logic tree for subduction (Chittagong-Tripura interface section) source models of this study. The assigned branch weights are shown in the parentheses. Subduction zone GMPEs: BC Hydro12- Abrahamson et al. (2016), ZH06-Zhao et al. (2006), and AB03- Atkinson and Boore (2003). .......................................................58

Figure 3.11 Peak ground acceleration (PGA) in g (gravitational acceleration) for 10% probability of exceedance in 50 years (475 years return period) for Bangladesh.....71

Figure 3.12 Peak ground acceleration (PGA) in g (gravitational acceleration) for 2% probability of exceedance in 50 years (2475 years return period) for Bangladesh. .................................................................72

Figure 3.13 Spectral accelerations (SA) in g (gravitational acceleration) for Bangladesh: (a) at 0.2 second (s) period for 10% probability in 50 years; (b) at 0.2s period for 2% probability in 50 years. .................................................................73
Figure 3.14 Spectral accelerations (SA) in g (gravitational acceleration) for Bangladesh: (a) at 0.5s period for 10% probability in 50 years; and (b) at 0.5s period for 2% probability in 50 years. ..............................................................74

Figure 3.15 Spectral accelerations (SA) in g (gravitational acceleration) for Bangladesh: (a) at 1.0 second (s) period for 10% probability in 50 years; (b) at 1.0s period for 2% probability in 50 years. ..............................................................75

Figure 3.16 Spectral accelerations (SA) in g (gravitational acceleration) for Bangladesh: (a) at 2.0s period for 10% probability in 50 years; and (b) at 2.0s period for 2% probability in 50 years. ..............................................................76

Figure 3.17 Spectral accelerations (SA) in g (gravitational acceleration) for Bangladesh: (a) at 5.0 second (s) period for 10% probability in 50 years; (b) at 5.0s period for 2% probability in 50 years. ..............................................................77

Figure 3.18 Spectral accelerations (SA) in g (gravitational acceleration) for Bangladesh: (a) at 10.0s period for 10% probability in 50 years; and (b) at 10.0s period for 2% probability in 50 years. ..............................................................78

Figure 3.19 Uniform hazard spectra for 10%, 5%, and 2% probability of exceedance in 50 years for Dhaka City. ..............................................................81

Figure 3.20 Uniform hazard spectra for 10%, 5%, and 2% probability of exceedance in 50 years for Chittagong City. ..............................................................82

Figure 3.21 Uniform hazard spectra for 10%, 5%, and 2% probability of exceedance in 50 years for Sylhet City. ..............................................................82
Figure 3.22 Uniform hazard spectra for 10%, 5%, and 2% probability of exceedance in 50
years for Comilla City. ............................................................................................................83

Figure 3.23 Uniform hazard spectra for 10%, 5%, and 2% probability of exceedance in 50
years for Mymensigh City .......................................................................................................83

Figure 3.24 Uniform hazard spectra for 10%, 5%, and 2% probability of exceedance in 50
years for Barisal City. .............................................................................................................84

Figure 3.25 Uniform hazard spectra for 10%, 5%, and 2% probability of exceedance in 50
years for Rangpur City. .............................................................................................................84

Figure 3.26 Uniform hazard spectra for 10%, 5%, and 2% probability of exceedance in 50
years for Rajshahi City. ............................................................................................................85

Figure 3.27 Uniform hazard spectra for 10%, 5%, and 2% probability of exceedance in 50
years for Khulna City. .............................................................................................................85

Figure 3.28 Magnitude-distance-epsilon deaggregation (mean magnitude 7.09 M\text{w}, mean
distance 114.15 km) for Dhaka city. Peak ground acceleration (PGA) is 0.144
g for 10% probability of exceedance in 50 years. ................................................................87

Figure 3.29 Magnitude-distance-epsilon deaggregation (mean magnitude 8.02 M\text{w}, mean
distance 101.60 km) for Chittagong city. Peak ground acceleration (PGA) is
0.56 g for 10% probability of exceedance in 50 years. ..........................................................88

Figure 3.30 Magnitude-distance-epsilon deaggregation (mean magnitude 7.5 M\text{w}, mean
distance 98.25 km) for Sylhet city. Peak ground acceleration (PGA) is 0.3 g for
10% probability of exceedance in 50 years. ........................................................................89
Figure 4.1 Surface geological map of Dhaka City showing the locations of downhole seismic (DS), multichannel analysis of surface waves (MASW) and small scale microtremor measurement (SSMM) surveys and standard penetration test (SPT) boreholes (Rahman et al. 2018a). Reprinted with permission of Springer Nature.

Figure 4.2 Subsurface geological classification of 50 boreholes in Dhaka City. FS, FC, HS, HC, PS and PC stand for Filling sand, Filling clay, Holocene sand, Holocene clay, Plio-Pleistocene sand and Plio-Pleistocene clay, respectively (Rahman et al. 2018a). Reprinted with permission of Springer Nature.

Figure 4.3 Typical shear wave velocity ($V_s$) profiles of downhole seismic (DS), surface waves (MASW and SSMM), and standard penetration test blow count (SPT-N) at (a) BH-4 and (b) BH-6 in Dhaka City (Rahman et al. 2018a). Reprinted with permission of Springer Nature.

Figure 4.4 Typical shear wave velocity ($V_s$) profiles of downhole seismic (DS), surface waves (MASW and SSMM), and standard penetration test blow count (SPT-N) at BH-8 in Dhaka City (Rahman et al. 2018a). Reprinted with permission of Springer Nature.

Figure 4.5 Layouts of multichannel analysis of surface waves (MASW) and small scale microtremor analysis methods (Rahman et al. 2018a). Reprinted with permission of Springer Nature.
Figure 4.6 Dispersion curve (top) and shear wave velocity (Vs) structure (bottom) of multichannel analysis of surface waves (MASW) data (Rahman et al. 2018a). Reprinted with permission of Springer Nature. .............................................................. 104

Figure 4.7 Dispersion curve (top) and shear wave velocity (Vs) structure (bottom) of both multichannel analysis of surface waves (MASW) and small scale microtremor measurement (SSMM) data (Rahman et al. 2018a). Reprinted with permission of Springer Nature. ........................................................................ 106

Figure 4.8 Empirical correlations of the present study along with the existing correlations between the Vs and SPT-N for all soils of Dhaka City (Rahman et al. 2018a). Reprinted with permission of Springer Nature. ......................................................... 109

Figure 4.9 Empirical correlations of the present study along with the existing correlations between the Vs and SPT-N for all sandy soils of Dhaka City (Rahman et al. 2018a). Reprinted with permission of Springer Nature. ......................................................... 109

Figure 4.10 Empirical correlations of the present study along with the existing correlations between the Vs and SPT-N for all clayey soils of Dhaka City (Rahman et al. 2018a). Reprinted with permission of Springer Nature. ......................................................... 110

Figure 4.11 Verification of the empirical equations of all soils, sandy soils and clayey soils to select the equation for the prediction of the $V_s^{30}$ using the SPT-N (Rahman et al. 2018a). Reprinted with permission of Springer Nature. ......................................................... 111

Figure 4.12 Map showing time-averaged shear wave velocity in the top 30 m ($V_s^{30}$) that is estimated at different sites of Dhaka City using downhole seismic (DS), multichannel analysis of surface waves (MASW), small scale microtremor...
measurement (SSMM) methods and correlation between the shear wave velocity (Vs) and SPT-N (Rahman et al. 2018a). Reprinted with permission of Springer Nature. 

Figure 4.13 Model for the prediction of the $V_s^{30}$ using the Holocene soil thickness for Dhaka City (Rahman et al. 2018a). Reprinted with permission of Springer Nature.

Figure 4.14 The Holocene soil thickness map of Dhaka City (Rahman et al. 2018a). Reprinted with permission of Springer Nature.

Figure 4.15 The $V_s^{30}$ map of Dhaka City (Rahman et al. 2018a). Reprinted with permission of Springer Nature.

Figure 4.16 Site class map of Dhaka City based on the $V_s^{30}$ according to the National Earthquake Hazards Reduction Program (NEHRP), USA and Eurocode 8 (EC 8) (Rahman et al. 2018a). Reprinted with permission of Springer Nature.

Figure 5.1 Surface geological map of Dhaka City showing geotechnical borehole locations and ground response analysis sites (Rahman et al. 2015c). Don’t require permission of Elsevier as the author of this article.

Figure 5.2 Typical soil profiles at Pleistocene terrace in Dhaka City.

Figure 5.3 Typical soil profiles at Pleistocene terrace, and (b) Holocene floodplain in Dhaka City.
Figure 5.4 Shear wave velocity (Vs) profiles at 10 borehole sites (a) BH-01, BH-02, and BH-07, (b) BH-04, BH-05, and BH-09, (c) BH-03, BH-06, and BH-10, and (d) BH-08 (CDMP 2009).………………………………………………………………………………………………133

Figure 5.5 Example of normalized modulus reduction, material damping, and shear strength curves with target strength (effective stress state with water table at ground surface). The reference curves are from Darendeli (2001). The curves are for clay at a depth of 11.3 m, where lateral earth pressure, plasticity index, over consolidation ratio, loading frequency, and number of loading cycles are 0.83, 30, 2, 1, and 10, respectively…………………………………………………………………………………………………………………………………134

Figure 5.6 Uniform hazard spectra (UHS) for bedrock condition \( V_{s30} = 760 \text{ m/s} \) and for ground surface using the \( V_{s30} \)-based site coefficients of the NEHRP for 10 % probability of exceedance in 50 years at 10 different borehole sites in Dhaka City………………………………………………………………………………………………………………………………………136

Figure 5.7 Uniform hazard spectra (UHS) for bedrock condition \( V_{s30} = 760 \text{ m/s} \) and for ground surface using the \( V_{s30} \)-based site coefficients of the NEHRP for 5 % probability of exceedance in 50 years at 10 different borehole sites in Dhaka City………………………………………………………………………………………………………………………………………137

Figure 5.8 Uniform hazard spectra (UHS) for bedrock condition \( V_{s30} = 760 \text{ m/s} \) and for ground surface using the \( V_{s30} \)-based site coefficients of the NEHRP for 2 % probability of exceedance in 50 years at 10 different borehole sites in Dhaka City………………………………………………………………………………………………………………………………………137
Figure 5.9 Spectral matching of the response spectrum of the time history with target response spectra for 10% probability of exceedance in 50 years. Initial is the response spectrum of the time history and step 4 is the matched response spectrum. .......................................................... 138

Figure 5.10 Spectral matching of the response spectrum of the time history with target response spectra for 2% probability of exceedance in 50 years. Initial is the response spectrum of the time history and step 4 is the matched response spectrum. .......................................................... 139

Figure 5.11 Initial time history (solid blue) form the PEER NGA WEST2 database and match time history (dashed brown line) for 10% probability of exceedance in 50 years. .......................................................... 140

Figure 5.12 Initial time history (solid green line) form the PEER NGA WEST2 database and match time history (dashed orange line) for 2% probability of exceedance in 50 years. .......................................................... 140

Figure 5.13 Response spectra of 14 time histories from 3 earthquakes (Table 5.1) with target response spectrum for 10% probability of exceedance in 50 years at BH-03 site. .......................................................... 143

Figure 5.14 Matched response spectra of 14 time histories with target response spectrum for 10% probability of exceedance in 50 years at BH-03 site................................. 144

Figure 5.15 Response spectra of 14 time histories from 6 earthquakes (Table 5.2) with target response spectrum for 2% probability of exceedance in 50 years at BH-03 site.......................................................... 144
Figure 5.16 Matched response spectra of 14 time histories with target response spectrum for 2 % probability of exceedance in 50 years at BH-03 site.................................145

Figure 5.17 Uniform hazard spectra (UHS) at ground surface using $V_s^{30}$-based site coefficients and UHS at ground surface using linear, equivalent-linear, and nonlinear ground response analysis at BH-03 site using the soil profile down to a depth of 303m at which the $V_s = 760$ m/s.........................................................148

Figure 5.18 Uniform hazard spectra (UHS) at bedrock condition ($V_s^{30} = 760$ m/s) using probabilistic seismic hazard analysis, UHS at ground surface using $V_s^{30}$-based site coefficients, nonlinear ground response analysis using different depths of soil profiles at BH-03 site for 10 % probability of exceedance in 50 years. ............151

Figure 5.19 Uniform hazard spectra (UHS) at ground surface for 10 % probability of exceedance in 50 years using nonlinear (NL) ground response analysis at 10 borehole (BH) sites. The depth where shear wave velocity ($V_s$) is equal to or more than 760 m/s is used as soil profile depth. The UHS for bedrock condition ($V_s^{30} = 760$ m/s) is at BH-03 using probabilistic seismic hazard analysis (PSHA) for 10% probability of exceedance in 50 years.........................152

Figure 5.20 Uniform hazard spectra (UHS) at ground surface for 2 % probability of exceedance in 50 years using nonlinear (NL) ground response analysis at 10 borehole (BH) sites. The depth, where shear wave velocity ($V_s$) is equal to or more than 760 m/s, is used as soil profile depth. The UHS for bedrock condition ($V_s^{30} = 760$ m/s) is at BH-03 using probabilistic seismic hazard analysis (PSHA) for 2% probability of exceedance in 50 years.........................152
Figure 5.21 (a) Shear wave velocity profiles at BH-09 in Dhaka and at FKSH10 station of KiK-net in Japan, (b) uniform hazard spectrum (UHS) at BH-09 in Dhaka and the response spectra of the surface and borehole seismographs at FKSH10 of KiK-net station in Japan................................................................. 156

Figure 5.22 Site amplification of the reference ground motion at borehole site BH-10 for (a) peak ground acceleration (PGA), and for ground acceleration at spectral periods of (b) 0.05 s, (c) 0.1 s, and (d) 0.2 s. The GRA and SS14 stand for ground response analysis and model of Seyhan and Stewart (2014), respectively. .............................................................. 159

Figure 5.23 Site amplification of the reference ground motion at borehole site BH-10 and for ground acceleration at spectral periods of (e) 0.5s and (f) 1.0 s. The GRA and SS14 stand for ground response analysis and model of Seyhan and Stewart (2014), respectively................................................................. 160

Figure 5.24 Interpolation site coefficients at borehole site BH-10 for a period range of 0.01 to 10 s. The GRA and SS14 stand for ground response analysis and model of Seyhan and Stewart (2014), respectively................................................................. 161

Figure 5.25 Merging the nonlinear site amplification with the rock ground motion using hybrid, modified hybrid, and convolution approaches at borehole site BH-10 for (a) peak ground acceleration (PGA), (b) spectral acceleration at 0.2 s, and (c) spectral acceleration at 1.0 s................................................................. 163

Figure 6.1 Location map of the study area (Rahman et al. 2015c). Don’t require permission of Elsevier as the author of this article................................................................. 172
Figure 6.2 Surface geological map of Dhaka City with borehole locations (Rahman et al. 2015c). Don’t require permission of Elsevier as the author of this article. 174

Figure 6.3 Cumulative frequency distributions of Liquefaction Potential Index (LPI) for three zones of Dhaka City. Number of SPT profiles used in each zone is shown in parentheses of the legend (Rahman et al. 2015c). Don’t require permission of Elsevier as the author of this article. 182

Figure 6.4 Liquefaction hazard map of Dhaka City. Liquefaction hazard have been categorized as very low for LPI = 0; low for 0 < LPI ≤ 5; high for 5 < LPI ≤ 15 and very high for LPI >15. The 8%, 50% and 72% of areas in Zone 1, Zone 2 and Zone 3 respectively, will show surface effects of liquefaction for a scenario earthquake of M 7 having peak horizontal ground acceleration of 0.15g (Rahman et al. 2015c). Don’t require permission of Elsevier as the author of this article. 184
## List of Abbreviations

<table>
<thead>
<tr>
<th>Abbreviation</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>AMT</td>
<td>Array Microtremor Measurements</td>
</tr>
<tr>
<td>BH</td>
<td>Borehole</td>
</tr>
<tr>
<td>BPT</td>
<td>Becker Penetration Test</td>
</tr>
<tr>
<td>BSSC</td>
<td>Building Seismic Safety Council</td>
</tr>
<tr>
<td>CDF</td>
<td>Cumulative Distribution Function</td>
</tr>
<tr>
<td>CDMP</td>
<td>Comprehensive Disaster Management Program</td>
</tr>
<tr>
<td>CMT</td>
<td>Global Centroid Moment Tensor</td>
</tr>
<tr>
<td>CPT</td>
<td>Cone Penetration Test</td>
</tr>
<tr>
<td>CRR</td>
<td>Cyclic Resistance Ratio</td>
</tr>
<tr>
<td>CS</td>
<td>Crosshole Seismic</td>
</tr>
<tr>
<td>CSR</td>
<td>Cyclic Stress Ratio</td>
</tr>
<tr>
<td>D</td>
<td>Material Damping</td>
</tr>
<tr>
<td>DS</td>
<td>Downhole Seismic</td>
</tr>
<tr>
<td>DSHA</td>
<td>Deterministic Seismic Hazard Analysis</td>
</tr>
<tr>
<td>EMS</td>
<td>European macroseismic scale</td>
</tr>
<tr>
<td>FEMA</td>
<td>Federal Emergency Management Agency</td>
</tr>
<tr>
<td>GIS</td>
<td>Geographic Information System</td>
</tr>
<tr>
<td>GMPEs</td>
<td>Ground Motion Prediction Equations</td>
</tr>
<tr>
<td>ISC-GEM</td>
<td>International Seismological Center- Global Earthquake Model</td>
</tr>
<tr>
<td>KIK-net</td>
<td>Kiban Kyoshin network</td>
</tr>
<tr>
<td>LPI</td>
<td>Liquefaction Potential Index</td>
</tr>
<tr>
<td>Abbreviation</td>
<td>Full Form</td>
</tr>
<tr>
<td>--------------</td>
<td>-----------</td>
</tr>
<tr>
<td>MASW</td>
<td>Multichannel Analysis of Surface Waves</td>
</tr>
<tr>
<td>MR</td>
<td>Modulus Reduction</td>
</tr>
<tr>
<td>NCEER</td>
<td>National Center for Earthquake Engineering Research</td>
</tr>
<tr>
<td>NEHRP</td>
<td>National Earthquake Hazards Reduction Program</td>
</tr>
<tr>
<td>NIED</td>
<td>National Research Institute for Earth Science and Disaster Prevention</td>
</tr>
<tr>
<td>PDF</td>
<td>Probability Density Function</td>
</tr>
<tr>
<td>PEER NGA</td>
<td>Pacific Earthquake Engineering Research Center’s Next Generation Attenuation for the Western United States</td>
</tr>
<tr>
<td>WEST</td>
<td>Generation Attenuation for the Western United States</td>
</tr>
<tr>
<td>PGA</td>
<td>Peak Ground Acceleration</td>
</tr>
<tr>
<td>PSHA</td>
<td>Probabilistic Seismic Hazard Analysis</td>
</tr>
<tr>
<td>P-wave</td>
<td>Primary Wave</td>
</tr>
<tr>
<td>SA</td>
<td>Spectral Acceleration</td>
</tr>
<tr>
<td>SASW</td>
<td>Spectral Analysis of Surface Waves</td>
</tr>
<tr>
<td>SPT</td>
<td>Standard Penetration Test</td>
</tr>
<tr>
<td>SPT-N</td>
<td>Standard Penetration Test Blow Count</td>
</tr>
<tr>
<td>SSMM</td>
<td>Small Scale Microtremor Measurement</td>
</tr>
<tr>
<td>S-wave</td>
<td>Shear Wave</td>
</tr>
<tr>
<td>UHS</td>
<td>Uniform Hazard Spectra</td>
</tr>
<tr>
<td>USGS</td>
<td>United States Geological Survey</td>
</tr>
</tbody>
</table>
## List of Symbols

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Area of the fault slip</td>
</tr>
<tr>
<td>$a_{max}$</td>
<td>Maximum horizontal acceleration at the ground surface</td>
</tr>
<tr>
<td>cm</td>
<td>Centimeter</td>
</tr>
<tr>
<td>$c(\omega)$</td>
<td>Phase velocity of waves</td>
</tr>
<tr>
<td>D</td>
<td>Average slip on the fault plane</td>
</tr>
<tr>
<td>Dip</td>
<td>Fault dip in degrees</td>
</tr>
<tr>
<td>$F_L$</td>
<td>Factor of safety</td>
</tr>
<tr>
<td>$F_M(m)$</td>
<td>Cumulative distribution function (CDF) for M (earthquake magnitude)</td>
</tr>
<tr>
<td>$f_M(m)$</td>
<td>Probability density function (CDF) for M (earthquake magnitude)</td>
</tr>
<tr>
<td>g</td>
<td>Acceleration of gravity</td>
</tr>
<tr>
<td>km</td>
<td>Kilometer</td>
</tr>
<tr>
<td>km/s</td>
<td>Kilometer/second</td>
</tr>
<tr>
<td>lnIM</td>
<td>Natural log of ground motion intensity measures</td>
</tr>
<tr>
<td>M</td>
<td>Earthquake magnitude</td>
</tr>
<tr>
<td>m</td>
<td>Meter</td>
</tr>
<tr>
<td>m/s</td>
<td>Meter/second</td>
</tr>
<tr>
<td>$M_b$</td>
<td>Body wave magnitude</td>
</tr>
<tr>
<td>$M_{c\text{ or max}}$</td>
<td>Characteristic or maximum magnitude of earthquake</td>
</tr>
<tr>
<td>$M_L$</td>
<td>Local magnitude</td>
</tr>
<tr>
<td>$M_o$</td>
<td>Seismic moment</td>
</tr>
<tr>
<td>$M_s$</td>
<td>Surface wave magnitude</td>
</tr>
<tr>
<td>Symbol</td>
<td>Definition</td>
</tr>
<tr>
<td>--------</td>
<td>------------</td>
</tr>
<tr>
<td>$M_w$</td>
<td>Moment magnitude</td>
</tr>
<tr>
<td>$N$</td>
<td>Uncorrected standard penetration test blow count (SPT-N)</td>
</tr>
<tr>
<td>$(N_{160})$</td>
<td>Corrected standard penetration test blow count (SPT-N)</td>
</tr>
<tr>
<td>NS</td>
<td>Normal slip</td>
</tr>
<tr>
<td>$r_d$</td>
<td>Stress reduction coefficient</td>
</tr>
<tr>
<td>$R_{JB}$</td>
<td>Closest distance (km) to the surface projection of the ruptured plane</td>
</tr>
<tr>
<td>$R_{RUP}$</td>
<td>Closest distance (km) to the ruptured plane</td>
</tr>
<tr>
<td>RS</td>
<td>Reverse slip</td>
</tr>
<tr>
<td>$R_x$</td>
<td>Horizontal distance (km) from top edge of rupture</td>
</tr>
<tr>
<td>$R_{y0}$</td>
<td>Horizontal distance (km) off the end of the rupture measure parallel to strike</td>
</tr>
<tr>
<td>S</td>
<td>Slip rate on the fault plane</td>
</tr>
<tr>
<td>SRL</td>
<td>Surface rupture length (km)</td>
</tr>
<tr>
<td>SS</td>
<td>Strike slip</td>
</tr>
<tr>
<td>$V_s$</td>
<td>Shear wave velocity</td>
</tr>
<tr>
<td>$V_{s30}$</td>
<td>Time-averaged shear wave velocity in the top 30 m</td>
</tr>
<tr>
<td>$W$</td>
<td>Down dip rupture width</td>
</tr>
<tr>
<td>$Z_I$</td>
<td>Depth to $V_s = 1.0 \text{ km/s}$ at the site</td>
</tr>
<tr>
<td>$Z_{TOR}$</td>
<td>Depth to top of rupture (km)</td>
</tr>
<tr>
<td>$\lambda_m$</td>
<td>Annual rate of earthquake greater than magnitude $m$</td>
</tr>
<tr>
<td>$\sigma_0$</td>
<td>Total vertical overburden stress</td>
</tr>
<tr>
<td>$\sigma'_0$</td>
<td>Effective vertical overburden stress</td>
</tr>
<tr>
<td>$\delta$</td>
<td>Fault dip angle</td>
</tr>
</tbody>
</table>
\( \lambda \)  
Rake angle on the plane of the fault

\( \mu \)  
Rigidity or shear modulus of the earth’s crust

\( \tau_{av} \)  
Average cyclic shear stress

\( \omega \)  
Angular frequency
Acknowledgements

I would like to express my sincere gratitude to my supervising professor Dr. Sumi Siddiqua for her continuous guidance and support throughout my PhD study and related research. It was not possible for me to complete this research without her inspiration, dedication, and persistent help.

I would like to thank my committee members Dr. Ahmad Rteil and Dr. A. S. M. Maksud Kamal for reviewing this dissertation and providing their valuable and helpful comments to shape this dissertation.

I would like to acknowledge the University of British Columbia for supporting this study through University Graduate Fellowship (UGF) and for procuring the EZ-FRISK software to carry out this research.

I would also like to acknowledge the Pacific Earthquake Engineering Research Center (PEER) and National Research Institute for Earth Science and Disaster Prevention (NIED) for giving access to use the strong ground motion data from NGA WEST2 and K-NET & KiK NET, respectively. I would like to express my sincere gratitude to the developer team of DEEPSOIL for providing the software freely to carry out this research.

I would like to acknowledge Comprehensive Disaster Management Program (CDMP), Geology Department of Dhaka University, Geological Survey of Bangladesh (GSB), Asian Disaster Preparedness Center (ADPC), and OYO International Corporation for their support to collect different types of data related to seismic hazard assessment.
I would like to thank the members of our research group, administrative staff and my friends who always inspired and helped me during my research in the University of British Columbia.

Special thanks to my beloved wife, Nusrat for her encouragement and support. I would like to thank my beloved daughter Saraf and son Arveen who were always curious to know the progress of my research, number of articles I have published so far, and how many articles I need to publish for completing this research. I would like to thank my parents who installed in me the values of excellent and provided me all the opportunities to reach my goal.
Dedication

This dissertation is dedicated to my parents, wife, daughter, and son for their love, continuous support, and encouragement.
Chapter 1: Introduction

1.1 Background

Earthquake is a common natural hazard in the seismically active zones on Earth. Tectonic plate boundaries are responsible for the occurrence of earthquakes. There are three types of plate boundaries: convergent, divergent, and transform (Meissner 2002). At convergent boundary, two plates are moving towards each other; at divergent boundary, two plates are moving away from each other; and at transform boundary, two plates are sliding past each other.

Earthquakes also occur along large intraplate active faults. Faults are mainly classified as: 1) dip slip where the displacement is mostly vertical (e.g., normal fault, reverse fault, and thrust fault), 2) strike slip where the displacement is mostly horizontal (e.g., strike slip fault) (Park 1997).

Earthquake vibrates the earth due to sudden release of elastic energy from the strained rocks of the earth. The released elastic energy transmits outward through the rocks as seismic waves. There are two types of seismic waves: body waves and surface waves (Shearer 2009). The body waves are of two types: compressional waves (P-waves) and shear waves (S-waves). The P-waves move by alternating compressions (pushes) and dilations (pulls). The P-waves are the fastest waves and arrive first at the seismic stations. These waves can travel through solid material and fluid. The S-waves travel by alternating transverse motions. The S-waves are slower than the P-waves and arrive second at the seismic stations. These waves can travel through solid material only.

The surface waves are also of two types: Rayleigh waves and Love waves. Surface waves are the slowest waves and arrive last at the seismic stations. Surface waves cause most of the
destruction and damage associated with earthquakes. The Rayleigh waves travel along ground surface in the direction of wave propagation like water waves in lakes and oceans. The Love waves travel along the ground surface by transverse horizontal motions.

The seismic waves that carry the elastic energy cause ground failure, ground breaking, structural damage, liquefaction, and landslide as direct effects of earthquake. Tsunami, flood, and fire may also occur as indirect effects of earthquake. Consequently, the damaging earthquake causes casualty and property damage that impede economic growth and development of a country. Therefore, seismic hazard analysis is required for an earthquake prone area to estimate seismic ground motion parameters, such as peak ground acceleration, peak ground velocity, peak ground displacement, and spectral acceleration of different spectral periods for the construction of earthquake resilient structures. The seismic risk of an area can be reduced significantly by seismic design of structures based on the intensity of the ground motion parameters.

The results of seismic hazard analysis are used in various applications, such as 1) site specific hazard analysis and design of structures and facilities; 2) preparation of regional seismic hazard maps to use in building codes; 3) social and financial loss estimations (Global Ground Motion Prediction Equations Program, accessed on November 10, 2015).

The national building code of a country is regularly updated to include the updated seismic ground motion parameters that are estimated using the latest available data and research methods for the construction of earthquake resilient structures. The purpose of the building codes is to provide minimum requirements for design and construction of structures that withstand the effects of seismic ground motions. Therefore, seismic hazard analysis using the latest data and science is necessary for seismically vulnerable regions to reduce seismic risk.
1.2 Seismic Hazard Analysis

The seismic source, wave propagation path, and local site that influence the seismic ground motions need to be characterized to perform seismic hazard analysis for a site (Figure 1.1). Seismic hazard analysis may consist of one of the following approaches: 1) probabilistic seismic hazard analysis (PSHA) or possibly deterministic seismic hazard analysis (DSHA); 2) PSHA or/and DSHA followed by dynamic site response analysis; and 3) dynamic site response analysis only (BSSC 2015).

![Figure 1.1 Components of seismic hazard analysis.]

In the DSHA method, the intensity of seismic ground motion distribution for a site is estimated using a certain scenario earthquake (Reiter 1990, Kramer 1996). Conversely, in the PSHA method,
all possible sources, magnitudes and distances are considered to estimate the intensity of ground motion distribution (Cornell 1968, Kramer 1996). Both deterministic and probabilistic methods can be performed to complement each other for providing additional insights to the seismic hazard and risk problems for decision making purposes (McGuire 2001).

The deterministic and probabilistic ground motion distributions are commonly estimated as function of earthquake magnitude and distance from the site using ground motion prediction equations (GMPEs) at the top of the bedrock, where time-averaged shear wave velocity in the top 30 m ($V_{s30}$) of a site is equal to or greater than 760 m/s. Then, the site response analysis is performed in order to estimate the ground motion distributions at the ground surface of a site by using dynamic properties of the soils that are estimated using geotechnical site investigation, such as shear wave velocity, density, modulus reduction, and material damping curves.

1.3 Seismic Site Characterization

The dynamic properties of the soils can be expressed in terms of average shear wave velocity. Therefore, the time-averaged shear wave velocity in the top 30 m ($V_{s30}$) is considered as an important parameter for seismic site characterization to estimate the amplification factor of seismic waves during an earthquake (Borcherdt, 1994; Anderson et al., 1996; BSSC, 1998; Park and Elrick, 1998). The shear wave velocity of the near-surface geological materials can be estimated using various seismic field investigations, such as downhole seismic (DS), crosshole seismic (CS), multichannel analysis of surface waves (MASW), etc. Other geotechnical properties, such as density, plasticity index, modulus reduction and material damping curves, are also required to perform one-dimensional site response analysis.
1.4 Site Response Analysis

As a simplified procedure, a common practice in the last three decades for ground motion prediction at the ground surface is simply multiplying the rock ground motion by the site amplification factor that is determined using the $V_s^{30}$ (Borcherdt 1994, BSSC 1994, 2015). However, the state-of-practice is to perform site response analysis for the ground motion prediction at the surface using the properties of the soil profile above the bedrock and the ground motion at the bedrock (Cramer 2003, Bazzurro and Cornell 2004a, 2004b, Kaklamanos et al. 2015). It is observed that the ground motion prediction at the ground surface using one-dimensional site response analysis improves the accuracy of the ground motion (Groholski et al. 2016, Stewart et al. 2017).

1.5 Liquefaction Potential Evaluation

During cyclic movement of earthquake, liquefaction can be a potential seismic hazard in the Holocene loose and poorly graded sands and low plastic silts existed at shallow depth (< 20 m) below the water table. Soil liquefaction is the transformation of the granular soils from solid state to a liquefied state as a consequence of increased pore water pressure and reduced effective stress during cyclic loading (Marcuson 1978). The structures founded on the liquefiable soils may experience bearing capacity failures due to sudden loss of effective stress, settlement and lateral spreading. Therefore, it is also required to evaluate the liquefaction potential of the Holocene cohesionless soils in the seismically active regions for seismic design of structures to reduce seismic risk.
1.6 Research Objectives

The overall objective of this dissertation is to perform probabilistic seismic hazard analysis with nonlinear site response to estimate seismic ground motion for soft and thick sedimentary deposits down to a depth where shear wave velocity ($V_s$) is equal to 760 m/s. In order to accomplish the overall objective of the research, the following sub-objectives are defined:

1. To develop a new seismic source model for the study area from the seismic sources;
2. To perform probabilistic seismic hazard analysis (PSHA) using logic tree approach to account for the uncertainties in ground motion prediction;
3. To develop a model for the prediction of time-averaged shear wave velocity in the top 30 m ($V_{s30}$) using the thickness of the Holocene sandy and silty soils;
4. To perform one-dimensional nonlinear site response analysis for soft and thick sedimentary deposits down to a depth where shear wave velocity ($V_s$) is equal to 760 m/s to develop a new seismic site coefficient model;
5. To evaluate the liquefaction hazard and to prepare liquefaction hazard map.

1.7 Thesis Organization

This dissertation is prepared based on the manuscripts that are submitted and published in journals and conference proceedings. The dissertation contains seven chapters. Chapter 1 includes the background, a brief introduction to the components of the research, and the objectives. Chapter 2 provides the review of the relevant literature on seismic hazard, site response, and liquefaction potential evaluations. Chapter 3 describes the models of the seismic sources and probabilistic seismic hazard of the study area. The seismic site characterization of the study area is presented in Chapter 4. Chapter 5 includes the evaluation of seismic site effects for the deep sedimentary
deposits. Chapter 6 presents liquefaction hazard mapping using liquefaction potential index. The conclusions, originality, contributions, limitations and recommendations for future research are included in Chapter 7.
2.1 Background

The literature review of the present study covers relevant literature on seismic hazard analysis, ground motion prediction equations (GMPEs), site response analysis, near-surface shear wave velocity, modulus reduction and material damping curves, and liquefaction hazard analysis.

2.2 Seismic Hazard Analysis

Two recognized approaches are commonly used for seismic hazard analysis. The approaches are deterministic seismic hazard analysis (DSHA) and probabilistic seismic hazard analysis (PSHA). The uses of these approaches depend on the purposes of seismic hazard analysis.

2.2.1 Deterministic Seismic Hazard Analysis (DSHA)

Deterministic seismic hazard analysis (DSHA) was the dominant method in the early years of geotechnical earthquake engineering. In the DSHA method, a particular seismic scenario is developed to estimate ground motion. The scenario is composed of the postulated occurrence of an earthquake of a specific size (magnitude) occurring at specific location (Kramer 1996). Krinitzsky (1995) strongly recommended using the deterministic method for critical engineering projects where consequences of failure are intolerable. Deterministic method can be used for planning of long-term recovery (local), training and planning of emergency response, and planning of post-earthquake recovery where mostly qualitative decisions are required (McGuire 2001).

The deterministic seismic hazard analysis (DSHA) are described using a four-step procedure (Reiter 1990, Kramer 1996):
1. All earthquake sources that are capable of producing significant ground motion at the site are identified and characterized. In this step, the geometry and earthquake potential of each source are defined.

2. The source-to-site distance is measured for each source. In most DSHA, the shortest distance between the source zone and the site of interest is determined. The epicentral or hypocentral distance is used depending on measuring distance of the ground motion prediction equations (GMPEs).

3. The controlling earthquake that is anticipated to generate the strongest shaking at the site is determined. The shaking is generally expressed at the site in terms of some ground motion parameters. The controlling earthquake is defined in terms of its size (magnitude) and the distance from the site.

4. The seismic hazard is expressed in terms of ground motions that is generated at the site by the controlling earthquake. The characteristics of the controlling earthquake are generally defined by one or more ground motion parameters that are estimated using ground motion prediction equations (GMPEs). Peak ground acceleration, peak ground velocity, and spectral response spectrum are frequently used to estimate seismic hazard at the site.

### 2.2.2 Probabilistic Seismic Hazard Analysis (PSHA)

The probabilistic method to evaluate the seismic hazard at the site of an engineering project was originally formulated by Cornell (1968) and Esteva (1969). In this method, the effects of all potential sources of earthquakes, their average activity rates, and their distances from the site are integrated to estimate the ground motion parameters at the site. The annual rate of exceedance of ground motion is expressed in terms of peak ground acceleration, peak ground velocity or spectral
acceleration at the site. The output of probabilistic seismic hazard analysis (PSHA) is a seismic hazard curve (annual rate of exceedance and ground motion amplitude) or a uniform hazard spectrum (spectral amplitude vs structural period, for a fixed annual rate of exceedance) (McGuire 2008). Probabilistic hazard analysis can be used for seismic design of structures, design of retrofit criteria and levels, financial planning for earthquake losses (insurance and reinsurance) and regional plans for long-term recovery (McGuire 2001).

The probabilistic seismic hazard analysis (PSHA) explicitly considers the uncertainties associated with the size, location, rate of occurrence of earthquakes, and the variations in the ground motion characteristics with size and earthquake location in the assessment of seismic hazard (Kramer 1996). The PSHA, originally proposed by Cornell (1968), are described from Baker (2013) as a five-step procedure:

1. All sources capable of producing damaging earthquakes are identified.
2. The distribution of earthquake magnitudes (the expected rate of occurrence of various magnitudes) is predicted.
3. The distribution of the source-to-site distances of potential earthquakes is estimated.
4. The distribution of ground motion as a function of earthquake magnitude, distance, etc. using appropriate ground motion prediction equations (GMPEs) is predicted.
5. The uncertainties in earthquake size, location, and ground motion parameters are combined to predict the ground motion that will be exceeded during a specific time period using a calculation known as the total probability theorem.

The five steps of the PSHA procedure are described in the following sections.
2.2.2.1 Identification of Earthquake Sources

In the PSHA, all earthquake sources that are capable to generate damaging earthquakes are considered. The earthquake sources could be faults, which are identified using various techniques, such as observation of past earthquakes, and geological evidences. If individual faults are not identifiable, the sources are identified as area sources in which earthquakes may occur anywhere. After identification of all sources, the distribution of earthquake magnitudes and the source-to-site distances associated with earthquakes are calculated.

2.2.2.2 Identification of Earthquake Magnitudes

Various magnitudes of earthquakes might occur during activation of source faults. Gutenberg and Richter (1944) first observed that the number of earthquakes in a region greater than a given size (magnitude) generally follows a specific distribution.

\[
\log_{10}\lambda_m = a - bm
\]

Eq. 2.1

where, \( \lambda_m \) is the annual rate of earthquake greater than magnitude \( m \), and \( a \) and \( b \) are the constants.

This relationship is called the *Gutenberg-Richter recurrence law*. The \( a \) and \( b \) values of *Eq. 2.1* are calculated from the statistical analysis of the historical and instrumental earthquake observations with additional information from the geological evidences of earthquake source faults. The constant \( a \) describes the overall rate of earthquakes in a region and the constant \( b \) describes the ratio of small to large magnitude earthquakes. If \( b \)-value increases, the number of larger magnitude earthquakes decreases compared to those of smaller magnitudes. The reciprocal
of the annual rate of exceeding for a given earthquake magnitude is generally called the return period of earthquakes exceeding that magnitude.

Eq. 2.1 can be used to calculate cumulative distribution function (CDF) for earthquake magnitudes greater than a specific minimum value. As low magnitude earthquakes (such as less than 4.0 \( M_w \)) have no engineering importance, low magnitude earthquakes are not included in the analysis. The CDF can be defined as:

\[
F_M(m) = P(M \leq m|M > m_{\text{min}})
\]  \hspace{1cm} \text{Eq. 2.2}

where \( F_M(m) \) denotes the cumulative distribution function (CDF) for M (earthquake magnitude).

The probability density function (PDF) can be calculated for M from the first derivative of the CDF.

\[
f_M(m) = \frac{d}{dm} F_M(m)
\]  \hspace{1cm} \text{Eq. 2.3}

where \( f_M(m) \) denotes the PDF for M.

It is noted that the PDF expressed by Eq. 2.3 is based on Eq. 2.1 of Gutenberg-Richter recurrence law, which theoretically predicts the occurrence of earthquakes with no upper limit. However, physical constrains make it unrealistic, therefore, there is generally upper limit for maximum magnitude of earthquakes in a region due to the limited size of source faults. The limited distribution of earthquake is called as bounded Gutenberg-Richter recurrence law.
2.2.2.3 Identification of Earthquake Distances

It is also necessary to model the distribution of distances from earthquake sources to the site of interest to predict the ground motion at the site. It is generally expected that earthquake will occur with equal probability at any location of the fault for a given earthquake source. It is considered that the locations are uniformly distributed and the distribution of the source-to-site distances are determined using the source geometry. The distance are measured in various ways depending on the ground motion prediction equation. It can be the distance to the epicenter or hypocenter, distance to the closest point on the rupture surface, distance to the closest point on the surface projection of the rupture. Area source is generally used to account for background seismicity or for earthquakes that are not related to specific fault.

The probability distribution function (PDF) of distances is defined as:

\[ F_R(r) = P(R \leq r) \]

Eq. 2.4

The probability density function (PDF) can be calculated by taking the derivative of cumulative distribution function (CDF) of Eq. 2.4:

\[ f_R(r) = \frac{d}{dr} F_R(r) \]

Eq. 2.5

2.2.2.4 Ground Motion Intensity Prediction

The ground motion prediction equations (GMPEs) or attenuation relationships are used to predict the ground motion intensity distribution and its associated uncertainty at a site as function of many predictor variables, such as earthquake magnitude, distance, faulting mechanism, and near-surface site condition. Ground motion prediction equations (GMPEs) are developed by
The seismic ground motion is influenced by earthquake magnitude, distance, characteristics of source, and effects of the source-to-site path. The prediction model provides a probability distribution of intensities. To describe the probability distribution, the prediction models are expressed as the following general form (Baker 2013):

\[ \ln IM = \ln IM(M, R, \theta) + \sigma(M, R, \theta). \varepsilon \]  
\[ Eq. 2.6 \]

where, \( \ln IM \) denotes the natural log of ground motion intensity measure of interest (for example, spectral acceleration at a given period); \( \ln IM(M, R, \theta) \) and \( \sigma(M, R, \theta) \) are the predicted mean and standard deviation, respectively of \( \ln IM \). The both terms are functions of earthquake’s magnitude (M), distance (D) and other parameters (generally denote as \( \theta \)). The \( \varepsilon \) is a standard normal random variable that represents the observed variability in \( \ln IM \). The positive values of \( \varepsilon \) produce larger than average value of \( \ln IM \) and the negative values of \( \varepsilon \) produce smaller than average value of \( \ln IM \).

2.2.2.5 Combined All Information

The probability of exceeding a given ground motion intensity can be calculated using ground motion prediction equations (GMPEs) for a given magnitude and distance. However, the magnitude and distance of future earthquake are not yet known. The probability distributions of
magnitudes and distances of earthquakes can be predicted from Step 2 (identification of magnitude) and Step 3 (identification of distance) of the PSHA. Then, probability of exceeding a given ground motion intensity can be predicted for all possible magnitudes and distances combining all information using total probability theorem.

$$P(IM > x) = \int_{m_{\text{min}}}^{m_{\text{max}}} \int_{0}^{r_{\text{max}}} P(IM > x|m, r) f_M(m) f_R(r) \, dr \, dm$$  \hspace{1cm} \textit{Eq. 2.7}$$

where $P(IM > x|m, r)$ is from ground motion prediction model, $f_M(m)$ and $f_R(r)$ are PDFs of magnitude and distance and integration is performed over all magnitudes and distances.

\textit{Eq. 2.7} provides the probability of exceeding a given ground motion intensity for given earthquakes, but it does not provide information on how often earthquake will occur on the source of interest. A simple modification of the equation can be made to calculate the rate of exceeding a given ground motion intensity rather than probability of exceeding a given ground motion intensity for the occurrence of an earthquake.

$$\lambda(IM > x) = \lambda(M > m_{\text{min}}) \int_{m_{\text{min}}}^{m_{\text{max}}} \int_{0}^{r_{\text{max}}} P(IM > x|m, r) f_M(m) f_R(r) \, dr \, dm$$  \hspace{1cm} \textit{Eq. 2.8}$$

where $\lambda(M > m_{\text{min}})$ denotes the rate of occurrence of a given earthquake greater than magnitude, $m_{\text{min}}$ from the source and $\lambda(IM > x)$ denotes the rate of exceeding a given ground motion intensity, $x$.

The generalized equation for the rate of exceeding a given ground motion intensity for all sources is simply the sum of the rate of occurrence of individual source (Baker 2013):
\[ \lambda(IM > x) = \sum_{i=1}^{n_{\text{sources}}} \lambda(M_i > m_{\text{min}}) \sum_{j=1}^{n_M} \sum_{k=1}^{n_R} P(IM > x|m_j, r_k)P(M_i = m_j)P(R_i = r_k) \]

where \( n_{\text{sources}} \) is the number of sources considered; \( f_{M_i} \) and \( f_{R_i} \) are the magnitude and distance distributions for source, \( i \).

As the analysis is always performed using computer, it is practical to discretize the continuous distributions for \( M \) and \( R \) and transform the integrals to discrete summations, as follows (Baker 2013):

\[ \lambda(IM > x) = \sum_{i=1}^{n_{\text{sources}}} \lambda(M_i > m_{\text{min}}) \sum_{j=1}^{n_M} \sum_{k=1}^{n_R} P(IM > x|m_j, r_k)P(M_i = m_j)P(R_i = r_k) \]

Eq. 2.9 or Eq. 2.10 is commonly used in the probabilistic seismic hazard analysis (PSHA).

2.2.3 Deterministic Versus Probabilistic Seismic Hazard Analysis

There is a long debate on deterministic versus probabilistic seismic hazard analysis approaches (Reiter 1990, Krinitzsky 1995, Orozova and Suhadolc 1999, McGuire 2001, Klügel 2008, Wang and Cobb 2013). The frequent criticism of the deterministic hazard analysis is that the method uses only the controlling earthquake (generally maximum magnitude within a given time and space) rather than frequencies of earthquake occurrences. Orozova and Suhadolc (1999) proposed a deterministic-probabilistic method to incorporate the frequency of earthquake occurrence instead of a fixed controlling earthquake to overcome the problem.

The use of deterministic and probabilistic analyses for seismic hazard and risk assessment have differences, advantages, disadvantages that often make the use of one advantageous over the other.
Probabilistic approach can be observed as an inclusive method that accounts for all deterministic events with a finite probability of occurrence. In this situation, deterministic approach emphasizes on a single earthquake which is a realistic event having a finite probability of occurrence. This is the complementary nature of deterministic and probabilistic analyses. Deterministic events can be tested with the probabilistic analysis to confirm that the event is realistic. The probabilistic analysis can also be tested using deterministic analysis to observe that the balanced and realistic hypothesis of concern have been included in the analysis. Therefore, emphasis is given to one over the other depending on the purposes of the hazard and risk assessment (McGuire 2001).

The results of any seismic hazard analysis are used for decision-making. The decision can be for selection of design or retrofit criteria and levels, financial planning for earthquake losses (levels of insurance or reinsurance, or self-insurance), investment for redundant industrial systems, planning for emergency response, post-earthquake recovery, long-term recovery, etc. The decision can be made using both deterministic and probabilistic approaches knowing the type of decision to be made (McGuire 2001).

Krinitzsky (2003, 1995) argued that the DSHA is more reliable, logical, and transparent than the PSHA. The DSHA is based on geological knowledge and observation, whereas PSHA is based on earthquake statistic and theory-guided numerical calculation. To design critical structures, one must not use the PSHA. Only the DSHA is the suitable method for that purpose.

Klügel (2008) reviewed the strength and weakness of the seismic hazard analysis methods that are currently been used. The review of the methods is performed in the perspective of various applications, such as design of critical and general infrastructures, technical and financial risk analysis. The traditional probabilistic hazard analysis methods have some deficiencies, which limit
their applications to design critical infrastructures without validation. On the other hand, traditional
deterministic or scenario-earthquake based seismic hazard analysis methods give reliable and
robust design basis for applications, such as design of critical infrastructures. Deterministic seismic
hazard analysis accounts the uncertainties in safely factor. Scenario-based or deterministic seismic
hazard analysis with high safety factor may be too conservative for civil structures of short lifetime
(e.g., 50 or 100 years). Scenario based seismic hazard analysis has clear physical basis. Deterministic methods are related to seismic sources that are identified by geomorphological,
geological, seismological, geodetic investigations or taken from historical records. Scenario-based
seismic hazard analysis can be extended for risk analysis applications incorporating the frequency
of earthquake occurrence. Such methods provide better risk models that are suitable for risk-
informed decision-making.

Radu and Grigoriu (2014) developed a site specific seismological model for probabilistic
seismic hazard assessment using the specific barrier model (SBM) which is a seismic source model
constructed using physical approaches and calibrated to a large sets of regional ground motion
records consistent to certain tectonic characteristics. This model can be used to simulate any
number of ground motion samples and to carry out the probabilistic seismic hazard assessment
(PSHA).

2.3 Ground Motion Prediction Equations (GMPEs)

The seismic ground motion at a site is measured as a function of magnitude and distance from
the source by using ground motion prediction equations (GMPEs). The GMPEs are empirically
derived from a large number of earthquake ground motion data by using statistical regression
analysis (Abrahamson and Silva 2008, Boore and Atkinson 2008, Campbell and Bozorgnia 2008,
Chiou and Youngs 2008, Idriss and Eeri 2008). Five sets of GMPEs are developed in 2008 for shallow crustal earthquakes using five global data sets of ground motions by five research groups through the Pacific Earthquake Engineering Research Center’s (PEER) ‘Next Generation of Ground Motion Attenuation Models’ for the western United States (NGA West) program. These GMPEs were updated in 2014 by those research groups using more global and recent ground motion data through the NGA West-2 program (Abrahamson et al. 2014, Boore et al. 2014, Campbell and Bozorgnia 2014, Chiou and Youngs 2014, Idriss 2014). Atkinson and Boore (2003) and Abrahamson et al. (2016) developed GMPEs using global ground motion data set for the subduction interface and intraslab earthquakes. Zhao et al. (2006) also developed GMPEs using mostly Japanese data set for subduction interface and intraslab earthquakes.

2.4 Site Response Analysis

The site response analysis is used to predict the ground motion at the ground surface for development of design response spectra, to evaluate the dynamic stresses and strains for liquefaction hazard assessment, and to estimate the earthquake-induced forces that can lead to instability of earth and earth-retaining structures (Kramer 1996).

In deterministic or probabilistic approaches, the ground motion parameters of a site are generally estimated at the top of bedrock ($V_{s30} \geq 760$ m/s) by using ground motion prediction equation (GMPE) as a function of earthquake magnitude and distance from the source (Cramer 2003). In traditional probabilistic seismic hazard analysis, seismic hazards are calculated using bedrock conditions and the results of rock hazards are modified using deterministic site-specific amplification factors (McGuire and Toro 2008). A common practice in the last two decades for site-specific seismic ground motion is simply multiplying the rock probabilistic ground motion by
a median value of amplification factor of a site. If the site-specific amplification factor is a single value, it means that there is no uncertainty in calculating amplification factor. However, there are several uncertainties in estimating amplification factor. Therefore, multiplying the rock probabilistic ground motion by a single value amplification factor is not truly probabilistic (Cramer 2003).

The effect of uncertainties on the estimation of amplification factor need to be measured and the resulting site-specific amplification distribution need to be used in a probabilistic methodology to overcome this problem. Cramer (2003) developed a probabilistic method to modify the rock ground motion attenuation relations into the site-specific relations by using site-amplification distribution function. A completely probabilistic approach for site-specific ground motion make a difference of 10% compared to the ground motion that is calculated simply multiplying the rock probabilistic ground motion by a median value of amplification factor of a site at 1 in 2475 annual probability of exceedance (Cramer 2003). Therefore, full site-amplification distribution should be used for site-specific seismic hazard instead of a single deterministic median value.

McGuire (2001) also suggested for using site-specific amplification distribution to modify the rock ground motion attenuation relations into site-specific relations prior to seismic hazard calculation. Accurate site hazard can be estimated by the convolution of rock seismic hazard with non-linear site response. Several uncertainties are important in calculating site-specific amplification factors for accurate site hazard estimation. Bazzurro and Cornell (2004a, 2004b) in their study incorporated the amplification of local soil deposit to the framework of probabilistic seismic hazard assessment (PSHA). In this method, the hazard at the ground surface or any desire depth is estimated by combining the site-specific seismic hazard at the bedrock with the probability
distribution of the amplification function. The approach provides more accurate surface ground motion hazard calculation than those estimated by using standard attenuation laws of generic soil conditions.

Among the methods of estimating site response, the assumption of vertically propagating shear waves into a horizontally layered medium is most popular, because of its simplicity and it is applicable in most of the sites for critical facilities. Other models, such as 2-D and 3-D wave propagation models, and models of basin effect may be appropriate for certain situations (McGuire and Toro 2008).

In site response analysis, site characterization is required to collect the dynamic properties of near-surface soils, such as shear wave velocity, modulus reduction factor, damping ratio to estimate site coefficient (amplification factor).

### 2.5 Site Characterization

A precise definition of site class and accurate estimation of site-specific amplification factors in terms of the average shear wave velocity of the near-surface materials was documented by Borcherdt (1994). The average shear wave velocity of the near-surface materials up to a depth of 30 m ($V_{s30}$) is used as an index to estimate amplification factor (Borcherdt 1994, BSSC 1994, 1998, Martin and Dobry 1994, Anderson et al. 1996, UBC 1997, Park and Elrick 1998, Xia et al. 1999, Dobry et al. 2000). The methods that are commonly used to collect the $V_s$ data are described in the following sections.
2.5.1 Shear Wave Velocity Measurement


Near-surface shear velocity ($V_s$) can be accurately estimated using downhole seismic (DS) and crosshole seismic methods (Boore and Brown 1998). However, these methods are more expensive than surface wave methods. Therefore, surface wave method, such as multichannel analysis of surface waves (MASW) is being increasingly used to estimate the near-surface shear wave velocity ($V_s$) for earthquake engineering site characterization. The $V_s$ of the near-surface materials is estimated from the dispersion of surface waves, such as Rayleigh waves that are recorded by the MASW survey (Anderson et al. 1996, Boore and Brown 1998, Park et al. 1999, Xia et al. 1999, Louie 2001, Tian et al. 2003, Xu et al. 2006, Xia 2014). Accurate mode separation and identification of surface wave are the important components to create the dispersion curve (Crampin and Bath 1965, Park et al. 2005). The $V_s$ of the near-surface materials can be determined precisely from the dispersion of the surface wave, such as Rayleigh waves (Park et al. 1999, Xia et al. 1999).
Tian et al. (2003) indicated that the MASW can be used efficiently to accurately estimate the $V_s$ of the near-surface materials to reduce time and cost incurred for data acquisition. Hayashi and Suzuki (2004) have proposed a method for the analysis of the MASW data using common mid-point (CMP) cross-correlation gathers to generate accurate phase velocity curves and to reconstruct two-dimensional (2D) velocity structures with higher resolution. Xu et al. (2006) have proposed a formula to determine the minimum distance between the source and first receiver (geophone) to carry out the MASW using a source, such as a sledgehammer. The minimum offset is an important parameter in a MASW survey to achieve the proper resolution of the dispersion image of the high frequency surface wave for accurate estimation of the $V_s$. The results of the MASW revealed that the formula derived to determine the minimum offset, is accurate for the near surface $V_s$ estimation. Xia (2014) also estimated the shear wave velocity ($V_s$) of the near surface materials from the dispersions of high frequency Rayleigh waves and Love waves. It was observed that the multichannel analysis of Love waves (MALW) has some fascinating advantages over the MASW. The dispersion curves of the Love waves are simpler having higher signal and noise ratio, less dependent on the initial model, and more stable than that of the Rayleigh waves.

Park et al. (2005) observed that the modal nature of the active and passive curves should be assessed with extended frequency ranges to an increased depth of investigation. The active MASW with frequencies greater than 30 Hz and the passive MASW with frequencies less than 30 Hz were used in their analysis. The combined sets of images from the active and passive surveys can be a highly effective approach for understanding the overall modal nature in extended frequencies and phase velocity ranges. Hayashi et al. (2005) have proposed a passive surface wave method to estimate the $V_s$ up to a depth of 100 m using triangular and L-shaped arrays of receivers to record the ambient vibrations of the earth (microtremor). In passive surface wave method, a two-
dimensional receiver array is necessary for accurate results. However, in an urban area with densely populated buildings, it is difficult to find a spacious area to carry out such a two-dimensional survey. Park and Miller (2008) proposed a passive MASW method along a roadside to overcome the problem of the two-dimensional receiver array in densely populated urban areas.


### 2.6 Modulus Reduction Factor and Damping Ratio

The modulus reduction factor and damping ratio are used in equivalent-linear and nonlinear site response analyses to estimate the nonlinear dynamic properties of soils. Several modulus reduction curves and material damping curves are developed by several researchers (Seed and Idriss 1970, Darendeli 2001, Andrus et al. 2003, Roblee and Chiou 2004) to estimate the nonlinear properties of soils. The modulus reduction and material damping curves are prepared by...
performing different geotechnical tests using the samples of the site of interest. If the appropriate samples from the site and the testing equipment are not available to construct the modulus reduction and material damping curves, the reference curves are used to fit the curves of the study area. The initial effective vertical stress, initial coefficient of effective lateral earth pressure, plasticity index, over consolidation ratio, loading frequency, number of loading cycles, etc. of the site are required to fit modulus reduction and material damping curves of the different layers of the soil profile with the reference curves.

### 2.7 Liquefaction Hazard Analysis


#### 2.7.1 Simplified Procedure

The simplified procedure, originally proposed by Seed and Idriss (1971), is the most widely used method for evaluating liquefaction resistance of soils. Then, the method has been modified
and improved by Seed and Idriss (1982), Seed et al. (1985), and later adjusted, modified and evaluated by Idriss and Boulanger (2004), Seed et al. (2001), and Youd et al. (2001).

Seed et al. (1983) described two methods to evaluate the cyclic liquefaction potential for soils. Method-1 is based on the observations of field performance of soils in previous earthquakes and use of in situ characteristics of soils to determine the possible similarities and dissimilarities between these sites and proposed new sites with their liquefaction potentials. Method-1 is based on the evaluation of the cyclic stress or strain developed in the field due to a design earthquake and a comparison of these stresses and strains with those observed to cause liquefaction of representative samples in some laboratory tests that provide an appropriate simulation of the field conditions. These methods are generally considered quite different approaches, as the first method is based on an empirical relationship between some in situ characteristics and observed performance, whereas the second method is based on stress-strain analysis using laboratory testing. In fact, two methods use same basic approach and differ in the techniques by which the liquefaction characteristics are determined in the field.

In general, most methods available for the evaluation of liquefaction resistance of soils have two parts (Robertson and Campanella 1985): 1) evaluation of the cyclic stress or strain developed in the field due to a design earthquake; and 2) evaluation of field liquefaction characteristics.

### 2.7.1.1 Evaluation of Cyclic Stress Ratio (CSR)

Under level ground condition, the cyclic characteristic of soil is best represented by cyclic stress ratio (CSR), i.e., the ratio of the average cyclic shear stress ($\tau_{av}$) developed on horizontal surface of soils due to cyclic or earthquake loading to the initial vertical effective stress ($\sigma_0'$) acting on the soil layer before the cyclic stresses are applied. This parameter has the advantage of taking
account the depth of the soil layer involved, the depth of groundwater level and the intensity of
earthquake shaking or other cyclic loading phenomena (Seed et al. 1983).

The Seed-Idriss (1971) simplified procedure is used to estimate the CSR developed in the field
due to an earthquake loading, at a depth of \( z \) from the ground surface, using the following equation:

\[
CSR = \frac{\tau_{av}}{\sigma'_0} = 0.65 \times \frac{a_{max}}{g} \times \frac{\sigma_0}{\sigma'_0} \times r_d
\]

Eq. 2.11

where \( a_{max} \) = maximum horizontal acceleration at the ground surface generated by the
earthquake; \( g \) = acceleration of gravity; \( \sigma_0 \) and \( \sigma'_0 \) are total and effective vertical overburden
stresses, respectively at depth of \( z \); and \( r_d \) = stress reduction coefficient that accounts for the
flexibility of the soil column (i.e., \( r_d = 1 \) corresponds to the rigid body behaviour). The factor
0.65 is used to convert the peak cyclic shear stress ratio to a cyclic stress ratio that is the
representative of most significant cycles over the full duration of loading (Idriss and Boulanger
2004).

2.7.1.2 Evaluation of Cyclic Resistance Ratio (CRR)

A plausible method to evaluate the CRR is to retrieve and test undisturbed specimens in
laboratory. However, in situ stress state cannot be re-established in the laboratory and the
specimens retrieved using traditional drilling and sampling techniques are too disturbed to achieve
meaningful results. Sufficiently undisturbed specimens can be retrieved by using some specialized
sampling techniques, such as ground freezing. The costs of such procedures are sometimes beyond
the budget and scope of the most engineering projects. To avoid difficulties associated with
sampling and laboratory testing, the in situ index tests have become the state-of-practice for
evaluation of liquefaction resistance. The state-of-the-art paper by Youd et al. (2001) summarized
and recommended four in situ test methods for liquefaction potential assessment: 1) standard penetration test (SPT); 2) cone penetration test (CPT); 3) in situ shear wave velocity measurement\( (V_s) \); and 4) Becker penetration test (BPT).

The cyclic resistance ratio for \( M_w = 7.5 \) earthquake (CRR\(_{7.5} \)) are determined from overburden stress corrected standard penetration test (SPT) resistance of equivalent clean sand, \( (N_1)_{60cs} \) (Seed et al. 2001, Youd et al. 2001, Idriss and Boulanger 2004). The CRR can also be evaluated from overburden stress corrected cone penetration test (CPT) resistance of equivalent clean sand, \( (q_{c1N})_{cs} \) (Robertson and Wride 1998), overburden stress corrected shear wave velocity of equivalent clean sand, \( V_{s1} \) (Andrus and Stokoe 1997, 2000), and overburden stress corrected Becker penetration test (BPT) resistance, \( N_{BC} \) (Harder 1997).


Robertson and Campanella (1985) have developed a procedure to estimate the CRR based on cone penetration test (CPT) due to intrinsic difficulties and poor repeatability of the SPT results. The procedure has been revised and updated by Olsen (1997, 1988), Olsen and Koester (1995), Robertson and Wride (1998), Seed and de Alba (1986), Stark and Olson (1995). The main advantage of the CPT is that a nearly continuous penetration resistance profile are generated for stratigraphic interpretation. The results of the CPT are more consistence and repeatable than the results of other penetration tests, such as the SPT (Youd et al. 2001).

Andrus and Stokoe (2000, 1997) have proposed a procedure to estimate the CRR from the field measurement of shear wave velocity. The advantages of using shear wave velocity are that: 1) the
measurement are possible in hard soils where SPT and CPT are difficult to penetrate or to collect undisturbed samples, such as gravelly soils or at the site where SPT and CPT may not be permitted; 2) shear wave velocity is a basic mechanical properties of soil materials, directly related to the small-strain shear modulus; 3) the small-scale shear modulus is a parameter required in analytical procedures for estimating dynamic soil response and soil-structure interaction analyses (Youd et al. 2001).

The SPT and CPT can be used to estimate the liquefaction resistance of non-gravelly soils. However, the penetration resistance measurements by SPT and CPT are not generally reliable in gravelly soils. Large gravels may interfere with the normal deformation of the soils around the penetrometer and misleadingly increase the penetration resistance. Becker penetration test (BPT) was recommended by Youd et al. (2001) to estimate the penetration resistance in gravelly soils.

The criteria for liquefaction resistance (i.e., cyclic resistance ratio, CRR) evaluation using standard penetration test (SPT) blow counts have been rather robust over the years (Youd et al. 2001).

2.7.1.3 Determination of Factor of Safety

In simplified procedure, Seed and Idriss (1971) compared the cyclic stress ratio (CSR) to the liquefaction resistance of the soils represented by the cyclic resistance ratio (CRR) for $M_w = 7.5$ earthquake (i.e., $CRR_{7.5}$). A magnitude scaling factor (MSF) is used to adjust $CRR_{7.5}$ to determine CRR for other earthquake magnitudes. The factor of safety ($F_L$) against liquefaction is defined in terms of CRR, CSR and MSF as follows.

$$F_L = \frac{CRR_{7.5}}{CSR} \times MSF$$  \hspace{1cm} Eq. 2.12
2.7.1.4 Magnitude Scaling Factor (MSF)

Magnitude scaling factor (MSF) of \(Eq. \, 2.12\) is used to adjust the CRR value for earthquake magnitude smaller or larger than 7.5. Various magnitude scaling factors are suggested by several researchers (Seed and Idriss 1982, Ambraseys 1988, Arango 1996, Andrus and Stokoe 1997, Youd and Noble 1997). The NCEER 1998 (Youd et al. 2001) recommended MSF, which is proposed by Idriss in 1995 Seed Memorial Lecture, is shown in \(Eq. \, 2.13\). This MSF is revised and defined using the data of the original MSF that is proposed by Seed and Idriss (1982).

\[
MSF = 10^{2.24}/M_w^{2.56}
\]

\(Eq. \, 2.13\)

2.7.1.5 Seismic Factors

Two seismic ground motion parameters are required to evaluate the liquefaction resistance of soils using simplified procedure. The parameters are earthquake magnitude and peak ground acceleration.

2.7.2 Liquefaction Potential Index (LPI)

The factor of safety \((F_L)\) is not a sufficient parameter for evaluation of liquefaction and its damage potential at any site. However, the thickness and depth of the liquefiable layer and the factor of safety are very important inputs for damage potential based on liquefaction. Since it was proposed by Iwasaki et al. (1978), the liquefaction potential index (LPI) has been a very popular tool due to the inclusion of the thickness and depth of the liquefiable layer and the factor of safety as inputs. The LPI was originally proposed by Iwasaki et al. (1982, 1978) to evaluate the potential for liquefaction to cause foundation damage. The LPI assumes that the severity of liquefaction is
proportional to (1) the thickness of the liquefied layer; (2) proximity of the liquefied layer from the ground surface; and (3) amount by which the factor of safety ($F_L$) is less than 1.0.

The LPI is defined as:

$$L_I = \int_0^{20} F(z) W(z) dz \quad \text{Eq. 2.14}$$

where:

$$F(z) = 1 - F_L \quad \text{for } F_L < 1.0 \quad \text{Eq. 2.14 (a)}$$

$$F(z) = 0 \quad \text{for } F_L \geq 1.0 \quad \text{Eq. 2.14 (b)}$$

$$W(z) = 10 - 0.5z \quad \text{for } z < 20 \text{m} \quad \text{Eq. 2.14 (c)}$$

$$W(z) = 0 \quad \text{for } z > 20 \text{m} \quad \text{Eq. 2.14 (d)}$$

where, $z$ is the depth from the ground surface in meters.

In the present study, the original form of the LPI proposed by Iwasaki et al. (1982, 1978) was followed to produce the liquefaction hazard map by considering some validation of the threshold values of LPI in literature.

Iwasaki et al. (1982) identified LPI values of 5 and 15 as the lower bounds of "moderate" and "major" liquefaction, respectively, from SPT measurements at 85 Japanese sites subjected to six earthquakes. Toprak and Holzer (2003) also found similar results using 50 CPT sounding at 20 sites affected by the 1989 Loma Prieta ($M_w = 6.9$) earthquake to correlate with surface manifestation of liquefaction. They ascertained that median values of LPI equal to 5 and 12 corresponded to the occurrence of sand boils and lateral spreading, respectively. The San Simson earthquake also support the use of LPI = 5 as the threshold for surface manifestations of
liquefaction (Holzer et al. 2005b). The LPI has the capability of the use for the spatial analysis of the liquefaction hazards because it allows to develop a two-dimesional representation of a three-dimensional phenomenon (i. e., $F_L$ vs. depth), which is ideal for mapping (Luna and Frost 1998), and it correlates well with liquefaction effects (Toprak and Holzer 2003).

### 2.8 Summary

Most of the earthquake sources in the present study area have not been identified and characterized yet. There are uncertainties in predicting earthquake size, location, rate of occurrence, and variation in ground motion intensity. In probabilistic seismic hazard analysis (PSHA), these uncertainties are explicitly considered to predict ground motion. The PSHA is widely used for seismic hazard mapping. Therefore, in the present study, the PSHA has been used to estimate ground motion parameters and to prepare seismic hazard map.

The ground motion prediction equations (GMPEs) that were developed in 2014 by the PEER NGA West-2 program were used for shallow crustal seismic sources in predicting ground motion intensity using probabilistic seismic hazard analysis (Abrahamson et al. 2014, Boore et al. 2014, Campbell and Bozorgnia 2014, Chiou and Youngs 2014, Idriss 2014). For deep seismic sources (subduction sources), the GMPEs that were developed by Atkinson and Boore (2003), Abrahamson et al. (2016), and Zhao et al. (2006) were used in the present study.

The time-averaged shear wave velocity in the top 30 m ($V_{s30}$) is generally used for site characterization to estimate site amplification factor. Shear wave velocity ($V_s$) estimation by geophysical methods needs expertise knowledge, sophisticated instruments and software for data acquisition and analysis. These facilities are not sufficiently available in developing countries. Therefore, it is necessary to develop the technique for the $V_s$ estimation using the relationship
between the $V_s$ and readily available in-situ test data, such as standard penetration test blow counts (SPT-N) that is a popular method in geotechnical earthquake engineering. In the present study, the $V_{s30}$ was estimated using the relationship between the $V_s$ and SPT-N, and the relationship between the $V_{s30}$ and the Holocene soil thickness. These relationships were developed using the $V_s$, SPT-N, and Holocene soil thickness of the study area.

As a simplified procedure, the surface ground motion are predicted by multiplying the bedrock ground motion with the site amplification factor (site coefficient) that is estimated using the time-averaged shear wave velocity in the top 30 m ($V_{s30}$). For deep sedimentary deposits where the bedrock ($V_s = 760$ m/s) exists more than 30 m depth, the prediction of surface ground motion using $V_{s30}$-based site amplification factor is not appropriate. Therefore, one-dimensional site response analysis is essential to predict surface ground motion. In the present study, one-dimensional linear, equivalent-linear, and non-linear site response analyses were performed to predict surface ground motion. The dynamic properties of the soils were modeled by using the normalized modulus reduction and material damping curves that were proposed by Darendeli (2001).

The liquefaction hazard was evaluated using simplified procedure form the standard penetration test blow count (SPT-N) data of the study area. The liquefaction potential index (LPI) was calculated using the safety factors of the liquefaction potential that were estimated using simplified procedure. The liquefaction hazard map was prepared using the LPI of different sites in the study area.
Chapter 3: Seismic Source Models and Probabilistic Seismic Hazard Analysis

3.1 Background

The probabilistic seismic hazard analysis (PSHA) method was originally formulated by Cornell (1968) and Esteva (1969) to evaluate the seismic hazard at the sites of engineering projects. In this method, the effects of all potential sources of earthquakes, their average activity rates, and their distances from a site are integrated to estimate the ground motion parameters at the site. The PSHA explicitly considers the uncertainties associated with the size, location, rate of occurrence of earthquakes, and the variations in the ground motion characteristics with size and earthquake location in the assessment of seismic hazard (Kramer 1996).

Bangladesh and the surrounding regions are seismically vulnerable due to their location in a seismically active zone on the earth (Morino et al. 2011, 2014, Steckler et al. 2016). Bilham (2009) mentioned that an epicentral hit of a large earthquake on a megacity of developing country has the potential to cause one million fatalities. The developing countries, which are located close to the seismically active zones on the earth, are more vulnerable compared to the developed countries due to their high population density, unplanned urbanization, non-engineered construction practice, inadequate knowledge on the seismic design of structures, ignorance of existing building codes, and poor monitoring system of the concerned authorities during construction of structures (Rahman and Siddiqua 2016).

Although Bangladesh is located close to one of the highest seismic risk zones in the world, a proper scenario of earthquake risk and related strategies, policies, and action plans regarding
provisions, response, and recovery has not been developed yet (Rahman et al. 2017a). The destructive and deadly hazard of an earthquake may create a real and serious threat to life, property damage, economic growth, and development of a country. Therefore, a proper understanding of the distribution and level of seismic hazard is essential for future planning, development, and safety of the country (Rahman et al. 2018a).

3.2 Seismotectonics

Bangladesh is situated close to the convergence plate boundary between the Indian and Eurasian plates (Figure 3.1). Large subduction earthquakes ($M_w > 8.0$) normally occur along plate boundary megathrusts beneath the accretionary prisms (Ujiie and Kimura 2014, Steckler et al. 2016). Steckler et al. (2016) suggested that the presence of the locked plate boundary megathrust beneath the accretionary prism covering the Indo-Burman Ranges to the Ganges-Brahmaputra Delta between the Indian and Eurasian plates represents an underappreciated seismic hazard in one of the most densely populated regions in the world. They estimated that a potential earthquake of $M_w 8.2 – 9.0$ might occur along the Arakan megathrust.

Bilham (2004) also identified the seismic gaps along two-thirds of the Himalayan Ranges that have developed during the last five centuries. The accumulated elastic strain energy, which was estimated by combining geodetic convergence rate of 1.8 cm/year between Eurasian Plate and Indian Plate, might generate one or more $M_w = 8$ earthquakes along these seismic gaps.

The northward collision of the Indian Plate with the Eurasian Plate has created the Himalayan Ranges between the Indian Plate and Eurasian Plate, and the Bengal Basin in the eastern part of the Indian Plate (Curray et al. 1982, Aitchison et al. 2007). The Bengal Basin covers the northeastern part of the Indian Plate, which includes Bangladesh and parts of West Bengal,
Tripura, and Assam of Indian states. The Bengal Basin is one of the largest sedimentary basins in the world. The maximum thickness of sedimentary deposits (sandstone, shale, siltstone, limestone) in the basin is more than 22 km (Alam et al. 2003). Bangladesh covers about three-fourths of the Bengal Basin, which is bounded in the west by the Indian Platform, in the north by the Precambrian Shilling Massif, in the east by the Indo-Burman Ranges, and in the south it is open to the Bay of Bengal (Reimann 1993).

Figure 3.1 Historical and recent earthquakes (magnitude ≥ 6.5) from 1762 to 2016 (Rahman and Siddiqua 2017b). Reprinted with permission of Springer Nature.
Two major active tectonic belts are responsible for the large and damaging earthquakes in Bangladesh, northern and northeastern parts of India, Nepal, Bhutan, and Myanmar. The tectonic elements are the Himalayan system in the north and the Arakan subduction-collision system in the east (Figure 3.1). The Himalayan Frontal Thrust (HFT) and the Dauki Fault (DF) are the main components of the Himalayan system. The Indo-Burman Ranges and underlying megathrust are the manifestation of the Arakan subduction-collision system (Steckler et al. 2008).

Several historical earthquakes occurred in the northeastern parts of India, Nepal, Myanmar, and Bangladesh along these tectonic belts in the last 256 years (Figure 3.1). Among them, the 1762 Bengal-Arakan, 1885 Bengal, 1897 Great Assam, 1918 Srimangal earthquakes (Figure 3.1 and Table 3.1) caused severe damage in northern, northeastern, southeastern and central parts of Bangladesh (Middlemiss 1885; Oldham 1899; Ambraseys and Douglas 2004; Martin and Szeliga 2010; Szeliga et al. 2010; Stuart 1920).

Several earthquakes having magnitudes from 4.0 to 6.0 occurred in Bangladesh in recent years. The earthquakes (magnitude ≥ 4) that occurred from 1762 to 2016 in these regions are shown in Figure 3.2.

3.3 Seismic Source Models

The seismic sources were modeled based on the existing published literature on active faults, geodesy, geodynamics, and seismotectonics of the study regions and available earthquake catalogs of these regions from different international organizations. The seismic sources of the study regions are: 1) background seismicity, 2) regional seismicity, 3) shallow crustal fault, and 4) subduction zone.
**Table 3.1 Damage and casualty of Major earthquakes in Bangladesh during the last 256 years**

<table>
<thead>
<tr>
<th>Date</th>
<th>Name of Earthquake</th>
<th>Magnitude (M&lt;sub&gt;W&lt;/sub&gt;)</th>
<th>Number of Death</th>
<th>Structural Damage</th>
</tr>
</thead>
<tbody>
<tr>
<td>April 2, 1762</td>
<td>Bengal-Arakan Earthquake</td>
<td>8.5&lt;sup&gt;(1)&lt;/sup&gt;</td>
<td>500 in Dhaka&lt;sup&gt;(2)&lt;/sup&gt;</td>
<td>The earthquake was very violent in Dhaka and Chittagong.&lt;sup&gt;(2)&lt;/sup&gt;</td>
</tr>
<tr>
<td>July 14, 1885</td>
<td>Bengal Earthquake</td>
<td>6.87&lt;sup&gt;(3)&lt;/sup&gt;</td>
<td>Not reported</td>
<td>The highest damage was observed in Siraiganj, Bogra, Jamalpur and Mymensigh. In Dhaka, the damage was very low compared with other areas located at same distance from the center of disturbance.&lt;sup&gt;(4)&lt;/sup&gt;</td>
</tr>
<tr>
<td>June 12, 1897</td>
<td>Great Assam Earthquake</td>
<td>8.03&lt;sup&gt;(3)&lt;/sup&gt;</td>
<td>545 in Sylhet&lt;sup&gt;(2)&lt;/sup&gt;</td>
<td>The highest damage was reported in Shillong, Assam (India). In Dhaka, almost all masonry buildings were badly damaged and some were entirely collapsed. In Sylhet, most of the masonry buildings were severely damaged.&lt;sup&gt;(5)&lt;/sup&gt;</td>
</tr>
<tr>
<td>July 08, 1918</td>
<td>Srimangal Earthquake</td>
<td>7.10&lt;sup&gt;(3)&lt;/sup&gt;</td>
<td>Exact numbers were not reported</td>
<td>Almost all tea factories and bungalows in Srimangal (Moulvibazar) were reported to be destroyed. Considerable damage was observed in Kishoreganj, Sylhet, Habiganj, Agartala (India). In Dhaka, several buildings were slightly cracked.&lt;sup&gt;(6)&lt;/sup&gt;</td>
</tr>
</tbody>
</table>

According to <sup>(1)</sup>Wang et al. (2014); <sup>(2)</sup>Banglapedia (accessed on 06 August, 2018); <sup>(3)</sup>Ambraseys and Douglas (2004), <sup>(4)</sup>Middlemiss (1885); <sup>(5)</sup>Oldham (1899); and <sup>(6)</sup>Stuart (1920).

### 3.3.1 Background Seismicity Model

A homogeneous and complete earthquake catalog is required to model the background seismicity. The background seismicity model estimates hazard from earthquakes on unidentified and uncharacterized faults (Petersen et al. 2014). The model is based on the assumption that future large damaging earthquakes may occur near past small and moderate earthquakes (Frankel 1995). It was derived from historical seismicity pattern (gridded and smooth spatially) to account for random earthquakes in the whole study regions. In the present study, the background seismicity model represents earthquakes from M<sub>W</sub> 4.5 to 7.5 in the areas of no mapped faults, subduction interface, deep intraslab, and earthquakes from M<sub>W</sub> 4.5 to 6.5 in the areas of characterized faults.
Figure 3.2 Seismotectonic map of Bangladesh and surrounding regions showing epicenters of earthquakes (declustered catalogue) from 1762 to 2016 (Rahman et al. 2018c). Reprinted with permission of Springer Nature.
3.3.1.1 Earthquake Catalog

The systematic record of earthquakes that occurred in Bangladesh and surrounding regions are not available in Bangladesh. In the present study, earthquake catalog of these regions was compiled from different international sources and published literatures. The sources of earthquake records are: 1) USGS Online Catalog from 1908 to October 26, 2016, 2) International Seismological Center (ISC) - Global Earthquake Model (GEM) Catalog from 1903 to September 20, 2013, 3) Global Centroid Moment Tensor (CMT) Catalog from 1976 to June 26, 2015. The published literatures are Szeliga et al. (2010) and Ambraseys and Douglas (2004). Earthquake records of different sources were compiled, reformatted and stored chronologically to prepare a composite, uniform earthquake catalog.

3.3.1.2 Magnitude Conversion

The moment magnitude ($M_w$) is the most reliable and useful magnitude scale to use in the seismic hazard analysis. The magnitude scale of all earthquake events in the compiled catalog was not moment magnitude ($M_w$). Therefore, all other magnitude scales need to be converted to moment magnitude to prepare a homogeneous earthquake catalog.

Scordilis (2006) used a large number of earthquake events that occurred all over the world during the period from 1976 to 2003 for which moment magnitude were available to derive relations between moment magnitude and other magnitude scales. The study indicated that well-defined relations exist between $M_w$ and $M_b$ (body wave magnitude) and $M_s$ (surface wave magnitude) and that can be reliably used for compiling homogeneous catalog.

The earthquake magnitudes $M_s$ between 3.0 and 8.2 and $m_b$ between 3.5 and 5.5 were converted using the following relations (Scordilis 2006):
The magnitude $M_b$ between 5.5 and 7.3 were converted using the relation below (Sipkin 2003):

$$M_w = 1.46M_b - 2.42, \quad 5.5 \leq M_b \geq 7.3$$  \hspace{1cm} Eq. 3.4

For magnitude $M_L \leq 6$, the following relation was used (Heaton et al. 1986):

$$M_w = M_L, \quad M_L \leq 6$$  \hspace{1cm} Eq. 3.5

After converting all earthquake events to moment magnitude scale, duplicate records were removed from the catalog using the recoding time and space, which were recorded at the same time and location. After deleting the duplicate records, total earthquake events collected were 4248. During deleting duplicate events, preference was given to Global Centroid Moment Tensor (CMT) Catalog, then USGS Online Catalog, and then International Seismological Center (ISC) - Global Earthquake Model (GEM) Catalog. The records of historical earthquakes were taken from International Seismological Center (ISC) - Global Earthquake Model (GEM) Catalog, Szeliga et al. (2010), and Ambraseys and Douglas (2004).

3.3.1.3 Declustering

For seismic hazard analysis, it is expected that the earthquake events in the catalog are statistically independent (Poisson assumption). Therefore, the earthquake catalog that is used in the seismic hazard analysis must be free of dependent events (foreshocks and aftershocks). The
foreshocks and aftershocks are earthquake events that are certainly connected with a main event that is generally large (Gardner and Knopoff 1974). The process of removing dependent earthquake events from the earthquake catalog is called declustering.

The earthquake catalog was declustered to eliminate foreshocks and aftershocks of a main shock using the algorithm of Gardner and Knopoff (1974). The algorithm defines that the aftershocks and foreshocks are dependent (a non-Poisson assumption) on the size of the main shock. The large magnitude earthquake event generally produces large numbers of aftershocks in a large area for a long time. Therefore, the space- and time-window of a large earthquake are larger than that of a small earthquake. The dependent events need to be eliminated to use the catalog for seismic hazard analysis. Thirty-four percent earthquake events of the catalog were identified as aftershocks and foreshocks (dependent events) using the algorithm. After removing the dependent events, the final catalog contained 2811 independent earthquake events greater than $M_w 4$ (Figure 3.2).

3.3.1.4 Catalog Completeness

The declustered earthquake catalog was not complete for the period from 1762 to 2016 (254 years). The predicted seismic rate using the earthquake events of this catalog may underestimate the occurrence of future earthquakes in these regions. The reliable mean seismic rate can be predicted by identifying the period during which the catalog is complete for particular magnitude range. Therefore, the completeness periods for different magnitude ranges were determined by using Stepp’s method (Stepp 1972).
3.3.1.5 Gutenberg-Richter Model

The earthquakes with magnitude ($M_w$) equal to or greater than 4.5 of the declustered catalog that pass a completeness test were used to estimate the parameters of the truncated Gutenberg-Richter model (Gutenberg and Richter 1944).

\[
\log_{10} \lambda_m = a - bm
\]

\[
\lambda_m = 10^a 10^{-bm}
\]

where, $\lambda_m$ is the annual rate of earthquake greater than magnitude $m$, and $a$ and $b$ are the constants.

The $a$ and $b$ values are determined from the statistical analysis of the historical and instrumental data of earthquake observations with additional information from the geological evidences of earthquake source faults (Baker 2013). The $a$ describes the overall earthquake rate in a region and the $b$ describes the ratio of small and large magnitude earthquakes.

The parameter $b$ is assumed uniform throughout the whole study regions for background seismicity model. Therefore, the $b$ value is calculated based on maximum-likelihood model (Weichert 1980) from earthquakes with $M_w$ between 4.5 and 7.5 using varying completeness times for different magnitude ranges and the $b$ value is 0.8 for whole background seismicity regions (Table 3.2).

The $a$-value varies from place to place throughout the study regions. A grid of 0.1-degree latitude and 0.1-degree longitude was overlain on the study regions. Then, the cumulative earthquake rate ($10^a$ value) was calculated by dividing the number of earthquakes equal to or
greater than $M_w$ 4.5 in a grid by the time interval of completeness. The incremental earthquake rate ($10^a$ value) was calculated from the cumulative earthquake rate (Herrmann 1977). The calculated a-grid (a-value) showed the annual rate of earthquakes with $M_w$ plus 0.05 and minus 0.05 magnitude units in each grid cell (Petersen et al. 2014).

**Table 3.2 Seismicity parameters (a- and b-values) for seismic source models.**

<table>
<thead>
<tr>
<th>Seismic Source Models</th>
<th>Magnitude Ranges ($M_w$)</th>
<th>b-value</th>
<th>a-value</th>
</tr>
</thead>
<tbody>
<tr>
<td>1. Background Seismicity (Gridded)</td>
<td>Shallow Seismicity (Focal Depth ≤ 40 km)</td>
<td>4.5 - 7.5</td>
<td>0.800</td>
</tr>
<tr>
<td></td>
<td>Deep Seismicity (Focal Depth &gt; 40 km)</td>
<td>4.5 - 7.5</td>
<td>0.800</td>
</tr>
<tr>
<td>2. Regional Shallow Seismicity</td>
<td>Crustal West</td>
<td>4.5 - 7.5</td>
<td>0.800</td>
</tr>
<tr>
<td></td>
<td>Crustal East</td>
<td>4.5 - 7.5</td>
<td>0.800</td>
</tr>
<tr>
<td></td>
<td>Chittagong-Tripura</td>
<td>4.5 - 7.5</td>
<td>0.800</td>
</tr>
<tr>
<td></td>
<td>Shillong-Assam</td>
<td>4.5 - 7.5</td>
<td>0.800</td>
</tr>
<tr>
<td></td>
<td>Subduction East</td>
<td>4.5 - 7.5</td>
<td>0.800</td>
</tr>
<tr>
<td></td>
<td>Subduction North</td>
<td>4.5 - 7.5</td>
<td>0.800</td>
</tr>
<tr>
<td>3. Crustal Faults</td>
<td>Geodetic (horizontal) strain rate (mm/year)</td>
<td>Dauki Fault</td>
<td>6.5 - 8.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dapsi Fault</td>
<td>6.5 - 7.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Dhuhri Fault</td>
<td>6.5 - 7.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Oldham Fault</td>
<td>6.5 - 8.1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Kopili Fault</td>
<td>6.5 - 7.7</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Churachandpur-Mao Fault</td>
<td>6.5 - 7.75</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Kabaw Fault</td>
<td>6.5 - 7.92</td>
</tr>
<tr>
<td>4. Subduction Zones</td>
<td>Chittagong-Tripura Section (subduction interface)</td>
<td>7.5 - 8.6</td>
<td>0.0001</td>
</tr>
<tr>
<td></td>
<td>Manipur-Mizoram Section (subduction intraslab)</td>
<td>7.5 - 8.0</td>
<td>0.0001</td>
</tr>
<tr>
<td></td>
<td>Ramree Section (subduction interface)</td>
<td>7.5 - 8.6</td>
<td>0.0001</td>
</tr>
<tr>
<td></td>
<td>Magway Section (subduction intraslab)</td>
<td>7.5 - 8.0</td>
<td>0.0001</td>
</tr>
<tr>
<td></td>
<td>Nagaland Section (subduction interface)</td>
<td>7.5 - 8.6</td>
<td>0.0001</td>
</tr>
<tr>
<td></td>
<td>Kachin Section (subduction intraslab)</td>
<td>7.5 - 8.0</td>
<td>0.0001</td>
</tr>
<tr>
<td></td>
<td>Nepal Section (subduction interface)</td>
<td>7.5 - 8.6</td>
<td>0.0001</td>
</tr>
<tr>
<td></td>
<td>Bhutan Section (subduction interface)</td>
<td>7.5 - 8.6</td>
<td>0.0001</td>
</tr>
<tr>
<td></td>
<td>Assam Section (subduction interface)</td>
<td>7.5 - 8.6</td>
<td>0.0001</td>
</tr>
</tbody>
</table>
In the present study, two background seismicity models have been developed for the study regions based on the focal depths of earthquakes that occurred in these regions. The models are shallow crustal background seismicity (focal depth ≤ 40 km) and deep background seismicity (focal depth > 40 km). The a-grid values of shallow crustal and deep seismicity models were smoothed spatially by a two-dimensional Gaussian function of Kernel interpolation using ArcGIS (Figure 3.3 and Figure 3.4). The a-grid and b values of the background seismicity models (shallow crustal and deep seismicity) were used for seismic hazard analysis.

![Figure 3.3 Background shallow crustal (focal depth ≤ 40 km) seismicity (gridded and spatially smooth) rate (a-value).](image)

Figure 3.3 Background shallow crustal (focal depth ≤ 40 km) seismicity (gridded and spatially smooth) rate (a-value).
Regional shallow crustal seismicity zone represents uniform seismicity throughout the tectonic or geologic region with constant geologic or strain characteristics (Petersen et al. 2008). Regional zone was used to calculate an average floor-rate of shallow crustal seismicity to account for future

Figure 3.4 Background deep (focal depth > 40 km) seismicity (gridded and spatially smooth) rate (a-value).
random earthquakes in areas with little or no historical seismicity. The study regions were divided into six regional shallow crustal zones based on the tectonic, geologic, and seismic characteristics of the regions. The zones are 1) crustal west, 2) crustal east, 3) Chittagong-Tripura, 6) Shillong-Assam, 5) subduction east, and 5) subduction north (Figure 3.5).

Figure 3.5 Regional shallow crustal seismicity zones with earthquake epicenters of shallow crustal (focal depth ≤ 40 km).
The rate of gridded shallow crustal seismicity was compared with the rate of regional shallow crustal seismicity for each grid cell. When the gridded rate was higher than the regional rate, the cell was assigned the gridded rate. And when the gridded rate was lower than the regional rate, the gridded and regional seismicity models were assigned weights of 0.67 and 0.33, respectively (Figure 3.6) as used in the United States National Earthquake Hazard Maps of 2014 for background seismicity (Petersen et al. 2014). The final cell seismicity was never below the gridded seismicity. The gridded deep seismicity model was given weight of 1.0 (Figure 3.6).

Figure 3.6 Logic tree for background seismicity-based source models of this study. The assigned branch weights are shown in the parentheses. Shallow crustal ground motion prediction equations (GMPEs): ASK14-Abrahamson et al. (2014), BSSA14-Boore et al. (2014), CB14-Campbell and Bozorgnia (2014), CY14-Chiou and Youngs (2014), and I14-Idriss (2014). Deep seismic GMPEs: BC Hydro12-Abrahamson et al. (2016), ZH06-Zhao et al. (2006), and AB03-Atkinson and Boore (2003).
3.3.3 Crustal Fault Source Model

Crustal faults may generate sufficiently large magnitude earthquakes. Therefore, it is a standard practice to include the explicit fault sources to supplement the background seismicity sources to extend the record of historical earthquakes date back to thousands of years (Petersen et al. 2014). Most of the crustal faults in Bangladesh were not identified and characterized in terms of their locations, extents, slip rates, and timing of ruptures. Some well-known crustal faults that are located within 300 km of Bangladesh boundary, generated earthquakes up to a maximum magnitude of $M_w$ 8.1 (Kayal et al. 2012). The geologic slip rates of these faults were not determined using paleoseismological trenching and seismicity studies.

Studies revealed that there is a good correlation between geologic slip rate and geodetic strain rate (McCaffrey et al. 2013, Petersen et al. 2013). Therefore, geodetic strain rate can be used to determine the activities of the faults. The geologic slip rate estimation using trenching and seismicity studies has been developed during the past century, whereas the geodetic strain data was beginning to be used in seismic hazard analysis in recent years (Petersen et al. 2014). The recording time of geodetic data was not sufficient to estimate the slip rate of the faults. Petersen et al. (2014) used 50% of the geodetic strain rate to characterize the shear zones of crustal faults in Western US, as some slips possibly did not generate earthquakes and some slips were released by small earthquakes and aftershocks that were not considered in the hazard analysis. Therefore, in the present study, 50% of geodetic strain rate was used to characterize the crustal faults. The characterized faults of the present study are shown in Figure 3.7 and their modeling parameters are listed in Table 3.2 and Table 3.3. The weights of the parameters to model the fault sources are shown in an example logic tree for Dauki fault (Figure 3.8). The geodetic strain rates for the crustal faults were used from Barman et al. (2017), Gahalaut et al. (2013), and Socquet et al. (2006) (Table 3.3).
The segmentation model for Dauki fault was according to Morino et al. (2014). The latest maximum magnitude earthquakes of crustal fault sources were taken from England and Bilham (2015), Kayal (2008), Ambraseys and Douglas (2004), and Bilham and England (2001).

Figure 3.7 Crustal faults that are characterized to use in seismic hazard analysis.
Figure 3.8 Logic tree for crustal fault (Dauki fault) source models of this study. The assigned branch weights are shown in the parentheses. Shallow crustal ground motion prediction equations (GMPEs): ASK14- Abrahamson et al. (2014), BSSA14- Boore et al. (2014), CB14- Campbell and Bozorgnia (2014), CY14- Chiou and Youngs (2014), and I14- Idriss (2014).
### Table 3.3 Parameters for crustal fault source models

<table>
<thead>
<tr>
<th>No.</th>
<th>Fault source</th>
<th>Fault rupture model</th>
<th>Rupture length (km)</th>
<th>Seismogenic depth (km)</th>
<th>Sense of slip</th>
<th>Dip direction</th>
<th>Dip angle</th>
<th>Width (km)</th>
<th>Down-dip rupture width (km)</th>
<th>Geodetic strain rate (mm/yr)</th>
<th>50% of strain rate (mm/yr)</th>
<th>Sources</th>
<th>Characteristic or maximum magnitude (Mw)</th>
<th>Observed maximum magnitude (Mw)</th>
<th>Date of last event</th>
<th>Sources</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Dauki Fault</td>
<td>Full-source</td>
<td>350 40</td>
<td>Thrust</td>
<td>North</td>
<td>45 40.0</td>
<td>56.6</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td>Barman et al. (2017)</td>
<td>4.95</td>
<td>8.10</td>
<td>1897 (Great Assam Earthquake)</td>
<td>Ambraseys and Douglas (2004)</td>
</tr>
<tr>
<td></td>
<td>Dauki Fault (segmented)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>45 40.0</td>
<td>56.6</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Morino et al. (2014)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>45 40.0</td>
<td>56.6</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>60 23.1</td>
<td>46.2</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>60 23.1</td>
<td>46.2</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>350 40 Reverse</td>
<td>60 23.1</td>
<td>46.2</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>77 40 Reverse</td>
<td>60 23.1</td>
<td>46.2</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>130 40 Reverse</td>
<td>60 23.1</td>
<td>46.2</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>65 40 Reverse</td>
<td>60 23.1</td>
<td>46.2</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Eastern</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>65 40 Reverse</td>
<td>60 23.1</td>
<td>46.2</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Eastern most</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>106 40 Reverse</td>
<td>60 23.1</td>
<td>46.2</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Dauki Fault</td>
<td>Floating-source</td>
<td>130 40</td>
<td>Thrust</td>
<td>North</td>
<td>45 40.0</td>
<td>56.6</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td>4.95</td>
<td>7.43</td>
<td>7.00</td>
<td>8.03</td>
<td>1897 (Great Assam Earthquake)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>45 40.0</td>
<td>56.6</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>60 23.1</td>
<td>46.2</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>60 23.1</td>
<td>46.2</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>60 23.1</td>
<td>46.2</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>350 40 Reverse</td>
<td>60 23.1</td>
<td>46.2</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>350 40 Reverse</td>
<td>60 23.1</td>
<td>46.2</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Dapsi Fault</td>
<td>Full-source</td>
<td>123 40</td>
<td>Thrust</td>
<td>North</td>
<td>45 40.0</td>
<td>56.6</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td>7.00</td>
<td>4.95</td>
<td>7.50</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>60 23.1</td>
<td>46.2</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Dhubri Fault</td>
<td>Full-source</td>
<td>60 40</td>
<td>Strike slip</td>
<td>North</td>
<td>90 40.0</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td>8.10</td>
<td>3.50</td>
<td>7.10</td>
<td>7.1</td>
<td>1930 (Dhubri Earthquake)</td>
<td>Eangland and Bilham (2015)</td>
</tr>
<tr>
<td>4</td>
<td>Oldham Fault</td>
<td>Full-source</td>
<td>110 40</td>
<td>Thrust</td>
<td>North</td>
<td>45 40.0</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td>7.00</td>
<td>3.50</td>
<td>8.10</td>
<td>8.1</td>
<td>1897 (Great Assam Earthquake)</td>
<td>Eangland and Bilham (2001)</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>60 23.1</td>
<td>46.2</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>60 23.1</td>
<td>46.2</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>60 23.1</td>
<td>46.2</td>
<td>7.00</td>
<td>3.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Kopili Fault</td>
<td>Full-source</td>
<td>180 10.2</td>
<td>Strike slip</td>
<td>North</td>
<td>90 10.2</td>
<td>4.70</td>
<td>2.60</td>
<td></td>
<td></td>
<td>2.35</td>
<td>7.70</td>
<td>7.7</td>
<td></td>
<td>1869 (Cacher Earthquake)</td>
<td>Kayal (2008)</td>
</tr>
<tr>
<td>6</td>
<td>Charachandpur-Mao Fault</td>
<td>Full-source</td>
<td>200 40</td>
<td>Strike slip</td>
<td>South</td>
<td>90 40.0</td>
<td>16.00</td>
<td>8.00</td>
<td></td>
<td>8.00</td>
<td>7.92</td>
<td>7.7</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>20 40.0</td>
<td>28.3</td>
<td>9.00</td>
<td>4.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>Kabaw Fault</td>
<td>Full-source</td>
<td>280 20</td>
<td>Thrust</td>
<td>East</td>
<td>45 20.0</td>
<td>9.00</td>
<td>4.50</td>
<td></td>
<td></td>
<td>6.36</td>
<td>9.00</td>
<td>7.92</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>115 20.0</td>
<td>23.1</td>
<td>9.00</td>
<td>4.50</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

52
The rupture model of Dauki fault was characterized as full-source, segmented source, and floating source. Paleoseismological studies suggested that in addition to the 1897 Great Assam Earthquake ($M_w > 8.0$), three large magnitude earthquakes ($M_w > 8.0$) occurred along Dauki fault (Sukhija et al. 1999a, 1999b). Large magnitude earthquake ($M_w \geq 8.0$) occurred due to rupture along the full length of Dauki fault (350 km). Morino et al. (2014) proposed a segmented rupture model for Dauki fault based on paleoseismological study. Ruptures that were permitted to break through the segment boundaries were termed as floating ruptures that may occur anywhere along the fault (Petersen et al. 2014). The 1897 Great Assam Earthquake ($M_w > 8.0$) and 1923 Mymensigh Earthquake ($M_w > 7.0$) have proved the occurrence of earthquakes due to full-source and segmented or floating source ruptures, respectively. Kayal et al. (2012) also suggested that complex tectonics and tectonic stress due to rapid shortening across the Dauki fault in Shillong Plateau may cause earthquake $M_w > 7.0$ any time, if an earthquake $M_w > 8.0$ earthquake do not occur immediately as predicted by Bilham and England (2001). Therefore, the logic tree weights for full-source, segmented source, and floating source were assigned as 0.4, 0.4, and 0.2, respectively (Figure 3.8).

Dauki fault is an east-west trending north-dipping reverse or thrust fault (Kayal 2001, Morino et al. 2011). The geometry of the fault has the dip uncertainty. Therefore, equal weight (0.5) was assigned for reverse and thrust faults.

### 3.3.4 Crustal Fault Recurrence Models

Both characteristic and Gutenberg-Richter recurrence models were used to characterize each fault source with a logic tree weight of 0.5 for each model, except Dauki fault that was characterized using the characteristic (full-source model) weight of 0.4, segmented (Gutenberg-
Richter model) weight of 0.4 and floating (Gutenberg-Richter model) weight of 0.2 in the logic tree branches (*Figure 3.8*). The characteristic magnitude of the full-rupture source and the maximum magnitude of the individual fault segment were calculated by using the following scaling relationship of Wells and Coppersmith (1994), which was proposed for all types of slip (normal, reverse, and strike slip):

\[ M_w = 5.08 + 1.16 \log \text{(SRL)} \]

*Eq. 3.8*

where, SRL is the surface rupture length (km).

To account for the uncertainty, the characteristic or maximum magnitude was fixed with an upper and a lower bound as \( M_c \text{ or max} - 0.2, M_c \text{ or max}, \) and \( M_c \text{ or max} + 0.2, \) and for these values of magnitude, the logic tree weights were assigned as 0.2, 0.6, and 0.2, respectively (*Figure 3.8*). The minimum magnitude of the Gutenberg-Richter model was set as \( M_w 6.5. \)

In case of characteristic and Gutenberg-Richter models, the b-values (ratio of small and large earthquakes) were assigned as 0.0001 (Allen et al. 2015) and 0.80 (regional b-value), respectively. The a-value (activity rate) was determined from the seismic moment and geodetic strain rate using the well-accepted relationships of Youngs and Coppersmith (1985) by using EZ-Frisk Software developed by Risk Engineering Inc., USA.

The seismic moment \( (M_o) \) is related to the rigidity or shear modulus of the earth’s crust \( (\mu, \) usually \( 3 \times 10^{11} \text{ dyne/cm}^2 \)), area of the fault slip \( (A) \), and average slip on the fault plane \( (D) \) by the following relationship (Brune 1968):

\[ M_o = \mu A D \]

*Eq. 3.9*
The seismic moment rate ($\dot{M}_o$) was estimated dividing both sides of the above equation by the time (years) of fault slip as follows:

$$\dot{M}_o = \mu A S \quad Eq. 3.10$$

where, $S$ is the slip rate.

The seismic moment ($M_o$) was estimated from the moment magnitude ($M_w$) by using the following relationship of (Hanks and Kanamori 1979):

$$\log M_o = 1.5 M_w + 16.1 \quad Eq. 3.11$$

### 3.3.5 Subduction Zone Source Model

The Indian Plate is subducting in the north and east under the Eurasian Plate due to convergence between the plates. Large magnitude earthquakes ($M_w > 8$) may occur along the plate convergent boundary megathrusts (Ujiie and Kimura 2014, Steckler et al. 2016). Therefore, the subduction zones of the study regions were characterized as subduction interface and subduction intraslab source zones based on their tectonic and seismic characteristics that were described in the existing literatures. The study regions were divided into nine subduction zones (interface and intraslab) to model the parameters of each zone (Figure 3.9). The parameters of the subduction source models are shown in Table 3.2 and Table 3.4. The subduction source model is illustrated using an example logic tree for Chittagong-Tripura Section (1 and 2) of subduction interface (Figure 3.10).
Figure 3.9. Subduction zones to estimate the seismicity parameters (a- and b-values)
<table>
<thead>
<tr>
<th>No.</th>
<th>Subduction seismic source</th>
<th>Rupture model</th>
<th>Rupture length (km)</th>
<th>Seismogenic depth (km)</th>
<th>Sense of slip</th>
<th>Dip direction</th>
<th>Dip angle</th>
<th>Width (km)</th>
<th>Down-dip rupture width (km)</th>
<th>Geodetic strain rate (mm/yr)</th>
<th>50% of strain rate (mm/yr)</th>
<th>Sources</th>
<th>Characteristic or maximum magnitude ($M_c$)</th>
<th>Observed maximum magnitude ($M_{max}$)</th>
<th>Date of last event</th>
<th>Sources</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Chattagong-Tripura Section-1 (interface)</td>
<td>Full-source</td>
<td>480</td>
<td>40</td>
<td>Thrust</td>
<td>East</td>
<td>11.31</td>
<td>200.0</td>
<td>204.0</td>
<td>7.00</td>
<td>3.50</td>
<td>Steckler et al. (2016)</td>
<td>3.57</td>
<td>8.60</td>
<td>1950 (Assam-Tibet Earthquake)</td>
<td>Ambraseys and Douglas (2004)</td>
</tr>
<tr>
<td></td>
<td>Chattagong-Tripura Section-2 (interface)</td>
<td>Floating-source</td>
<td>480</td>
<td>40</td>
<td>Thrust</td>
<td>East</td>
<td>11.31</td>
<td>200.0</td>
<td>204.0</td>
<td>7.00</td>
<td>3.50</td>
<td>3.57</td>
<td>8.60</td>
<td>1934 (Nepal-Bihar Earthquake)</td>
<td>Ambraseys and Douglas (2004)</td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Manipur-Mizoram Section-1 (intraslab)</td>
<td>Full-source</td>
<td>360</td>
<td>160</td>
<td>Thrust</td>
<td>East</td>
<td>38.65</td>
<td>200.0</td>
<td>256.2</td>
<td>10.00</td>
<td>5.00</td>
<td>Socquet et al. (2006)</td>
<td>6.40</td>
<td>8.00</td>
<td>1762 (Wang et al. 2014)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Manipur-Mizoram Section-2 (intraslab)</td>
<td>Floating-source</td>
<td>360</td>
<td>160</td>
<td>Thrust</td>
<td>East</td>
<td>38.65</td>
<td>200.0</td>
<td>256.2</td>
<td>10.00</td>
<td>5.00</td>
<td>6.40</td>
<td>8.00</td>
<td>1762 (Wang et al. 2014)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Manipur-Mizoram Section-3 (intraslab)</td>
<td>Full-source</td>
<td>360</td>
<td>160</td>
<td>Thrust</td>
<td>East</td>
<td>38.65</td>
<td>200.0</td>
<td>256.2</td>
<td>10.00</td>
<td>5.00</td>
<td>6.40</td>
<td>8.00</td>
<td>1762 (Wang et al. 2014)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>Manipur-Mizoram Section-4 (intraslab)</td>
<td>Floating-source</td>
<td>360</td>
<td>160</td>
<td>Thrust</td>
<td>East</td>
<td>38.65</td>
<td>200.0</td>
<td>256.2</td>
<td>10.00</td>
<td>5.00</td>
<td>6.40</td>
<td>8.00</td>
<td>1762 (Wang et al. 2014)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Ramree Section (interface)</td>
<td>Full-source</td>
<td>450</td>
<td>40</td>
<td>Thrust</td>
<td>East</td>
<td>14.00</td>
<td>160.0</td>
<td>165.3</td>
<td>14.00</td>
<td>7.00</td>
<td>Wang et al. (2014)</td>
<td>7.21</td>
<td>8.60</td>
<td>8.5 - 8.8</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Ramree Section (interface)</td>
<td>Floating-source</td>
<td>450</td>
<td>40</td>
<td>Thrust</td>
<td>East</td>
<td>14.00</td>
<td>160.0</td>
<td>165.3</td>
<td>14.00</td>
<td>7.00</td>
<td>7.21</td>
<td>8.00</td>
<td>1762 (Wang et al. 2014)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Magway Section (intraslab)</td>
<td>Full-source</td>
<td>450</td>
<td>160</td>
<td>Thrust</td>
<td>East</td>
<td>36.87</td>
<td>200.0</td>
<td>266.7</td>
<td>9.00</td>
<td>4.50</td>
<td>5.63</td>
<td>8.00</td>
<td>1934 (Nepal-Bihar Earthquake)</td>
<td>Ambraseys and Douglas (2004)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Magway Section (intraslab)</td>
<td>Floating-source</td>
<td>450</td>
<td>160</td>
<td>Thrust</td>
<td>East</td>
<td>36.87</td>
<td>200.0</td>
<td>266.7</td>
<td>9.00</td>
<td>4.50</td>
<td>5.63</td>
<td>8.00</td>
<td>1934 (Nepal-Bihar Earthquake)</td>
<td>Ambraseys and Douglas (2004)</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Nagaland Section (interface)</td>
<td>Full-source</td>
<td>460</td>
<td>40</td>
<td>Thrust</td>
<td>East</td>
<td>18.43</td>
<td>120.0</td>
<td>126.5</td>
<td>2.50</td>
<td>1.25</td>
<td>Wang et al. (2014)</td>
<td>1.32</td>
<td>8.60</td>
<td>1934 (Nepal-Bihar Earthquake)</td>
<td>Ambraseys and Douglas (2004)</td>
</tr>
<tr>
<td></td>
<td>Nagaland Section (interface)</td>
<td>Floating-source</td>
<td>460</td>
<td>40</td>
<td>Thrust</td>
<td>East</td>
<td>18.43</td>
<td>120.0</td>
<td>126.5</td>
<td>2.50</td>
<td>1.25</td>
<td>1.32</td>
<td>8.00</td>
<td>1934 (Nepal-Bihar Earthquake)</td>
<td>Ambraseys and Douglas (2004)</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>Kachin Section (intraslab)</td>
<td>Full-source</td>
<td>340</td>
<td>160</td>
<td>Thrust</td>
<td>East</td>
<td>53.00</td>
<td>100.0</td>
<td>200.3</td>
<td>2.50</td>
<td>1.25</td>
<td>2.08</td>
<td>8.00</td>
<td>1934 (Nepal-Bihar Earthquake)</td>
<td>Ambraseys and Douglas (2004)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Kachin Section (intraslab)</td>
<td>Floating-source</td>
<td>340</td>
<td>160</td>
<td>Thrust</td>
<td>East</td>
<td>53.00</td>
<td>100.0</td>
<td>200.3</td>
<td>2.50</td>
<td>1.25</td>
<td>2.08</td>
<td>8.00</td>
<td>1934 (Nepal-Bihar Earthquake)</td>
<td>Ambraseys and Douglas (2004)</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>Assam Section (interface)</td>
<td>Full-source</td>
<td>400</td>
<td>17</td>
<td>Thrust</td>
<td>North</td>
<td>7.45</td>
<td>130.0</td>
<td>131.1</td>
<td>23.00</td>
<td>11.50</td>
<td>Berthet et al. (2014)</td>
<td>11.60</td>
<td>8.60</td>
<td>1950 (Assam-Tibet Earthquake)</td>
<td>Ambraseys and Douglas (2004)</td>
</tr>
<tr>
<td></td>
<td>Assam Section (interface)</td>
<td>Floating-source</td>
<td>400</td>
<td>17</td>
<td>Thrust</td>
<td>North</td>
<td>7.45</td>
<td>130.0</td>
<td>131.1</td>
<td>23.00</td>
<td>11.50</td>
<td>11.60</td>
<td>8.44</td>
<td>1950 (Assam-Tibet Earthquake)</td>
<td>Ambraseys and Douglas (2004)</td>
<td></td>
</tr>
</tbody>
</table>
Recent studies revealed that the Bangladesh portion of the megathrust in the eastern boundary of the Indian Plate (Figure 3.9) is concealed under the thick sediments of the Ganges-Brahmaputra Delta (Wang et al. 2014, Steckler et al. 2016). Some other studies showed that the Bangladesh portion of the megathrust runs along the Chittagong Coastal Fault (CDMP 2009, Kundu and Gahalaut 2013). The seismicity data indicated that Chittagong-Tripura Section-2 (interface) contained more earthquake events than Chittagong-Tripura Section-1 (interface) (Figure 3.1 and Figure 3.9). Therefore, Chittagong-Tripura Section-2 (interface) is seismically more active than...
Chittagong-Tripura Section-1 (interface). The accurate location of the Bangladesh portion of the megathrust is very important for the seismic hazard analysis of Bangladesh. To reduce the uncertainty in locating the megathrust plate boundary in Bangladesh, in the present study, Chittagong-Tripura Section-1 (interface) and Chittagong Tripura Section-2 (interface) were assigned the logic tree weights of 0.25 and 0.75, respectively (Figure 3.10). Similarly, the logic tree weights of 0.25 and 0.75 were assigned to Manipur-Mizoram Section-1 (intraslab) and Manipur-Mizoram Section-2 (intraslab), respectively. The logic tree weight for other interface and intraslab subduction sources was assigned as 1.0.

The geodetic strain rates for the subduction models were used from Barman et al. (2017), Steckler et al. (2016), Wang et al. (2014), Berthet et al. (2014), and Socquet et al. (2006) (Table 3.4). The latest maximum magnitude earthquakes of subduction sources were taken from Wang et al. (2014) and Ambraseys and Douglas (2004).

3.3.6 Subduction Zone Recurrence Models

Historical records indicate that large magnitude \( (M_w > 8.0) \) earthquake did not occur in Chittagong-Tripura Section (1 and 2) during the last 400 years. Steckler et al. (2016) predicted that a characteristic earthquake of \( M_w 8.2 - 9.0 \) might occur due to full-rupture of Chittagong-Tripura Section of the megathrust. As earthquakes of \( M_w \geq 7.0 \) did not occur in this section at a regular interval, earthquakes of \( M_w 7.5 \) to 8.6 also might occur due to floating rupture at any location in this section. The probability of occurrence of characteristic earthquake is very high in this seismic gap of the megathrust. Therefore, the recurrence models of characteristic and Gutenberg-Richter (floating rupture) were used with a logic tree weight of 0.90 and 0.10, respectively. The characteristic and maximum magnitude were calculated by using the relationships of Blaser et al.
(2010) and Strasser et al. (2010), which were proposed for scaling of earthquake source by using source length for interface and intraslab subduction earthquakes (Eq. 2.12, Eq. 2.13, and Eq. 2.14). For each relationship, a logic tree weight of 0.5 was assigned. The characteristic or maximum magnitude was fixed with an upper and a lower bound as $M_{c or max} - 0.2$, $M_{c or max}$, and $M_{c or max} + 0.2$ and the logic tree weights were assigned as 0.2, 0.6, and 0.2, respectively (Figure 3.10). The minimum magnitude of the Gutenberg-Richter model (floating rupture) in the subduction zones was set as $M_w7.5$ to avoid double counting of the hazard, as the maximum magnitude of the background seismicity was set as $M_w7.5$.

\[
\text{Log } L = -2.310 + 0.570 M_w \quad \text{(Blaser et al. 2010)} \tag{Eq. 3.12}
\]

\[
\text{Log } L = -2.477 + 0.585 M_w \quad \text{(interface) (Strasser et al. 2010)} \tag{Eq. 3.13}
\]

\[
\text{Log } L = -2.350 + 0.562 M_w \quad \text{(intraslab) (Strasser et al. 2010)} \tag{Eq. 3.14}
\]

where, $L$ is the rupture length (km).

The $b$-value for characteristic and Gutenberg Richter (floating rupture) models was used as 0.0001. The Gutenberg-Richter frequency distribution was used from $M_w$ 7.5 to 8.6 for floating rupture. The $b$-value close to zero (0.0001) for floating rupture indicates equal probability of occurring an earthquake $M_w7.5$ or $M_w8.6$ (Petersen et al. 2014). The $a$-values (activity rate) was calculated from the seismic moment and geodetic strain rate using well-accepted relationship of Youngs and Coppersmith (1985) by using EZ-Frisk Software.
3.4 Ground Motion Prediction Equations (GMPEs)

The seismic ground motion at a site was measured as a function of magnitude and distance from the source by using ground motion prediction equations (GMPEs). The GMPEs were empirically derived from a large number of earthquake ground motion data by using statistical regression analysis. It is desirable to use the GMPEs of the study region, which were derived using the ground motion data of that region. Deriving precise GMPEs using limited ground motion data of this region was not possible. Therefore, the best alternative was to use the GMPEs of similar tectonic environment of other regions.

Five sets of GMPEs were developed in 2008 for shallow crustal earthquakes using five global data sets of ground motions by five research groups through the Pacific Earthquake Engineering Research Center’s (PEER) Next Generation Attenuation for the western United States (NGA West) program. The GMPEs were updated in 2014 by those research groups using more global and recent ground motion data through the NGA West-2 program (Abrahamson et al. 2014, Boore et al. 2014, Campbell and Bozorgnia 2014, Chiou and Youngs 2014, Idriss 2014). In the present study, the GMPEs of the NGA West-2 were used for shallow crustal earthquake sources (shallow background seismicity and crustal faults) to estimate peak ground acceleration (PGA), spectral acceleration (SA) for a site. The logic tree weights of the GMPEs are shown in Figure 3.6 and Figure 3.8. The weights were assigned similar to the weights that are used for the United States seismic hazard map of 2014 (Petersen et al. 2014).

The five teams of the horizontal GMPE developers for shallow crustal earthquakes are listed below:

1. Abrahamson, Silva, and Kamai (ASK14)
2. Boore, Stewart, Seyhan, and Atkinson (BSSA 14)
3. Campbell and Bozorgnia (CB 14)
4. Chiou and Youngs (CY 14)
5. Idriss (I 14)

The details of these ground motion prediction equations (GMPEs) were described in Abrahamson et al. (2014), Boore et al. (2014), Campbell and Bozorgnia (2014), Chiou and Youngs (2014), and Idriss (2014).

The GMPE of Abrahamson et al. (2014) for median ground motion estimation is mentioned below:

\[
\ln S_a(g) = f_1(M, R_{RUP}) + F_{RV} f_7(M) + F_N f_8(M) + F_{AS} f_{11}(CR_{JB}) + \\
f_5(\bar{S}_{A_{1180}V_S^{30}}) + F_{HW} f_4(R_{JB}, R_{RUP}, R_x, R_{y0}, W, dip, Z_{TOR}, M) + f_6(Z_{TOR}) + \\
f_{10}(Z_1, V_S^{30}) + Regional(V_{S30}R_{RUP})
\]  

Eq. 3.15

where, \( f_1, f_4, f_5, f_6, f_7, f_8, f_{10}, \) and \( f_{11} \) are the coefficients. The parameters of Eq. 3.15 are defined as: \( M \): Moment magnitude, \( CR_{JB} \): Centroid \( R_{JB} \) (see Abrahamson et al. 2014 for a detailed description), \( Z_{TOR} \): Depth to top of rupture (km), \( F_{RV} \): Flag for reverse faulting earthquakes, \( F_N \): Flag for normal faulting earthquakes, \( F_{AS} \): Flag for aftershocks, \( R_{RUP} \): Rupture distance (km), \( V_S^{30} \): Shear wave velocity in the top 30 m (m/s), \( Z_1 \): Depth to \( V_S = 1.0 \) km/s at the site, \( \bar{S}_{A_{1180}} \): Median peak spectral acceleration (g) for \( V_{S30} = 1.180 \) m/s, \( F_{HW} \): Flag for hanging wall sites, \( R_{JB} \): Joyner-Boore distance (km), \( R_x \): Horizontal distance (km) from top edge of rupture, \( R_{y0} \): Horizontal distance (km) off the end of the rupture measure parallel to strike, \( Dip \): Fault dip in degrees, and \( W \): Down dip rupture width.
The GMPE of Boore et al. (2014) for median ground motion estimation is stated below:

$$\ln Y = F_E(M, mech) + F_P(R_{JB}, M, region) + F_S(V_s^{30}, R_{JB}, M, region, z_1)$$

$$+ \mathcal{E}_n \sigma(M, R_{JB}, V_s^{30})$$  \hspace{1cm} \text{Eq. 3.16}$$

where, $\ln Y$ is the natural logarithm of a ground motion IM (PGA, PGV, or PSA); FE, FP, and FS represent functions for source (E for event, P for path, and S for site) effects, respectively; $\mathcal{E}_n$ stands for the fractional number of standard deviations of a single predicted value of $\ln Y$ away from the mean; and $\sigma$ represent the total standard deviation of the model. The predictor variables are $M, mech, R_{JB}$ (in km), $region, V_s^{30}$ (in m/s) and $z_1$ (in km). Parameter $mech$ (mechanism) is 0, 1, 2, and 3 for unspecified SS (strike slip), NS (normal slip), RS (reverse slip), respectively. Parameter $region$ is 0 if no regional correction is to be performed (default value). The PGA and PGV are in g (gravitational acceleration) and cm/s, respectively.

The GMPE of Campbell and Bozorgnia (2014) for median ground motion estimation (PGA, PGV, and PSA) is given below:

$$\ln Y = \begin{cases} 
\ln PGA; & \text{PSA} < \text{PGA and } T < 0.25 \text{ s} \\
\sum f \text{- terms}; & \text{otherwise}
\end{cases} \hspace{1cm} \text{Eq. 3.17}$$

where, $Y$ represents the ground motion intensity measure of interest and the $f$-terms are the scaling of the ground motion with respect to earthquake magnitude, geometric attenuation, sense of faulting, hanging wall geometry, site response, basin response, hypocentral depth, fault dip, and anelastic attenuation, respectively. The PGA (in g), PSA (in g), $T$ (in second) stand for peak ground acceleration, pseudo-spectral acceleration, and spectral period, respectively.
The GMPE of Chiou and Youngs (2014) for median ground motion estimation is given below:

\[
\ln(y_{refij}) = c_1 + \left\{c_{1a} + \frac{c_{1c}}{\cosh(2 \cdot \max(M_i - 4.5, 0))}\right\} F_{RVi}
\]
\[
+ \left\{c_{1b} + \frac{c_{1d}}{\cosh(2 \cdot \max(M_i - 4.5, 0))}\right\} F_{NMI}
\]
\[
+ \left\{c_7 + \frac{c_{7b}}{\cosh(2 \cdot \max(M_i - 4.5, 0))}\right\} \Delta Z_{TORi}
\]
\[
+ \left\{c_{11} + \frac{c_{11b}}{\cosh(2 \cdot \max(M_i - 4.5, 0))}\right\} (\cos \delta_i)^2
\]
\[
+ c_2(M_i - 6) + \frac{c_2 - c_3}{c_n} \ln \left(1 + e^{c_n(c_M - M_i)}\right)
\]
\[
+ c_4 \ln \left(\frac{R_{RUPij} + c_5 \cosh(c_6 \cdot \max(M_i - c_{HM}, 0))}{R_{RUPij}}\right)
\]
\[
+(c_{4a} - c_4 \ln \left(\sqrt{\frac{R_{RUPij}^2 + c_{RB}^2}{R_{RUPij}}}\right)
\]
\[
+ \left\{c_{y1} + \frac{c_{y2}}{\cosh(\max(M_i - c_{y3}, 0))}\right\} R_{RUPij}
\]
\[
+C_{5\max} \left(1 - \frac{\max(R_{RUPij} - 40, 0)}{30}, 0\right)
\]
\[
\times \min \left(\max(M_i - 5.5, 0) \div 0.8, 1\right) e^{-c_{3a}(M_i - c_{3b}) \Delta DPP_{ij}}
\]
\[
+c_9 F_{HWij} \cos \delta_i \left\{c_{9a} + (1 - c_{9a}) \tanh \left(\frac{R_{Xi}}{c_{9b}}\right)\right\} \left\{1 - \frac{\sqrt{R_{JBij}^2 + Z_{TORi}^2}}{R_{RUPij} + 1}\right\} \quad \text{Eq. 3.18}
\]
\[
\ln(y_{ij}) = \ln(y_{refij}) + \eta_i + \phi_1 \cdot \min\left(\ln\left(\frac{V_{s30}}{1130}\right), 0\right)
\]
\[
+ \phi_2 \left( e^{\phi_3 \left( \min\left(\frac{V_{s30}}{1130}, 360\right) - e^{\phi_4 (1130-360)} \right)} \right) \ln\left(\frac{y_{refij} e^{\eta_i} + \phi_4}{\phi_4}\right)
\]
\[
+ \phi_5 \left( 1 - \frac{e^{\Delta Z_{1.0j}}}{\phi_6} \right) + \epsilon_{jj}
\]

\text{Eq. 3.19}

The \( y_{ij} \) in Eq. 3.19 is the ground motion intensity for earthquake \( i \) at station \( j \). The \( y_{refij} \) is the population median ground motion intensity for reference condition \( V_{s30} = 1130 \text{ m/s} \). The random variables \( \eta_i \) and \( \epsilon_{jj} \) are the modeling errors. The predictor variables in Eq. 3.18 and Eq. 3.19 are: \( M \) = Moment magnitude; \( R_{RUP} \) = Closest distance (km) to the ruptured plane; \( R_{JB} \) = Closest distance (km) to the surface projection of the ruptured plane; \( R_X \) = Site coordinate (km) measured perpendicular to the fault strike from the fault line, with the down-dip direction being positive; \( F_{HW} \) = Hanging wall flag: 1 for \( R_X \geq 0 \) and 0 for \( R_X < 0 \); \( \delta \) = Fault dip angle; \( Z_{TOR} \) = Depth (km) to the top of the ruptured plane; \( \Delta Z_{TOR} = Z_{TOR} \) centered on the \( M \)-dependent average \( Z_{TOR} \) (km); \( F_{RV} \) = Reverse faulting flag: 1 for \( 30^0 \leq \lambda \leq 150^0 \) (combined reverse and reverse-oblique), 0 otherwise, \( \lambda \) is the rake angle; \( F_{NM} \) = Normal faulting flag: 1 for \( -120^0 \leq \lambda \leq 60^0 \) (excludes normal-oblique), 0 otherwise; \( V_{S30} \) = Time-averaged shear wave velocity (m/s) in the top 30 m of soil; \( Z_{1.0} \) = Depth (m) to shear wave velocity of 1.0 km/s; \( \Delta Z_{1.0} = Z_{1.0} \) centered on the \( V_{S30} \)-dependent average \( Z_{1.0} \) (m); \( DPP \) = Direct point parameter for directivity effect; and \( \Delta DPP = DPP \) centered on the site- and earthquake-specific average DPP (Chiou and Youngs 2014).

The GMPE of Idriss (2014) for median ground motion estimation is as follows:
\[
\ln[PSA] = a_1 + a_2 M + a_3 (8.5 - M)^2 - [\beta_1 + \beta_2 M] \ln(R_{RUP} + 10) + \xi \ln(V_s^{30})
+ \gamma R_{RUP} + \varphi F
\]

The variables in Eq. 3.20 are as follows: PSA (in g) is the 5% damped pseudoabsolute spectral acceleration; \( M \) is the moment magnitude; \( R_{RUP} \) (km) is the closest distance to the rupture surface; \( V_s^{30} \) (m/s) represents the timeaveraged shear wave velocity in the top 30 m of soil; and \( F \) represents source mechanism, with \( F=0 \) referring to strike-slip and normal mechanisms and \( F=1 \) referring to reverse and oblique mechanisms. The \( a_1, a_2, a_3, \beta_1, \beta_2, \xi, \gamma, \) and \( \varphi \) are the regression coefficients.

Atkinson and Boore (2003) and Abrahamson et al. (2016) developed GMPEs using global ground motion data set for the subduction interface and intraslab earthquakes. Zhao et al. (2006) also developed GMPEs using mostly Japanese data set for subduction interface and intraslab earthquakes. In the present study, these three sets of GMPEs were used for deep background seismicity, subduction interface and intraslab sources to estimate peak ground acceleration (PGA), spectral acceleration (SA) for a site. The logic tree weights of these GMPEs are shown in Figure 3.6 and Figure 3.10.

The GMPE of Atkinson and Boore (2003) for ground motion prediction from subduction source earthquake is as follows:

\[
\log Y = fn(M) + c_3 h + c_4 R + g \log R + c_5 s l S_C + c_6 s l S_D + c_7 s l S_E
\]

Eq. 3.21

where, \( Y = \) the peak ground acceleration (PGA) or 5% damped pseudo-spectral acceleration (PSA) in cm/s\(^2\) for random horizontal component; \( M = \) the moment magnitude; \( fn(M) = c_1 + c_2 M; h = \)
focal depth in km; \( R = \sqrt{D_{\text{fault}}^2 + \Delta^2} \) with \( D_{\text{fault}} \) being the closest distance to fault surface in km (use \( h = 100 \) km for events with depth > 100 km) and \( \Delta \) is a near-source saturation term, given by \( \Delta = 0.00724 \times 10^{0.507M} \). The \( S_C, S_D, \) and \( S_E \) are the parameters determined based on the soil classes of the National Earthquake Hazards Reduction Program (NEHRP). The \( sl \) is the parameter determined based on the rock PGA and frequency content of the seismic waves. The \( g \) is \( 10^{(1.2 - 0.18M)} \) for interface events, and \( 10^{(0.301 - 0.01M)} \) for in-slab events. \( c_1, c_2, c_3, c_4, c_5, c_6, \) and \( c_7 \) are the regression coefficients. For detail definitions of the parameters and coefficients please see Atkinson and Boore (2003).

The GMPEs of Abrahamson et al. (2016) for ground motion prediction from subduction interface and intra-slab sources are as follows:

\[
\ln S_a(\text{interface}) = \theta_1 + \theta_4 \Delta C_1 + \left( \theta_2 + \theta_3(M - 7.8) \right) \ln \left( R_{\text{rup}} + C_4 \exp \left( \theta_9(M - 6) \right) \right) \\
+ \theta_6 R_{\text{rup}} + f_{\text{mag}}(M) + f_{\text{FABA}}(R_{\text{rup}}) + f_{\text{site}}(PGA_{1100}, V_{S30}) \tag{Eq. 3.22 (a)}
\]

\[
\ln S_a(\text{slab}) = \theta_1 + \theta_4 \Delta C_1 + \left( \theta_2 + \theta_4 F_{\text{event}} + \theta_3(M - 7.8) \right) \ln \left( R_{\text{hypo}} + C_4 \exp \left( \theta_9(M - 6) \right) \right) \\
+ \theta_6 R_{\text{hypo}} + \theta_{10} F_{\text{event}} + f_{\text{mag}}(M) + f_{\text{depth}} Z_h + f_{\text{FABA}}(R_{\text{hypo}}) \\
+ f_{\text{site}}(PGA_{1100}, V_{S30}) \tag{Eq. 3.22 (b)}
\]

where, \( S_a \) = spectral acceleration in g; \( M \) = moment magnitude; \( Z_h \) = hypocentral depth (km), \( F_{\text{event}} = 0 \) for interface events and 1 for intra-slab events; \( F_{\text{FABA}} = 0 \) for forearc or unknown sites and 1 for backarc sites; \( f_{\text{mag}}(M) \) for magnitude scaling; \( f_{\text{depth}} \) for depth scaling; \( f_{\text{site}} \) for site response; and \( PGA_{1100} \) for median PGA value for \( V_{S30} = 1000 \) m/s. The \( C_1 \) is 7.8 and values of \( \Delta C_1 \) capture the epistemic uncertainty surrounding the break in magnitude scaling. All values of \( \theta \) are the regression coefficients.
The GMPE of Zhao et al. (2006) for ground motion prediction from subduction sources is as follows:

\[
\log_e(y_{i,j}) = aM_{wi} + bx_{i,j} - \log_e(r_{i,j}) + e(h - h_c)\delta h + F_R + S_I + S_S + S_{SL}\log_e(x_{i,j})
\]

\[+ c_k + \xi_{i,j} + \eta_i \quad \text{Eq. 3.23 (a)}
\]

\[
r_{i,j} = x_{i,j} + c \exp(dM_{wi}) \quad \text{Eq. 3.23 (b)}
\]

where, \( y \) is either PGA or 5% damped acceleration response spectrum (in the unit of cm/s²); \( M_w \) is the moment magnitude; \( x \) is the source distance in km; and \( h \) is the focal depth in km; \( F_R \) is the reverse fault parameter that applies only to crustal events with a reverse faulting mechanism and is zero for all other events. The \( S_I \) is the tectonic-source parameter that applies to interface events and is 0 for all other types of events and parameter \( S_S \) is for subduction slab events only and is 0 for all other types of events. Parameter \( S_{SL} \) is the magnitude-independent path modification term for slab events. Parameter \( C_k \) is the site-class term. The subscription \( i \) represents event number and \( j \) represents record number from event \( i \). The coefficient \( h_c \) is a depth term. The random variable \( \eta_i \) is the inter event error. For detail description of the parameters and coefficients please see Zhao et al. (2006).

All these GMPEs were formulated with their respective logic tree weights using EZ-FRISK software to use in the probabilistic seismic hazard analysis (PSHA).

### 3.5 Probabilistic Seismic Hazard Analysis (PSHA)

#### 3.5.1 EZ-FRISK software

The EZ-FRISK is a graphical user interface software developed by Risk Engineering, Inc., USA to perform seismic hazard analysis, spectral matching, and site response analysis (Fugro Consultants Inc. 2014). In this study, the version 8.0 of this software was used to perform probabilistic seismic hazard analysis (PSHA). For PSHA, seismic source database was prepared.
from the seismic source models of this study using this software. The appropriate ground motion prediction equations (GMPEs) with necessary parameters were defined from the database of EZ-FRISK.

The seismic hazard analysis includes: 1) defining an analysis, 2) performing seismic analysis, and 3) viewing and printing results.

For defining a probabilistic seismic hazard analysis, the inputs were organized as following:

1. Specifying analysis parameters: intensity type, amplitude unit, soil amplification, site location (multisite analysis), probabilistic seismic hazard analysis (seismic hazard deaggregation), ground motion amplitudes, uniform hazard spectra, spectral values to analysis;
2. Selecting seismic sources (background seismicity, crustal fault, and subduction source models) and ground motion prediction equations (attenuation equations);
3. Weighting seismic sources and attenuation equations using the logic tree approach;
4. Specifying attenuation equation site parameters;
5. Specifying calculation parameters.

After defining all input parameters, probabilistic seismic hazard analysis was executed. Seismic hazard results were viewed and printed completing the analysis.

3.5.2 Probabilistic Seismic Hazard Maps

The PSHA was performed for Bangladesh at a grid size of 0.25°. At each grid, the peak ground acceleration (PGA) and spectral acceleration (SA) for natural periods of 0.2, 0.5, 1.0, 2.0, 5.0, and 10.0 s at 5% damping ratio were estimated for 10% (return period of 475 years) and 2% (return
period of 2475 years) probability of exceedance in 50 years. The ground motion parameters were estimated at a reference bedrock condition where time-averaged shear wave velocity in the top 30 m \( (V_{s}^{30}) \) is 760 m/s. This reference bedrock condition is the boundary between the site classes B (rock) and C (very dense soils or soft rock) of the NEHRP (National Earthquake Hazards Reduction Program, USA).

Peak ground acceleration (PGA) maps for Bangladesh were prepared for 10% and 2% probability of exceedance in 50 years using ArcGIS software (Figure 3.11 and Figure 3.12). The spectral acceleration maps for the periods of 0.2, 0.5, 1.0, 2.0, 5.0, and 10 s were also prepared for Bangladesh for 10% and 2% probability of exceedance in 50 years (Error! Reference source not found. to Figure 3.18).
Figure 3.11 Peak ground acceleration (PGA) in g (gravitational acceleration) for 10% probability of exceedance in 50 years (475 years return period) for Bangladesh.
Figure 3.12 Peak ground acceleration (PGA) in g (gravitational acceleration) for 2% probability of exceedance in 50 years (2475 years return period) for Bangladesh.
Figure 3.13 Spectral accelerations (SA) in g (gravitational acceleration) for Bangladesh: (a) at 0.2 second (s) period for 10% probability in 50 years; (b) at 0.2s period for 2% probability in 50 years.
Figure 3.14 Spectral accelerations (SA) in g (gravitational acceleration) for Bangladesh: (a) at 0.5s period for 10% probability in 50 years; and (b) at 0.5s period for 2% probability in 50 years.
Figure 3.15 Spectral accelerations (SA) in g (gravitational acceleration) for Bangladesh: (a) at 1.0 second (s) period for 10% probability in 50 years; (b) at 1.0s period for 2% probability in 50 years.
Figure 3.16 Spectral accelerations (SA) in g (gravitational acceleration) for Bangladesh: (a) at 2.0s period for 10% probability in 50 years; and (b) at 2.0s period for 2% probability in 50 years.
Figure 3.17 Spectral accelerations (SA) in g (gravitational acceleration) for Bangladesh: (a) at 5.0 second (s) period for 10% probability in 50 years; (b) at 5.0s period for 2% probability in 50 years.
Figure 3.18 Spectral accelerations (SA) in g (gravitational acceleration) for Bangladesh: (a) at 10.0s period for 10% probability in 50 years; and (b) at 10.0s period for 2% probability in 50 years.
3.6 Discussions

The most challenging part of the seismic hazard analysis is to characterize the seismic sources. The foremost constrains to characterize the seismic sources are the uncertainties in determining the location, geometry, earthquake size, and recurrence interval of the sources. Large earthquakes ($M_w > 7$) occur at intervals of hundreds to thousands of years. However, historical records of earthquakes in these regions date back to 1548 AD. The proper identification of earthquake’s epicenter was not possible until the early twentieth century, when the instrumental recordings of earthquakes were started. Therefore, the epicenters of the historical earthquakes were identified based on the intensity of damages to the properties. The ruptures of small and deeper earthquakes do not appear to the ground surface. The large and shallow crustal earthquakes generally caused surface ruptures, but due to lack of knowledge on earthquake mechanism, the rupture of those earthquakes were not documented properly. The slip on faults (seismic sources) during prehistoric and historical earthquakes can be estimated through paleoseismological studies, such as morphotectonics, trenching.

During recent paleoseismological studies (Morino et al. 2011, 2014), it was observed that the geomorphic evidences of the past earthquakes were removed due to tropical monsoon climate in these regions. It is very challenging to determine the activity of seismic sources during prehistoric and historical earthquakes. Therefore, most of the faults (seismic sources) in Bangladesh were unidentified and uncharacterized.

The background seismicity source model was used to estimate the seismic hazard of the unidentified and uncharacterized fault sources and the earthquakes whose sources were not
definitely known. To reduce the uncertainty in hazard estimation from these sources, the logic tree weights were used for different parameters of the seismic sources (*Figure 3.6*).

Some crustal faults that have records of past earthquakes within 300 km from Bangladesh boundary were modeled to supplement the hazard of the background seismicity model. However, the slip rates of these faults were not known. Therefore, the rate of activity of these faults were determined from the geodetic strain rate using the well-accepted relationships of Youngs and Coppersmith (1985). The uncertainties in hazard calculation were accounted providing different weights in the logic tree branches of different parameters of the fault (*Figure 3.8*).

The subduction convergent plate boundaries exist close to the study area in the north and east. The position of subduction interface (megathrust) in the east is uncertain. Recent studies suggested that the megathrust would be further west from the earlier position of the megathrust (Wang et al. 2014, Steckler et al. 2016). The uncertainties in the position of the megathrust and intraslab zones were accounted setting the megathrust and intraslab boundaries at different positions and providing weights for different positions in the logic tree branches (*Figure 3.9* and *Figure 3.10*). The rate of activity in the subduction zones were determined form the geodetic strain rate, as there was no information on the slip rates of the megathrust faults from paleoseismological studies. The logic tree was used to account for the uncertainties of different parameters of the subduction zones (*Figure 3.10*).

The ground motion prediction equations (GMPEs) developed in 2014 from global data sets by the PEER NGA WEST2 were used for shallow crustal background seismicity and crustal fault models. The GMPEs developed from subduction earthquake ground motions were used for deep crustal background seismicity and subduction interface (megathrust) and intraslab models. The
uncertainties in selecting GMPEs were accounted by assigning logic weights for different GMPEs (Figure 3.6, Figure 3.8, and Figure 3.10).

The probabilistic seismic hazard analysis (PSHA) for Bangladesh was performed by using the characterized seismic sources and GMPEs to estimate peak ground acceleration (PGA) and spectral acceleration (SA) at different periods for 10% and 2% probability of exceedance in 50 years. The PGA and SA were estimated at bedrock condition ($V_{s30} = 760$ m/s) at a grid size of $0.25^0$. Then, the PGA and SA maps for Bangladesh were prepared using ArcGIS form the PGA and SA values for the periods of 0.2, 0.5, 1.0, 2.0, 5.0, and 10.0 s at each grid for 10% and 2% probability of exceedance in 50 years (Figure 3.11 to Figure 3.18). The uniform hazard spectra (UHS) for 10 %, 5% and 2% probability of exceedance in 50 years for nine major cities in Bangladesh were also prepared (Figure 3.19 to Figure 3.27).

![Figure 3.19 Uniform hazard spectra for 10%, 5%, and 2% probability of exceedance in 50 years for Dhaka City.](image-url)
Figure 3.20 Uniform hazard spectra for 10%, 5%, and 2% probability of exceedance in 50 years for Chittagong City.

Figure 3.21 Uniform hazard spectra for 10%, 5%, and 2% probability of exceedance in 50 years for Sylhet City.
Figure 3.22 Uniform hazard spectra for 10%, 5%, and 2% probability of exceedance in 50 years for Comilla City.

Figure 3.23 Uniform hazard spectra for 10%, 5%, and 2% probability of exceedance in 50 years for Mymensingh City.
Figure 3.24 Uniform hazard spectra for 10%, 5%, and 2% probability of exceedance in 50 years for Barisal City.

Figure 3.25 Uniform hazard spectra for 10%, 5%, and 2% probability of exceedance in 50 years for Rangpur City.
Figure 3.26 Uniform hazard spectra for 10%, 5%, and 2% probability of exceedance in 50 years for Rajshahi City.

Figure 3.27 Uniform hazard spectra for 10%, 5%, and 2% probability of exceedance in 50 years for Khulna City.
The relative contributions of the seismic sources (deaggregation model) to the seismic hazard (PGA) for 10% probability of exceedance in 50 years for three big cities (Dhaka, Chittagong, and Sylhet) are shown in Figure 3.28 to Figure 3.30. The deaggregation model for Dhaka city showed that the PGA will be 0.144 g for 10% probability of exceedance in 50 years for an earthquake of magnitude \( M_w 7.09 \) at a distance of 114.15 km. The deaggregation model for Chittagong city showed that the PGA will be 0.56 g for 10% probability of exceedance in 50 years for an earthquake of magnitude \( M_w 8.02 \) at a distance of 108.6 km. The deaggregation model for Sylhet city showed that the PGA will be 0.3 g for 10% probability of exceedance in 50 years for an earthquake of magnitude \( M_w 8.02 \) at a distance of 108.6 km.
Figure 3.28 Magnitude-distance-epsilon deaggregation (mean magnitude 7.09 $M_w$, mean distance 114.15 km) for Dhaka city. Peak ground acceleration (PGA) is 0.144 g for 10% probability of exceedance in 50 years.
Figure 3.29 Magnitude-distance-epsilon deaggregation (mean magnitude 8.02 $M_w$, mean distance 101.60 km) for Chittagong city. Peak ground acceleration (PGA) is 0.56 g for 10% probability of exceedance in 50 years.
Figure 3.30 Magnitude-distance-epsilon deaggregation (mean magnitude $7.5 \text{ M}_{w}$, mean distance $98.25 \text{ km}$) for Sylhet city. Peak ground acceleration (PGA) is $0.3 \text{ g}$ for $10\%$ probability of exceedance in $50 \text{ years.}$
The PGA and SA for 10% and 2% probability of exceedance in 50 years are higher in the southeastern part of the country compared to the rest of the country, as this part of the country is located close to the Ramree subduction interface and Magway intraslab sections of the eastern plate boundary, where the geodetic strain rate is high. In the future research, the boundaries of seismic sources in these parts of the plate boundary should be reevaluated and modeled using geodetic strain rate of longer period and more geological information. The seismic source models need to be updated using geological slip rates that are estimated through paleoseismological studies to update the seismic hazard maps of the present study.

3.7 Summary

The seismic sources in the study regions were characterized as background seismicity, regional seismicity, crustal fault, and subduction zone sources. The seismicity parameters (a- and b-value) of background and regional seismicity source models were estimated using a declustered and complete earthquake catalog. The crustal fault and subduction zone sources were modeled using geodetic strain rate. The GMPEs of the NGA project developed in 2014 for shallow crustal earthquakes were used for background seismicity and crustal fault sources. The GMPEs of the subduction zones were used for deep crustal and subduction sources. The ground motions (PGA and SA) were estimated at bedrock condition ($V_{s30} = 760$ m/s) for Bangladesh at grid size of $0.25^\circ$. The PGA and SA maps for different periods were prepared for Bangladesh including uniform hazard spectra for nine major cities. These seismic hazard maps and uniform hazard spectra can be used for seismic risk management in Bangladesh.
Chapter 4: Seismic Site Characterization

4.1 Background

Seismic site characterization is essential for site-specific seismic hazard analysis to accurately estimate the ground motion distribution at a site. As the dynamic properties of the geological materials can be expressed in terms of the shear wave velocity \( V_s \), the time-averaged shear wave velocity in the top 30 m \( V_s^{30} \) is used for seismic site characterization (Borcherdt 1994, BSSC 1994, 1998, 2015b, Martin and Dobry 1994, Anderson et al. 1996, UBC 1997, Park and Elrick 1998, Xia et al. 1999, Dobry et al. 2000).

The \( V_s \) can be estimated directly using downhole seismic (DS), crosshole seismic (CS) methods and indirectly using surface wave methods, such as spectral analysis of surface waves (SASW), multichannel analysis of surface waves (MASW), etc. In the DS test, seismic waves are generated on the ground surface and the travel times of the compressional waves (P-waves) and shear waves (S-waves) are measured placing the receiver at different depths within a borehole and then the velocities of the P- and S-waves are estimated using the distance between the source and receiver, and the travel times of the P- and S-waves, respectively. In the CS test, the source and receiver are placed at the same depths within two adjacent boreholes and the velocities of the P- and S-waves are estimated using the distance between the source and receiver, and the travel times of the P- and S-waves, respectively. In surface wave methods, the dispersion properties of the surface waves, generally the Rayleigh waves are used to estimate the \( V_s \) of the near-surface materials (Park and Elrick 1998, Foti et al. 2011). When the surface waves, such as the Rayleigh waves, propagate from the ground surface to the deeper layers of the earth, the phase velocities increase with...
decreasing frequencies of the surface waves. As the phase velocities of surface waves increase with decreasing frequencies, the Rayleigh waves are dispersive (Xia et al. 1999, Foti et al. 2014).

The near-surface shear velocity \( (V_s) \) estimation using the DS method is more accurate than the surface wave methods (Boore and Brown 1998). But, the DS and CS methods are more expensive than the surface wave methods. Therefore, surface wave methods, such as multichannel analysis of surface waves (MASW) are being increasingly used to estimate the \( V_s \) for seismic site characterization. In surface wave methods, the seismic waves are recorded using active and passive sources. When the seismic waves are recorded using any artificial energy sources, such as sledgehammer, it is called the active surface wave method (Mcmechan and Yedlin 1981, Park et al. 1999, Hayashi and Suzuki 2004). And when the seismic waves are recorded without artificial energy sources, it is called the passive surface wave method. In this case, seismic waves are generated due to the ambient vibration of the earth. The earth is vibrating continuously due to ocean waves, winds, cultural noises, such as traffic movements, industrial activities, etc. (Okada 2003, Hayashi et al. 2005, Park et al. 2005).

Shear wave velocity \( (V_s) \) estimation by geophysical methods needs expertise knowledge, sophisticated instruments and software for data acquisition and analysis. As a result, the \( V_s \) estimation using geophysical methods are often beyond the budget and scope of most engineering projects. Moreover, most of the techniques require spacious area, traffic and industrial noise free environment, which are often not available in densely built-up areas of a city. Therefore, the \( V_s \) prediction using standard penetration test blow counts (SPT-N) has become a popular method in geotechnical earthquake engineering. The \( V_s \) has been estimated from the SPT-N by many researchers in different parts of the world using empirical correlation between the \( V_s \) and SPT-N.

4.2 Study Area

Dhaka, the capital city of Bangladesh, is one of the world's most densely populated cities. The city covers an area of 321 sq. km having a population of more than 14 million. About 10% of Bangladesh's population which contributes 36% of the country's gross domestic product (GDP) lives in Dhaka metropolitan area (Muzzini and Aparicio 2013). Dhaka City is located close to the seismically active convergent plate boundary between the Eurasian and the Indian plates (Figure 3.2). The city is one of the highest risk cities in the world for its earthquake vulnerability (Cardona et al. 1999, CDMP 2009). The destructive and deadly hazard of an earthquake may create a real and serious threat to life, property damage, economic growth, and development of the country. If a large magnitude earthquake (magnitude more than 8.0) occurs close to a densely populated city
of a developing country, millions of casualties may happen (Bilham 2009). Dhaka City has been grown in an unplanned and haphazard way during the last 400 years. Most of the old buildings and a large number of new buildings in Dhaka City were not constructed following proper seismic design codes. The emergency management systems of the city are very weak. A recent study predicted that in Dhaka City about 270,604 buildings (83% of the total buildings) will be moderately damaged and out of them, about 238,164 buildings (which was 73% of the total buildings) will be damaged beyond repair for a scenario earthquake of 7.5 Mw occurred at a distance of about 50 km far from city center (CDMP 2009). Therefore, a proper understanding of the distribution and the level of seismic hazard is necessary for future planning, development, and safety of Dhaka City.

4.3 Geology of Dhaka City

Dhaka City is built partly on the elevated Pleistocene terrace (Madhupur Terrace) having a maximum elevation of 14 m and partly on the Holocene floodplains having a minimum elevation of 2 m. The Pleistocene terrace consists of yellowish brown medium stiff to very stiff clayey silt and medium dense to dense silty sand. The Holocene floodplains are composed of gray very soft to medium stiff clayey silt and very loose to medium dense clayey silt, silty sand (Karim and Rahman 2002, Rahman and Karim 2005, Rahman et al. 2015c). The surface geology of the city has been divided into six units: 1) Pleistocene terrace deposit (Qpty), 2) Holocene alluvial valley fill deposit (Qhav), 3) Holocene terrace deposits (Qhty), 4) Holocene alluvium (Qha), 5) Holocene channel deposit (Qhc), and 6) artificial fill (af) (Figure 4.1).
Figure 4.1 Surface geological map of Dhaka City showing the locations of downhole seismic (DS), multichannel analysis of surface waves (MASW) and small scale microtremor measurement (SSMM) surveys and standard penetration test (SPT) boreholes (Rahman et al. 2018a). Reprinted with permission of Springer Nature.
The Pleistocene terrace deposit (Qpty) is the oldest unit of the city, which is generally composed of yellowish brown to reddish brown, stiff to very stiff clayey silt, silty clay, and medium dense to dense silty sand. This unit covers the central part of the city from the north to south. The Holocene alluvial valley fill deposit (Qhav) consists of dark grey to grey, very soft to soft silty clay, clayey silt, and grey to yellowish brown, very loose to medium dense silty sand. It is deposited in the depressions or valleys of the Pleistocene terrace. The Holocene terrace deposits (Qhty) are the point bars and channel bars consisting of grey, loose to medium dense silty sand, sand and very soft to stiff clayey silt. The Holocene alluvium (Qha) is composed of grey, very soft to medium stiff silty clay, clayey silt, and very loose to loose silty sand. It covers the eastern, southeastern, southwestern, and northwestern parts of the city. The Holocene channel deposit (Qhc) consists of grey, very loose to loose silty sand and sand. It is deposited in the present river channels. The artificial fill (af) is composed of grey, very soft to soft clayey silt, very loose to loose silty sand and sand. It covers the western and eastern parts of the city. Most of the artificial filling materials were emplaced both by hydraulic dredging from the river and by truck from land. The ground was not improved during or after the artificial filling considering the earthquake hazard of the city.

The subsurface geological materials of Dhaka City are divided into six lithological units. The units are 1) Filling sand (FS), 2) Filling clay (FC), 3) Holocene sand (HS), 4) Holocene clay, 5) Plio-Pleistocene sand (PS), and 6) Plio-Pleistocene clay (PC). The subsurface lithological units that are encountered in 50 boreholes are shown in Figure 4.2.
Figure 4.2 Subsurface geological classification of 50 boreholes in Dhaka City. FS, FC, HS, HC, PS and PC stand for Filling sand, Filling clay, Holocene sand, Holocene clay, Plio-Pleistocene sand and Plio-Pleistocene clay, respectively (Rahman et al. 2018a). Reprinted with permission of Springer Nature.
4.4 Methodology

Shear wave velocity ($V_s$) and standard penetration test blow count (SPT-N) data including other geotechnical properties of the near surface geological materials of Dhaka City were collected from existing literature, Comprehensive Disaster Management Program (CDMP) and other government and non-government organizations.

Standard penetration test (SPT) were performed at 101 boreholes which are located in different surface geological units of Dhaka City. In each borehole, the SPT was conducted at each 1.5 m intervals up to a depth of 30 m. Among 101 SPT boreholes, downhole seismic (DS) test was performed at 16 boreholes at each 1 m intervals up a depth of 30 m. In some boreholes, the DS test could not be performed up to the desired depth due to filling up of the bottom part of the borehole by the suspended sediments in the borehole. The $V_s$ data, which was collected using the DS test, were used to develop empirical correlations between the $V_s$ and SPT-N. Multichannel analysis of surface waves (MASW) surveys using active and passive sources were also conducted at 50 sites. Most of the MASW survey sites were selected close to the SPT borehole sites where spacious and low traffic areas were available to compare the shear wave velocities of the DS and SPT-N with those of the MASW.

4.4.1 Downhole Seismic (DS) Method

The downhole seismic (DS) has been used for deep investigation for many years; one such example is oil and gas exploration. It has been used for the last several decades to estimate the $V_s$ of the shallow subsurface for seismic site characterization. The $V_s$ estimation using a direct method, such as downhole seismic is more reliable and accurate than the surface wave method (Boore and Brown 1998).
The Freedom Data PC of Olson Instruments for downhole seismic (DS) was used for the investigation. One borehole of 30 m depth casing by PVC pipe was used at each site to conduct the DS investigation. The borehole was used to insert the geophone for the recording of seismic waves at different depths. Polarized seismic waves were generated by hitting a wooden plank horizontally on the right and left sides, and vertically on the top using a sledgehammer (Olson Instruments 2007). A three-component geophone was lowered into the borehole to sense the seismic wave energy at different depths. Three different tests were performed at each depth for the 3 different wave polarizations. The tests were performed at each 1 m interval starting from the deepest depth to 1 m depth from the ground surface. The $V_s$ was calculated at each depth using the arrival time of the shear wave and corresponding depth (*Figure 4.3 and Figure 4.4*). Then, the time-averaged shear wave velocity in the top 30 m ($V_s^{30}$) was estimated from the shear wave velocities of 30 m depth using *Eq. 4.1* and *Eq. 4.2*.

\[
T_{30} = \sum \frac{H_i}{V_i} \quad \text{Eq. 4.1}
\]

\[
V_s^{30} = \frac{30}{T_{30}} \quad \text{Eq. 4.2}
\]

where, $H_i$: Thickness of i th layer and $30 = \sum H_i$; $V_i$: Shear wave velocity of i th layer.

### 4.4.2 Surface Wave Methods

The shear wave velocity ($V_s$) were also estimated in Dhaka City using active and passive surface wave methods, i.e., multichannel analysis of surface waves (MASW) and small scale microtremor measurement (SSMM). Artificial energy sources, such as a sledgehammer were used to generate seismic waves in active surface wave method. In passive surface wave method, the seismic waves were recorded from the ambient vibration of the earth.
4.4.2.1 Multichannel Analysis of Surface Waves (MASW)

Geode Seismic Recorder of Geometrics was used to perform the MASW survey. Twelve channel geophones were used along a 22 m long line with geophones spaced 2 m apart (Figure 4.5). An acrylic board was placed at 1 m outside from the 1st geophone, at an interval of 1 m between two adjacent geophones, and at 1 m outside from last geophone (12th geophone). Impact energy was generated by hitting on the acrylic board using a sledgehammer at 13 points along the survey line of 22 m to create seismic waves. The generated seismic waves were recorded by the geophones.

The investigation depth of this method was from 6 to 10 m, because the target frequency, geophone spacing, and survey line length were not sufficient for 30 m depth of investigation. The purpose of this method was to estimate the shear wave velocity ($V_s$) at shallower depth. The method has the benefit of controlling the seismic waves due to artificial sources. Therefore, the noises and signals can be separated easily.
Figure 4.3 Typical shear wave velocity ($V_s$) profiles of downhole seismic (DS), surface waves (MASW and SSMM), and standard penetration test blow count (SPT-N) at (a) BH-4 and (b) BH-6 in Dhaka City (Rahman et al. 2018a). Reprinted with permission of Springer Nature.
Figure 4.4 Typical shear wave velocity ($V_s$) profiles of downhole seismic (DS), surface waves (MASW and SSMM), and standard penetration test blow count (SPT-N) at BH-8 in Dhaka City (Rahman et al. 2018a). Reprinted with permission of Springer Nature.

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>BH-8</th>
<th>Lithology</th>
<th>SPT-N</th>
<th>$V_s$ (m/sec) from SPT-N (BH-8), Downhole Seismic (DS-4), and MASW &amp; SSMM (SW-37)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5</td>
<td>FS</td>
<td>Filling SAND</td>
<td>3</td>
<td>0 50 100</td>
</tr>
<tr>
<td>3.0</td>
<td>HC</td>
<td>Holocene CLAY</td>
<td>2</td>
<td>0 100 200</td>
</tr>
<tr>
<td>4.5</td>
<td>RS</td>
<td>Holocene SAND</td>
<td>3</td>
<td>300 400 500</td>
</tr>
<tr>
<td>6.0</td>
<td>HC</td>
<td>Holocene CLAY</td>
<td>9</td>
<td>600 700 800</td>
</tr>
<tr>
<td>7.5</td>
<td>HC</td>
<td>Holocene CLAY</td>
<td>11</td>
<td>1100 1200 1300</td>
</tr>
<tr>
<td>9.0</td>
<td>HC</td>
<td>Holocene CLAY</td>
<td>14</td>
<td>1400 1500 1600</td>
</tr>
<tr>
<td>10.5</td>
<td>HC</td>
<td>Holocene CLAY</td>
<td>18</td>
<td>1800 1900 2000</td>
</tr>
<tr>
<td>12.0</td>
<td>HC</td>
<td>Holocene CLAY</td>
<td>19</td>
<td>1900 2000 2100</td>
</tr>
<tr>
<td>13.5</td>
<td>HC</td>
<td>Holocene CLAY</td>
<td>26</td>
<td>2600 2700 2800</td>
</tr>
<tr>
<td>15.0</td>
<td>HC</td>
<td>Holocene CLAY</td>
<td>20</td>
<td>2000 2100 2200</td>
</tr>
<tr>
<td>16.5</td>
<td>HC</td>
<td>Holocene CLAY</td>
<td>16</td>
<td>1600 1700 1800</td>
</tr>
<tr>
<td>18.0</td>
<td>HC</td>
<td>Holocene CLAY</td>
<td>25</td>
<td>2500 2600 2700</td>
</tr>
<tr>
<td>19.5</td>
<td>HC</td>
<td>Holocene CLAY</td>
<td>27</td>
<td>2700 2800 2900</td>
</tr>
<tr>
<td>21.0</td>
<td>HC</td>
<td>Holocene CLAY</td>
<td>28</td>
<td>2800 2900 3000</td>
</tr>
<tr>
<td>22.5</td>
<td>HC</td>
<td>Holocene CLAY</td>
<td>26</td>
<td>2600 2700 2800</td>
</tr>
<tr>
<td>24.0</td>
<td>HC</td>
<td>Holocene CLAY</td>
<td>29</td>
<td>2900 3000 3100</td>
</tr>
<tr>
<td>25.5</td>
<td>HC</td>
<td>Holocene CLAY</td>
<td>34</td>
<td>3400 3500 3600</td>
</tr>
<tr>
<td>27.0</td>
<td>HC</td>
<td>Holocene CLAY</td>
<td>33</td>
<td>3300 3400 3500</td>
</tr>
<tr>
<td>28.5</td>
<td>HC</td>
<td>Holocene CLAY</td>
<td>38</td>
<td>3800 3900 4000</td>
</tr>
<tr>
<td>30.0</td>
<td>HC</td>
<td>Holocene CLAY</td>
<td>55</td>
<td>5500 5600 5700</td>
</tr>
</tbody>
</table>

$V_s$ is 240, 188, and 174 m/sec for SPT-N, DS, and SSMM & MASW methods, respectively.
SeisImager/SW software of Geometrics was used to process and analyze the MASW data. The recorded seismic waves were analyzed by common mid-point (CMP) cross-correlation method to generate phase velocity image in frequency domain from surface waves, i.e., Rayleigh waves (Hayashi and Suzuki 2004). Then, the dispersion curves of phase velocity versus frequency were generated from the phase velocity image (Figure 4.6). A one-dimensional inversion using a non-linear least square technique was applied to the dispersion curve to generate one-dimension shear wave velocity ($V_s$) structure of the near surface materials for the shallower depth (Xia et al. 1999, Hayashi and Suzuki 2004).
Figure 4.6 Dispersion curve (top) and shear wave velocity (Vs) structure (bottom) of multichannel analysis of surface waves (MASW) data (Rahman et al. 2018a). Reprinted with permission of Springer Nature.

4.4.2.2 Small Scale Microtremor Measurement (SSMM)

The SSMM is different from the MASW method for the source and frequency range, etc. In the case of the SSMM, the source was natural (microtremor). As the shear wave velocity up to a depth 30 m could not be estimated using the MASW, the small scale microtremor measurement (SSMM) was carried out at the same sites of the MASW to estimate the $V_s$ structure up to a depth
of 30 m combining the dispersion curves of the MASW and SSMM datasets to obtain the highest resolution over the entire sampled depth range.

Geode Seismic Recorder of Geometrics was also used to perform the SSMM survey. In this method, an L-shaped array of 60 m with geophones spaced 6 m apart was used for the $V_s$ estimation (Figure 4.7). Eleven geophones were used to record microtremors. Because of the L-shaped array, the resultant one-dimensional structures can be interpolated into a three-dimensional structure (Hayashi et al. 2005).

SeisImager/SW software of Geometrics was used to process and analyze the SSMM data. The Spatial Autocorrelation (SPAC) is used to determine the phase velocity from the surface wave records, such as Rayleigh waves (Aki 1957, Okada 2003, Hayashi et al. 2005). Spatial Autocorrelation function $\rho(\omega, r)$ is expressed by the Bessel function (Okada 2003). The phase velocity can be estimated at each frequency using Eq. 4.3.

$$
\rho(\omega, r) = J_0(\omega r / c(\omega))
$$

where, $r$ is the distance between receivers, $\omega$ is the angular frequency, $c(\omega)$ is the phase velocity of waves, $J_0$ is the first kind of Bessel function.

The dispersion curves of phase velocity versus frequency were generated using the surface wave dataset of the SSMM (Figure 4.7). Then, the dispersion curves of the shallower depth that were generated using the higher frequency content dataset of the MASW were combined with the dispersion curves of the deeper depth that were generated using the low frequency content dataset of the SSMM to obtain the highest resolution up to the depth 30 m. A one-dimensional inversion using a non-linear least square technique was applied to the combined dispersion curves of the
MASW and SSMM to generate one-dimension shear wave velocity \( (V_s) \) structure of the near surface materials up to a depth of 30 m to estimate the \( V_s^{30} \) (**Figure 4.7**).

**Figure 4.7** Dispersion curve (top) and shear wave velocity \( (V_s) \) structure (bottom) of both multichannel analysis of surface waves (MASW) and small scale microtremor measurement (SSMM) data (Rahman et al. 2018a). Reprinted with permission of Springer Nature.
Empirical Correlations between the $V_s$ and SPT-N

The standard penetration test (SPT) is an in-situ investigation technique, which is widely used for geotechnical site characterization. It is always desirable to estimate near-surface shear wave velocity ($V_s$) using experimental field tests. However, it is often not economically feasible to estimate the $V_s$ at all sites of an area using experimental tests. So, standard penetration test blow count (SPT-N) is generally used worldwide to predict the $V_s$ using the empirical correlation between the $V_s$ and SPT-N (Table 4.1). The SPT-N data was easily available in Dhaka City. Therefore, if appropriate empirical equations between the $V_s$ and SPT-N can be developed for Dhaka City, the $V_s$ can be predicted from the SPT-N data using the empirical correlations. In the present study, 152 data pairs of the $V_s$ and SPT-N were used to derive empirical correlations for all soils, sandy soils, and clayey soils (Figure 4.8 to Figure 4.10). The proposed correlations between the $V_s$ and uncorrected SPT-N (N) using nonlinear regression of power law model are expressed by Eq. 4.4, Eq. 4.5, and Eq. 4.6.

$$V_s = 97.3062 N^{0.3393} \, (r = 0.7496 \, \text{and} \, R^2 = 0.5618) \, \text{for all soils}$$  \hspace{1cm} \text{Eq. 4.4}

$$V_s = 82.01 N^{0.3829} \, (r = 0.6689 \, \text{and} \, R^2 = 0.4474) \, \text{for all sandy soils}$$  \hspace{1cm} \text{Eq. 4.5}

$$V_s = 100.58 N^{0.341} \, (r = 0.7304 \, \text{and} \, R^2 = 0.5334) \, \text{for all clayey soils}$$  \hspace{1cm} \text{Eq. 4.6}

The SPT-N data was corrected for overburden stress, hammer efficiency, hole diameter, rod length, and sampler liner according to Youd et al. (2001). Then, the correlations between the $V_s$ and corrected SPT-N ($(N_1)_{60}$) are expressed by Eq. 4.7, Eq. 4.8, and Eq. 4.9.

$$V_s = 129.51 (N_1)_{60}^{0.365} \, (r = 0.6944 \, \text{and} \, R^2 = 0.4822) \, \text{for all soils}$$  \hspace{1cm} \text{Eq. 4.7}
\[ V_s = 105.84 \ (N_1)^{0.4569} \ (r = 0.6525 \text{ and } R^2 = 0.4258) \] for all sandy soils\text{ Eq. 4.8}

\[ V_s = 139.33 \ (N_1)^{0.2835} \ (r = 0.6222 \text{ and } R^2 = 0.3872) \] for all clayey soils\text{ Eq. 4.9}

Most of the researchers suggested correlations between the \( V_s \) and uncorrected SPT-N (Table 4.1). In present study, it was observed that the corrections between the \( V_s \) and uncorrected SPT-N were better than correlations between the \( V_s \) and corrected SPT-N (Figure 4.8 to Figure 4.10). Therefore, in the present study, the correlations between the \( V_s \) and uncorrected SPT-N were used to predict the \( V_s \).

\textit{Table 4.1 Existing empirical correlations of different researchers between the \( V_s \) and SPT-N (Rahman et al. 2018a). Reprinted with permission of Springer Nature.}

<table>
<thead>
<tr>
<th>Authors</th>
<th>Shear wave velocity, ( V_s ) (m/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>All soils</td>
</tr>
<tr>
<td>Shibata (1970)</td>
<td>-</td>
</tr>
<tr>
<td>Fujiwara (1972)</td>
<td>( V_s = 92.1N^{0.337} )</td>
</tr>
<tr>
<td>Imai (1977)</td>
<td>( V_s = 91N^{0.337} )</td>
</tr>
<tr>
<td>Ohta and Goto (1978)</td>
<td>( V_s = 85.35N^{0.348} )</td>
</tr>
<tr>
<td>Imai and Tonouchi (1982)</td>
<td>( V_s = 97N^{0.314} )</td>
</tr>
<tr>
<td>Seed et al. (1983)</td>
<td>-</td>
</tr>
<tr>
<td>Sykora and Stokoe (1983)</td>
<td>-</td>
</tr>
<tr>
<td>Fumal and Tinsley (1985)</td>
<td>-</td>
</tr>
<tr>
<td>Athanasopoulos (1995)</td>
<td>( V_s = 107.6N^{0.36} )</td>
</tr>
<tr>
<td>Raptakis et al. (1995)</td>
<td>( V_s = 100N^{0.24} )</td>
</tr>
<tr>
<td>Kayabali (1996)</td>
<td>-</td>
</tr>
<tr>
<td>Kiku et al. (2001)</td>
<td>( V_s = 68.3N^{0.292} )</td>
</tr>
<tr>
<td>Jafari (2002)</td>
<td>-</td>
</tr>
<tr>
<td>Hasançebi and Ulusay (2007)</td>
<td>( V_s = 90N^{0.309} )</td>
</tr>
<tr>
<td>Hanumantharao and Ramana (2008)</td>
<td>( V_s = 82.6N^{0.43} )</td>
</tr>
<tr>
<td>Dikman (2009)</td>
<td>( V_s = 44N^{0.48} )</td>
</tr>
<tr>
<td>Akin et al. (2011)</td>
<td>( 38.55N^{0.176}D^{0.481} )</td>
</tr>
<tr>
<td>Kuo et al. (2011)</td>
<td>( 169.04+4.46N+0.59D )</td>
</tr>
<tr>
<td>Mhaske and Choudhury (2011)</td>
<td>( V_s = 72N^{0.4} )</td>
</tr>
</tbody>
</table>
Figure 4.8 Empirical correlations of the present study along with the existing correlations between the $V_s$ and SPT-N for all soils of Dhaka City (Rahman et al. 2018a). Reprinted with permission of Springer Nature.

Figure 4.9 Empirical correlations of the present study along with the existing correlations between the $V_s$ and SPT-N for all sandy soils of Dhaka City (Rahman et al. 2018a). Reprinted with permission of Springer Nature.
The time-averaged shear wave velocity in the top 30 m ($V_{s30}$) was estimated using the DS method and correlation equations of all soils, sandy soils, and clayey soils at the same borehole sites. Then, the $V_{s30}$ estimated using these techniques were verified to select the correlating equations that predicted the $V_{s30}$ accurately (Figure 4.11). It was observed that most of the data was close to the equivalent line and the correlations of all soils, sandy soils, and clayey soils predict the $V_{s30}$ more or less similar. Hasançebi and Ulusay (2007) mentioned that the SPT-N is a significant parameter in correlations between the $V_s$ and SPT-N while the type of soil has no important influence. The use of equation for all soils based on the uncorrected SPT-N is appropriate for the indirect prediction of the $V_s$ (Hasançebi and Ulusay 2007). Although the use of the SPT in clayey soils is not appropriate, many researchers used the SPT-N of clayey soils to predict the $V_s$ (e.g. Dikmen, 2009; Hasançebi and Ulusay, 2007; Imai, 1977; Lee, 1990). Therefore, the correlating
equations of all soils were used in the present study to predict the $V_{s}^{30}$ for avoiding complexity of defining soil types.

### 4.5 Results

The time-averaged shear wave velocity in the top 30 m ($V_{s}^{30}$) of the soil was estimated at 151 sites in Dhaka City using the DS, surface waves (MASW and SSMM), and SPT-N. The results of the $V_{s}^{30}$ are shown as a point map on the surface geology map of Dhaka City to show the spatial distribution of the $V_{s}^{30}$ in the city (Figure 4.12). The $V_{s}^{30}$ results of all methods vary from 127 m/sec to 420 m/sec. The lowest value of the $V_{s}^{30}$ was estimated at the surface wave measurement site of SW-4 that was located on the Alluvium in the eastern part of the city. The highest value of the $V_{s}^{30}$ was estimated at the borehole site of BH-7 that was situated on the Pleistocene terrace deposit in the western part of the city.

![Figure 4.11 Verification of the empirical equations of all soils, sandy soils and clayey soils to select the equation for the prediction of the $V_{s}^{30}$ using the SPT-N (Rahman et al. 2018a). Reprinted with permission of Springer Nature.](image-url)
In several sites, the $V_s^{30}$ was estimated using all methods (DS, surface waves and SPT-N) to observe the variation of the $V_s^{30}$ results at the same site due to different methods of estimation. It was observed that the $V_s^{30}$ results of different methods at the same site were not equal and the variation of the $V_s^{30}$ results are random. Therefore, the $V_s^{30}$ results of different methods need to be standardized to prepare a $V_s^{30}$ map for Dhaka City. In the present study, the relationship between the $V_s^{30}$ and Holocene soil thickness was used to consistently predict the $V_s^{30}$ from the Holocene soil thickness at different areas of the city.
Figure 4.12 Map showing time-averaged shear wave velocity in the top 30 m ($V_s^{30}$) that is estimated at different sites of Dhaka City using downhole seismic (DS), multichannel analysis of surface waves (MASW), small scale microtremor measurement (SSMM) methods and correlation between the shear wave velocity ($V_s$) and SPT-N (Rahman et al. 2018a). Reprinted with permission of Springer Nature.
4.6 Relationship between $V_s^{30}$ and Holocene Soil Thickness

Dhaka City is covered partly by the Pleistocene clayey soils, and partly by the Holocene clayey and sandy soils (Figure 4.1). The Pleistocene clayey soils are exposed and present in the central part of the city and the Holocene clayey and sandy soils are overlaying on the Pleistocene and Plio-Pleistocene clayey and sandy soils and present in the eastern, southeastern, southwestern and northwestern parts of the city. Therefore, the Holocene soil thickness in the city varies from 0 m to more than 30 m (Figure 4.2). As there is a good relationship between the $V_s^{30}$ and Holocene soil thickness, a model was developed for Dhaka City using the $V_s^{30}$ of all methods and Holocene soil thickness to predict the $V_s^{30}$ based on the Holocene soil thickness (Figure 4.13). During model development, more importance was given to the $V_s^{30}$ of the DS method, because the $V_s^{30}$ estimation using the DS method is more accurate than any other method of shear wave velocity estimation. The $V_s^{30}$ values of the model range from 260 m/sec to 145 m/sec. The $V_s^{30}$ is 260 m/sec in the Pleistocene clayey soils and 145 m/sec in the Holocene clayey and sandy soils where the Holocene soil thickness is 30 m.

4.7 Preparation of $V_s^{30}$ Map

The $V_s^{30}$ map is an important component for seismic microzonation. Matsuoka et al. (2006) were prepared a $V_s^{30}$ map of Japan from an engineering geomorphic map. Kanli et al. (2006), Anbazhagan and Sitharam (2008), Mhaske and Choudhury (2011), Eker et al. (2012), Kuo et al. (2012), Naik et al. (2014) and others were also prepared the $V_s^{30}$ maps for seismic hazard assessment. Most of the researchers used different interpolation techniques to prepare the $V_s^{30}$ map using the $V_s^{30}$ data of different sites. If the density of the $V_s^{30}$ data is low and the variation of the geological materials is high in an area, it is not possible to prepare an accurate $V_s^{30}$ map using
interpolation techniques. Furthermore, as the investigation sites of the $V_s$ estimation were selected based on the surface geological units, the $V_{s30}$ data was not uniformity distributed in the city and the numbers of the $V_{s30}$ investigation sites were not sufficient enough to prepare a $V_{s30}$ map for Dhaka City using an interpolation technique to accurately predict the $V_{s30}$. Therefore, in the present study, the $V_{s30}$ map was prepared from the $V_{s30}$ data that was predicted using the relationship between the $V_{s30}$ and Holocene soil thickness.

![Figure 4.13 Model for the prediction of the $V_{s30}$ using the Holocene soil thickness for Dhaka City (Rahman et al. 2018a). Reprinted with permission of Springer Nature.](image)

Figure 4.13 Model for the prediction of the $V_{s30}$ using the Holocene soil thickness for Dhaka City (Rahman et al. 2018a). Reprinted with permission of Springer Nature.
The digital elevation model (DEM) of 30 m resolution from the Shuttle Radar Topographic Mission (SRTM) was used to determine the ground surface elevation at each 30 m grid of Dhaka City. The elevation at the base of the Holocene soils was predicted at each 30 m grid of the city from the Holocene soil thicknesses that were encountered in 101 boreholes using inverse distance weighting (IDW) interpolation technique. Then, the Holocene soil thickness at each 30 m grid of the city was predicted by subtracting the elevation of the base of the Holocene soils from the ground surface elevation. A Holocene soil thickness map was prepared using the Holocene soil thickness of each 30 m grid using inverse distance weighting (IDW) interpolation technique (Figure 4.14). The $V_s^{30}$ at each 30 m grid of the city was predicted from the Holocene soil thickness of each 30 m grid using the model that was prepared using the relationship between the $V_s^{30}$ and Holocene soil thickness (Figure 4.13). Then, the $V_s^{30}$ map was prepared from the $V_s^{30}$ of each 30 m grid using inverse distance weighting (IDW) interpolation technique (Figure 4.15). All maps of the present study were prepared using ArcGIS software.

The point map of the $V_s^{30}$ (Figure 4.12) was overlaid on the $V_s^{30}$ map (Figure 4.15). It was observed that most of the $V_s^{30}$ values of the point map that were estimated using the DS, surface waves (MASW and SSMM) and SPT-N data was equal to the $V_s^{30}$ values of the $V_s^{30}$ map that were predicted using the Holocene soil thickness. Some $V_s^{30}$ values of the point map that were estimated using SPT-N data were higher than that of the $V_s^{30}$ map and few $V_s^{30}$ values of the point map that were estimated using surface wave data were lower than that of the $V_s^{30}$ map. It verifies that the $V_s^{30}$ prediction using the relationship between the $V_s^{30}$ and Holocene soil thickness is accurate.
Figure 4.14 The Holocene soil thickness map of Dhaka City (Rahman et al. 2018a). Reprinted with permission of Springer Nature.
Figure 4.15 The $V_{s}^{30}$ map of Dhaka City (Rahman et al. 2018a). Reprinted with permission of Springer Nature.
4.8 Discussions

The near surface shear wave velocity ($V_s$) profiles that were estimated at the same sites using the DS, surface waves (MASW and SSMM) methods, and correlation between the $V_s$ and SPT-N, were plotted together to observe the variation of the $V_s$ estimation of different methods (Figure 4.3 and Figure 4.4). It was noticed that the $V_s$ profiles of different methods were almost similar except some random variations at some parts of the $V_s$ profiles. These variations of the $V_s$ profiles have affected the overall $V_s^{30}$ of each method. Due to variation of the $V_s^{30}$ of different methods, same soil type may be classified as different site class. The variations of the $V_s$ profiles might be due to the measurement error of each method. Therefore, it should be very careful during estimation of the $V_s$ using only one method. Two or more methods of the $V_s$ estimation can be used to accurately estimate the $V_s$ of an area.

The empirical correlations between the $V_s$ and SPT-N for all soils, sandy soils and clayey soils of Dhaka City were compared with the existing correlations of several researchers (Figure 4.8 to Figure 4.10). It is observed that the correlation of the present study for all soils have a good match with the existing correlations of Fujiwara (1972), Imai (1977), Ohta and Goto (1978), Imai and Tonouchi (1982), Mhaske and Choudhury (2011), and Naik et al. (2014). However, the empirical correlations of Kiku et al. (2001) and Dikmen (2009) predicted lower $V_s^{30}$ than that of all soils. On the other hand, the empirical correlations of Athanasopoulos (1995) and Hanumantharao and Ramana (2008) predicted higher $V_s^{30}$ than that of all soils. Similarly, the correlations for sandy soils of the present study have a good match with the existing correlations of several researchers, but the correlation for clayey soils of the study area predicts higher $V_s$ than the existing correlations of different researchers (Figure 4.8 to Figure 4.10). This variation might be due to a small number of $V_s$ and SPT-N pairs to develop the correlation of clayey soil for the present study. The $V_s$ and SPT-
N estimation methods might also be responsible for these variations. However, these correlation equations of the present study can be used for the $V_s$ prediction in the areas where the $V_s$ estimation using direct methods is limited. These equations can also be used in the areas where the $V_s$ estimation instruments are not available and in the areas of low cost projects.

The variation of the $V_s$ results that were estimated at the same sites of a surface geological unit using different methods, were random (Figure 4.11). Therefore, the $V_s$ results of different methods should be identical at the same site to prepare a $V_{s30}$ map using these $V_{s30}$ results. Several researchers stated that there is a good relationship between the $V_{s30}$ and geological units (e.g. Wills and Silva 1998; Holzer et al. 2005). Therefore, the relationship between the $V_{s30}$ and Holocene soil thickness was used in the present study to uniform the $V_{s30}$ results and to predict the $V_{s30}$ from the Holocene soil thickness (Figure 4.13). Then, the $V_{s30}$ map was prepared from the $V_{s30}$ that were predicted using the relationship between the $V_{s30}$ and Holocene soil thickness. There is a good match between the predicted $V_{s30}$ and estimated $V_{s30}$ (Figure 4.15). Therefore, the $V_{s30}$ can be predicted easily from the relationship between the $V_{s30}$ and Holocene soil thickness in any area of Dhaka City to estimate the amplification factor of seismic waves for seismic design of structures.

In Bangladesh, there was no seismic site class characterization system based on the $V_{s30}$. Therefore, the geological materials of the Dhaka City were classified based on the $V_{s30}$ as site classes D (stiff soils) and E (soft soils) according to the NEHRP (National Earthquake Hazards Reduction Program, USA) and as site classes C and D according to the Eurocode 8 (Table 4.2 and Figure 4.16). According to NEHRP, the $V_{s30}$ is from 180 m/sec to 360 m/sec for site class D and below 180 m/sec for site class E. According to the Eurocode 8, the $V_{s30}$ is from 180 m/sec to 360 m/sec for subsoil class C and below 180 m/sec for subsoil class D.
<table>
<thead>
<tr>
<th>Site class or Soil profile type</th>
<th>Description</th>
<th>NEHRP, USA</th>
<th>Eurocode 8</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Hard rock</td>
<td>&gt; 1500</td>
<td>-</td>
</tr>
<tr>
<td>B</td>
<td>Rock</td>
<td>760 - 1500</td>
<td>A</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Rock or other rock-like geological formation, including at most 5m of weaker material at the surface</td>
</tr>
<tr>
<td>C</td>
<td>Very dense soil/soft rock</td>
<td>360 - 760</td>
<td>B</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Deposits of very dense sand, gravel or very stiff clay, at least several tens of m in thickness, characterized by a gradual increase of mechanical properties with depth</td>
</tr>
<tr>
<td>D</td>
<td>Stiff soil</td>
<td>180 - 360</td>
<td>C</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of m</td>
</tr>
<tr>
<td>E</td>
<td>Soft soil</td>
<td>&lt; 180</td>
<td>D</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of cohesive soil predominantly soft-to-firm</td>
</tr>
<tr>
<td>F</td>
<td>Special soils requiring site-specific evaluation (1. Soils vulnerable to potential failure or collapse under seismic loading, e.g., liquefiable soils, quick and highly sensitive clays, collapsible weakly cemented soils; 2. Peats and/or highly organic clays (3 m or thicker layer); 3. Very highly plasticity clays (8 m or thicker layer with plasticity index &gt; 75); 4. Very thick soft/medium stiff clays (36 m or thicker layer))</td>
<td></td>
<td>E</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>A soil profile consisting of a surface alluvium layer with $V_{s}^{30}$ values of class C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with $V_{s}^{30}$ &gt; 800 m/sec</td>
</tr>
<tr>
<td>S1</td>
<td>Deposits consisting- or containing a layer at least 10 m thick- of soft clays/silts with high plasticity index (PI &gt; 40) and high water content</td>
<td></td>
<td>-</td>
</tr>
<tr>
<td>S2</td>
<td>Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in classes A-E or S1</td>
<td></td>
<td>-</td>
</tr>
</tbody>
</table>
Figure 4.16 Site class map of Dhaka City based on the $V_{s30}$ according to the National Earthquake Hazards Reduction Program (NEHRP), USA and Eurocode 8 (EC 8) (Rahman et al. 2018a). Reprinted with permission of Springer Nature.
4.9 Summary

In the present study, the shear wave velocity of the near-surface materials ($V_s$) was estimated using the downhole seismic (DS), multichannel analysis of surface waves (MASW), small scale microtremor measurement (SSMM), and standard penetration test blow count (SPT-N) data. The $V_s$ was also estimated using the relationship between the $V_s^{30}$ and Holocene soil thickness. Then, the $V_s^{30}$ map was prepared from the $V_s^{30}$ results that was predicted using the relationship between the $V_s^{30}$ of different methods and Holocene soil thickness. It is observed that the estimated and predicted $V_s^{30}$ results have a good match. As the $V_s$ estimations using direct methods, such as downhole seismic, surface waves are costly and need expertise knowledge and sophisticated software, the $V_s^{30}$ can easily be predicted using the correlation between the $V_s$ and SPT-N. The $V_s^{30}$ can also be predicted using the relationship between the $V_s^{30}$ and the Holocene soil thickness. Therefore, the correlation between the $V_s$ and SPT-N, and relationship between the $V_s^{30}$ and Holocene soil thickness can be used for the $V_s^{30}$ estimation in Dhaka City where most of the organizations and companies have no geophysical instruments and expertise personnel to conduct geophysical surveys for near-surface shear wave velocity estimations.

The $V_s^{30}$ map of the present study will be used to estimate the amplification factors of the seismic waves for seismic hazard analysis of Dhaka City. As the city is growing fast and expanding on the Alluvium in the western and eastern parts of the city, the city authority needs to consider the $V_s^{30}$ map for future urban development of the city.
Chapter 5: Site Response Analysis

5.1 Background

The frequency content of seismic waves depends on the source, path, and site condition. Therefore, the seismic waves that travel from source to site are amplified or de-amplified due to the variation in the frequency content of the seismic waves within the near-surface soils of the site. The site effects of the near-surface soils during the 1964 Alaska, 1964 Niigata, 1985 Mexico, 1989 Loma Prieta earthquakes have been well documented. Ground failure, liquefaction, structural damage, etc. have occurred during these earthquakes due to site effects. Therefore, site-specific ground response analysis is an important component of seismic hazard analysis to estimate site effects of the near-surface soils.

The earthquake ground motion at a site is generally predicted at bedrock condition by using ground motion prediction equations (GMPEs) as a function of earthquake magnitude and distance from the sources (Abrahamson et al. 2014, Boore et al. 2014, Campbell and Bozorgnia 2014, Chiou and Youngs 2014, Idriss 2014). The ground condition where the shear wave velocity ($V_s$) is equal to or greater than 760 m/s is generally considered as bedrock ground condition. As a simplified procedure, in the last three decades, a common practice for ground motion prediction at the ground surface is simply multiplying the bedrock ground motion by the site amplification factor (site coefficient) that is determined using the time-averaged shear wave velocity in the top 30 m ($V_{s30}$) at the site (Borcherdt 1994, BSSC 1994, 2015). However, the state-of-practice is to perform site response analysis for the prediction of surface ground motion using the dynamic properties of the different soil types above the bedrock and propagating the bedrock ground motion at the base of
the soil profile (Cramer 2003, Bazzurro and Cornell 2004a, 2004b, Kaklamanos et al. 2015). It is observed that the ground motion prediction at the ground surface using one-dimensional site response analysis improves the accuracy of the surface ground motion (Groholski et al. 2016, Stewart et al. 2017).

One-dimensional site response analysis includes linear, equivalent-linear, and non-linear models. Linear model represents strain-constant shear modulus of soils at small-strain and constant damping ratio. The nonlinear properties of soils are approximated by using equivalent-linear model where the shear modulus and damping ratio are iteratively adapted to be consistent with the effective level of shear strain in each soil type and the converged values are then used in the site response estimation (Kaklamanos et al. 2015). The linear and equivalent-linear response analyses can be performed in the frequency domain as well as in the time domain. In the nonlinear analysis, the shear modulus and damping ratio change throughout the duration of loading and the analysis is performed in the time domain only.

SHAKE (Schnabel et al. 1972), SHAKE91 (Idriss and Sun 1992), and SHAKE2000 (Ordóñez 2010) are the most widely used computer programs to perform one-dimensional equivalent-linear site response analysis. Frequency domain equivalent-linear site response models are widely used to estimate site effects due to their robustness, simplicity, flexibility, and low computational requirement, but these models have some limitations (Hashash et al. 2010). It is observed that equivalent-linear model becomes inaccurate at shear strains of approximately 0.1-0.4% (Kim and Hashash 2013, Yee et al. 2013, Kaklamanos et al. 2015). Therefore, non-linear site response analysis is required to accurately estimate the surface ground motion at high strains during strong ground motions. Several programs, such as DEEPSOIL (Hashash et al. 2017), SUMDES (Li et al.
OpenSees (Silvia et al. 2006) are available to perform non-linear site response analysis. However, the usage of these programs in the standard engineering practice is limited (Kaklamanos et al. 2015). The present study aimed at evaluating the site effects of the thick and soft sedimentary deposits in Dhaka City using the $V_s^{30}$-based site coefficients and one-dimensional linear, equivalent-linear, and nonlinear site response analyses.

### 5.2 Mechanical and Dynamic Properties of the Near-surface Soils

The geological materials of Dhaka City is broadly divided into two classes based on geologic age (Alam et al. 1990), i.e., the Holocene and Pleistocene deposits (Figure 5.1). The Pleistocene silty clay, clayey silt, and silty sand are present in the elevated Pleistocene terrace, which is called Madhupur Terrace. The Holocene sand, silt, and clay are deposited in the low-lying floodplains of the Buriganga River in the south and southwest, Turag River in the west, and Balu River in the east. In the city, the maximum elevation is 14 m on the Pleistocene terrace and the minimum elevation is 2 m on the Holocene floodplains (Rahman et al. 2018a).

The Holocene clayey silt dominated sediments exist in the eastern and northwestern parts of the city area, where the Pleistocene silty sands are present at a depth of 10 - 20 m. The silty sand dominated sediments exist in the western and southwestern parts, where the Pleistocene sands are present at a depth of more than 30 m. The Pleistocene silty clay and clayey silt exist in the central part with valleys filled with the Holocene silty sand and clayey silt. For urban development, the valleys of the Pleistocene terraces in the central part and the Holocene floodplains in the eastern, northwestern, western, and southwestern parts have been artificially filled with about 3 - 6 m thick clayey silt, silty sand, and sand by mechanical dredging from nearby rivers.
On different surface geological units of Dhaka City, ten geotechnical boreholes were drilled down to a depth of 30 m (Figure 5.1). At each borehole, the standard penetration test (SPT) was performed at an interval of 1.5 m and undisturbed and disturbed samples were collected to perform soil classification and laboratory tests, such as, grain size analysis, density, Atterberg limits, direct shear, consolidation.

Figure 5.1 Surface geological map of Dhaka City showing geotechnical borehole locations and ground response analysis sites (Rahman et al. 2015c). Don’t require permission of Elsevier as the author of this article.
5.2.1 Shear Wave Velocity ($V_s$)

At boreholes from BH-04 to BH-10, downhole seismic test was conducted at an interval of 1 m down to a depth of 30 m to estimate shear wave velocity ($V_s$) (CDMP 2009). At boreholes from BH-01 to BH-03, the $V_s$ has been estimated down to a depth of 30 m using the correlation between the $V_s$ and standard penetration test blow counts (SPT-N) (Rahman et al. 2018a).

The site coefficients of the National Earthquake Hazards Reduction Program (NEHRP) are based on the time-averaged shear wave velocity in the top 30 m ($V_{s}^{30}$) of the soil profile (BSSC 1998). The coefficients do not account for the site effect of the deeper soft soils, where the soil and bedrock boundary ($V_{s}^{30} \geq 760$ m/s) exists deeper than 30 m (Park and Hashash 2005). In the study area, this boundary exists down to a depth of approximately 195-328 m within the Pliocene sediments that are predominantly composed of sand with some clay layers and it is not a geological boundary. The boundary does not exhibit high impedance contrast and the sediments above and below the boundary are not consolidated. Therefore, the soil and bedrock boundary in the study area is gradational. There is no available shear wave velocity ($V_s$) data below 200 m depth for the study area. Using the technique of the receiver functions, Singh et al. (2016) predicted that the thickness of sedimentary rocks is 16 km in Dhaka, which is located at the center of the Bengal Basin. The high impedance contrast boundary exists at more than 4 km depth at the boundary between the younger syn-Himalayan clastics ($V_s = 1.3$ to $1.5$ km) and the Eocene Limestone or older strata ($V_s = 2.6$ km) (Singh et al. 2016).

In Dhaka, the sediments of the Holocene floodplains and the Pleistocene terrace are underlain by the uniform Pliocene sand with some clay layers from depths 30 m to more than 328 m. The Pliocene sand is the main aquifer of Dhaka City for groundwater extraction. Several lithological
logs of deep boreholes were available, which were drilled for groundwater exploration. These boreholes were not located at the same sites of the 10 geotechnical boreholes. As the Pliocene sandy sediments uniformly underlie the Holocene floodplains and Pleistocene terrace deposits, the boreholes of groundwater exploration that were located close to the geotechnical boreholes were used to classify the soils of the geotechnical borehole sites from depths 30 m to 328 m. All soils were classified using the Unified Soil Classification System (USCS). The types of soils of the study area that were encountered down to a depth 328 m are shown in Table 5.1.

<table>
<thead>
<tr>
<th>Geological age</th>
<th>Type of Soils</th>
<th>Classification (USCS)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Recent Filling</td>
<td>Low plastic inorganic CLAY</td>
<td>CL</td>
</tr>
<tr>
<td></td>
<td>High plastic inorganic CLAY</td>
<td>CH</td>
</tr>
<tr>
<td></td>
<td>Low plastic inorganic SILT</td>
<td>ML</td>
</tr>
<tr>
<td></td>
<td>High plastic inorganic SILT</td>
<td>MH</td>
</tr>
<tr>
<td></td>
<td>Silty SAND</td>
<td>SM</td>
</tr>
<tr>
<td></td>
<td>Poorly graded SAND</td>
<td>SP</td>
</tr>
<tr>
<td>Holocene</td>
<td>Organic SILT</td>
<td>OL</td>
</tr>
<tr>
<td></td>
<td>Low plastic inorganic CLAY</td>
<td>CL</td>
</tr>
<tr>
<td></td>
<td>High plastic inorganic CLAY</td>
<td>CH</td>
</tr>
<tr>
<td></td>
<td>Low plastic inorganic SILT</td>
<td>ML</td>
</tr>
<tr>
<td></td>
<td>High plastic inorganic SILT</td>
<td>MH</td>
</tr>
<tr>
<td></td>
<td>Silty SAND</td>
<td>SM</td>
</tr>
<tr>
<td></td>
<td>Poorly graded SAND</td>
<td>SP</td>
</tr>
<tr>
<td>Plio-Pleistocene</td>
<td>Low plastic inorganic CLAY</td>
<td>CL</td>
</tr>
<tr>
<td></td>
<td>High plastic inorganic CLAY</td>
<td>CH</td>
</tr>
<tr>
<td></td>
<td>Low plastic inorganic SILT</td>
<td>ML</td>
</tr>
<tr>
<td></td>
<td>High plastic inorganic SILT</td>
<td>MH</td>
</tr>
<tr>
<td></td>
<td>Silty SAND</td>
<td>SM</td>
</tr>
<tr>
<td></td>
<td>Poorly graded SAND</td>
<td>SP</td>
</tr>
</tbody>
</table>

The array microtremor measurements (AMT) were conducted at 10 sites in Dhaka City to estimate shear wave velocity ($V_s$) of the deeper subsurface (CDMP 2009). The AMT sites were
located close to the geotechnical boreholes sites. Therefore, the $V_s$ for the depths from 30 m to 200 m were used from the AMT data. The $V_s$ for the depths from 200 m to 328 m were interpolated based on the lithological characteristics of the borehole log. Two typical soil profiles with the $V_s$ are shown in Figure 5.2 and Figure 5.3. The sites of the ten boreholes were divided into four groups based on the surface and subsurface lithological characteristics. The sites are on a) the Pleistocene terrace (BH-01, BH-02, and BH-07), b) artificial filling and alluvium (3 - 10 m thick) in the valleys of the Pleistocene terrace (BH-04, BH-05, and BH-09), c) artificial filling and the alluvial floodplain deposits (8 – 19 m thick) that are underlain by the Pleistocene deposits (BH-03, BH-06, and BH-10); and d) artificial filling and the alluvial floodplain deposits (> 30 m thick) that are underlain by the Plio-Pleistocene deposits (BH-08) (Figure 5.1). The shear wave velocity ($V_s$) profiles of the four groups are shown in Figure 5.4.
Figure 5.2 Typical soil profiles at Pleistocene terrace in Dhaka City.
Figure 5.3 Typical soil profiles at Pleistocene terrace, and (b) Holocene floodplain in Dhaka City.
Figure 5.4 Shear wave velocity ($V_s$) profiles at 10 borehole sites (a) BH-01, BH-02, and BH-07, (b) BH-04, BH-05, and BH-09, (c) BH-03, BH-06, and BH-10, and (d) BH-08 (CDMP 2009).
5.2.2 Modulus Reduction and Material Damping Curves

As material specific test results were not available for the study area, modulus reduction (MR) and material damping (D) curves were used from literature. In this study, the normalized modulus reduction and material damping curves proposed by Darendeli (2001) were used to approximate the nonlinear behavior of the soft sedimentary deposits of Dhaka City (Figure 5.5).

![Normalized Modulus Reduction Curves](image1)

![Material Damping Curves](image2)

![Shear Strength Curves](image3)

*Figure 5.5 Example of normalized modulus reduction, material damping, and shear strength curves with target strength (effective stress state with water table at ground surface). The reference curves are from Darendeli (2001). The curves are for clay at a depth of 11.3 m, where lateral earth pressure, plasticity index, overconsolidation ratio, loading frequency, and number of loading cycles are 0.83, 30, 2, 1, and 10, respectively.*
5.3 Probabilistic Seismic Hazard Analysis

The probabilistic seismic hazard analysis (PSHA) has been performed following standard procedure using the EZ-FRISK computer program (Fugro Consultants Inc. 2014) at 10 different sites in Dhaka City to create the target response spectra (uniform hazard spectra) at the reference bedrock condition. The PSHA has been executed to prepare acceleration response spectra at the bedrock condition using the seismic sources and the ground motion prediction equations (GMPEs). The seismic sources of the study region have been characterized as background seismicity (gridded), regional seismicity, crustal fault, subduction interface and intra-slab seismic sources (please see Chapter 3). (Rahman et al. 2018b) (Abrahamson et al. 2014, Boore et al. 2014, Campbell and Bozorgnia 2014, Chiou and Youngs 2014, Idriss 2014). The GMPEs for subduction interface and intra-slab seismic sources were used from Atkinson and Boore (2003), Zhao et al. (2006), and Abrahamson et al. (2016). The uncertainties in characterizing the seismic sources and assigning the weights for the GMPEs were accounted using logic trees that have been described in Chapter 3.

The peak ground acceleration (PGA) and spectral accelerations (SA) at various natural periods from 0.01 to 10 s were estimated at bedrock condition for 10%, 5%, and 2% probability of
exceedance in 50 years at 10 different sites in Dhaka City (Figure 5.1). For each site, the uniform hazard spectra at bedrock condition were prepared for 10\%, 5\%, and 2\% probability of exceedance in 50 years (Figure 5.6 to Figure 5.8). These uniform hazard spectra (response spectra) were used as target response spectra for spectral matching.

![Figure 5.6 Uniform hazard spectra (UHS) for bedrock condition ($V_{s30} = 760$ m/s) and for ground surface using the $V_{s30}$-based site coefficients of the NEHRP for 10 % probability of exceedance in 50 years at 10 different borehole sites in Dhaka City.](image)
Figure 5.7 Uniform hazard spectra (UHS) for bedrock condition ($V_{s30} = 760$ m/s) and for ground surface using the $V_{s30}$-based site coefficients of the NEHRP for 5% probability of exceedance in 50 years at 10 different borehole sites in Dhaka City.

Figure 5.8 Uniform hazard spectra (UHS) for bedrock condition ($V_{s30} = 760$ m/s) and for ground surface using the $V_{s30}$-based site coefficients of the NEHRP for 2% probability of exceedance in 50 years at 10 different borehole sites in Dhaka City.
5.4 Spectral Matching

Spectral matching has been performed using the EZ-FRISK to modify the acceleration time history of real earthquake by matching its response spectrum with the target response spectrum (bedrock) that was generated using the PSHA for 10% and 2% probability of exceedance in 50 years (Figure 5.9 to Figure 5.12). Twenty-eight horizontal acceleration time histories of nine earthquakes, whose response spectra were similar to the target response spectra of 10% and 2% probability of exceedance in 50 years, were downloaded from the PEER NGA WEST2 database (Ancheta et al. 2103). The details of the acceleration time histories are given in Table 5.2 and Table 5.3.

![Spectral matching of the response spectrum of the time history with target response spectra for 10% probability of exceedance in 50 years. Initial is the response spectrum of the time history and step 4 is the matched response spectrum.](image)

Figure 5.9 Spectral matching of the response spectrum of the time history with target response spectra for 10% probability of exceedance in 50 years. Initial is the response spectrum of the time history and step 4 is the matched response spectrum.
Figure 5.10 Spectral matching of the response spectrum of the time history with target response spectra for 2% probability of exceedance in 50 years. Initial is the response spectrum of the time history and step 4 is the matched response spectrum.
Figure 5.11 Initial time history (solid blue) from the PEER NGA WEST2 database and match time history (dashed brown line) for 10% probability of exceedance in 50 years.

Figure 5.12 Initial time history (solid green line) from the PEER NGA WEST2 database and match time history (dashed orange line) for 2% probability of exceedance in 50 years.
Table 5.2 The response spectra of the time histories of the following earthquakes are used for spectral matching with the target response spectra for 10% probability of exceedance in 50 years.

<table>
<thead>
<tr>
<th>Earthquake Name</th>
<th>Year</th>
<th>Station Name</th>
<th>Magnitude</th>
<th>Mechanism</th>
<th>$R_{JB}$ (km)</th>
<th>$R_{RUP}$ (km)</th>
<th>$V_{s30}$ (m/s)</th>
<th>Horizontal-1 Acc. Filename</th>
<th>Horizontal-2 Acc. Filename</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loma Prieta, USA</td>
<td>1989</td>
<td>Piedmont Jr High School Grounds</td>
<td>6.93</td>
<td>Reverse oblique</td>
<td>72.9</td>
<td>73.0</td>
<td>895.36</td>
<td>RSN788_LOMAP_PJH045.AT2</td>
<td>RSN788_LOMAP_PJH315.AT2</td>
</tr>
<tr>
<td>Loma Prieta, USA</td>
<td>1989</td>
<td>SF - Rincon Hill</td>
<td>6.93</td>
<td>Reverse oblique</td>
<td>74.04</td>
<td>74.1</td>
<td>873.10</td>
<td>RSN797_LOMAP_RIN000.AT2</td>
<td>RSN797_LOMAP_RIN090.AT2</td>
</tr>
<tr>
<td>Chi-Chi, Taiwan</td>
<td>1999</td>
<td>HWA002</td>
<td>7.62</td>
<td>Reverse oblique</td>
<td>53.3</td>
<td>56.9</td>
<td>789.18</td>
<td>RSN1256_CHICHI_HWA002-N.AT2</td>
<td>RSN1256_CHICHI_HWA002-W.AT2</td>
</tr>
<tr>
<td>Chi-Chi, Taiwan</td>
<td>1999</td>
<td>TAP046</td>
<td>7.62</td>
<td>Reverse oblique</td>
<td>116.6</td>
<td>118.0</td>
<td>816.90</td>
<td>RSN1432_CHICHI_TAP046-E.AT2</td>
<td>RSN1432_CHICHI_TAP046-N.AT2</td>
</tr>
<tr>
<td>Chi-Chi, Taiwan</td>
<td>1999</td>
<td>TTN042</td>
<td>7.62</td>
<td>Reverse oblique</td>
<td>62.11</td>
<td>65.3</td>
<td>845.34</td>
<td>RSN1587_CHICHI_TTN042-N.AT2</td>
<td>RSN1587_CHICHI_TTN042-W.AT2</td>
</tr>
<tr>
<td>Tottori, Japan</td>
<td>2000</td>
<td>OKYH08</td>
<td>6.61</td>
<td>Strike slip</td>
<td>24.84</td>
<td>24.8</td>
<td>694.21</td>
<td>RSN3926_TOTTORI_OKYH08NS.AT2</td>
<td>RSN3926_TOTTORI_OKYH08EW.AT2</td>
</tr>
<tr>
<td>Tottori, Japan</td>
<td>2000</td>
<td>OKYH14</td>
<td>6.61</td>
<td>Strike slip</td>
<td>26.51</td>
<td>26.5</td>
<td>709.86</td>
<td>RSN3932_TOTTORI_OKYH14NS.AT2</td>
<td>RSN3932_TOTTORI_OKYH14EW.AT2</td>
</tr>
</tbody>
</table>
Table 5.3 The response spectra of the time histories of the following earthquakes are used for spectral matching with the target response spectra for 2% probability of exceedance in 50 years.

<table>
<thead>
<tr>
<th>Earthquake Name</th>
<th>Year</th>
<th>Station Name</th>
<th>Magnitude</th>
<th>Mechanism</th>
<th>Rjb (km)</th>
<th>Rrup (km)</th>
<th>$V_{s30}$ (m/s)</th>
<th>Horizontal-1 Acc. Filename</th>
<th>Horizontal-2 Acc. Filename</th>
</tr>
</thead>
<tbody>
<tr>
<td>Loma Prieta, USA</td>
<td>1989</td>
<td>Gilroy - Gavilan Coll.</td>
<td>6.93</td>
<td>Reverse oblique</td>
<td>9.19</td>
<td>9.96</td>
<td>729.65</td>
<td>RSN763_LOMAP_GIL067.AT2</td>
<td>RSN763_LOMAP_GIL337.AT2</td>
</tr>
<tr>
<td>Northridge-01, USA</td>
<td>1994</td>
<td>LA - Chalon Rd</td>
<td>6.69</td>
<td>Reverse</td>
<td>9.87</td>
<td>20.45</td>
<td>740.05</td>
<td>RSN989_NORTHR_CHL070.AT2</td>
<td>RSN989_NORTHR_CHL160.AT2</td>
</tr>
<tr>
<td>Northridge-01, USA</td>
<td>1994</td>
<td>Santa Susana Ground</td>
<td>6.69</td>
<td>Reverse</td>
<td>1.69</td>
<td>16.74</td>
<td>715.12</td>
<td>RSN1078_NORTHR_SSU000.AT2</td>
<td>RSN1078_NORTHR_SSU090.AT2</td>
</tr>
<tr>
<td>Kocaeli, Turkey</td>
<td>1999</td>
<td>Gebze</td>
<td>7.51</td>
<td>Strike slip</td>
<td>7.57</td>
<td>10.92</td>
<td>792.00</td>
<td>RSN1161_KOCAELI_GBZ000.AT2</td>
<td>RSN1161_KOCAELI_GBZ270.AT2</td>
</tr>
<tr>
<td>Hector Mine, USA</td>
<td>1999</td>
<td>Hector</td>
<td>7.13</td>
<td>Strike slip</td>
<td>10.35</td>
<td>11.66</td>
<td>726.00</td>
<td>RSN1787_HECTOR_HECO00.AT2</td>
<td>RSN1787_HECTOR_HEC090.AT2</td>
</tr>
<tr>
<td>Iwate, Japan</td>
<td>2008</td>
<td>IWT010</td>
<td>6.9</td>
<td>Reverse</td>
<td>16.26</td>
<td>16.27</td>
<td>825.83</td>
<td>RSN5618_IWATE_IWT010NS.AT2</td>
<td>RSN5618_IWATE_IWT010EW.AT2</td>
</tr>
<tr>
<td>Duzce, Turkey</td>
<td>1999</td>
<td>IRIGM 487</td>
<td>7.14</td>
<td>Strike slip</td>
<td>2.65</td>
<td>2.65</td>
<td>690.00</td>
<td>RSN8164_DUZCE_487-NS.AT2</td>
<td>RSN8164_DUZCE_487-EW.AT2</td>
</tr>
</tbody>
</table>
The response spectra of 14 acceleration time histories of 3 earthquakes have been matched with the target spectra of 10% probability of exceedance in 50 years (Figure 5.13 and Figure 5.14), and the response spectra of 14 acceleration time histories of 6 earthquakes have been matched with the target spectra of 2% probability of exceedance in 50 years (Figure 5.15 and Figure 5.16). The acceleration time histories of the matched response spectra were used for one-dimensional linear, equivalent-linear, and nonlinear site response analysis.

Figure 5.13 Response spectra of 14 time histories from 3 earthquakes (Table 5.1) with target response spectrum for 10% probability of exceedance in 50 years at BH-03 site.
Figure 5.14 Matched response spectra of 14 time histories with target response spectrum for 10% probability of exceedance in 50 years at BH-03 site.

Figure 5.15 Response spectra of 14 time histories from 6 earthquakes (Table 5.2) with target response spectrum for 2% probability of exceedance in 50 years at BH-03 site.
5.5 Site Response Analysis

The surface uniform hazard spectra (UHS) were generated from the bedrock UHS by using 1) the $V_{s30}$-based site coefficients of the NEHRP and 2) one-dimensional linear, equivalent-linear, and nonlinear site response analysis.

5.5.1 Site Response Analysis using $V_{s30}$-based Site Coefficients

The bedrock uniform hazard spectra (UHS) that were prepared using probabilistic seismic hazard analysis (PSHA) were multiplied by the $V_{s30}$-based site coefficients (amplification factors) to empirically predict the surface UHS. The UHS for 10 different sites in Dhaka City were prepared for 10%, 5%, and 2% probability of exceedance in 50 years (Figure 5.6 to Figure 5.8). The surface UHS were estimated using the EZ-FRISK.
5.5.2 One-dimensional Site Response Analysis

One-dimensional site response analysis has been performed to estimate the surface UHS by setting the dynamic properties of the soil profile and propagating the bedrock ground motion at the base of the soil profile. Linear, equivalent-linear, and nonlinear site response analyses were executed using the DEEPSOIL (Hashash et al. 2017). The one-dimensional site response analyses were performed for site effect estimation at the 10 sites where the surface UHS were estimated using the $V_s^{30}$-based site coefficients.

The dynamic properties of the soils were modeled by using the normalized modulus reduction and material damping curves that were proposed by Darendeli (2001). The initial effective vertical stress, initial coefficient of effective lateral earth pressure, plasticity index, over consolidation ratio, loading frequency, and number of loading cycles were required to use the relationships of Darendeli (2001) (Figure 5.5). Several soil models are available in the DEEPSOIL to perform site response analysis. In this study, the general quadratic/ hyperbolic model (GQ/H) with non-Masing reloading-unloading hysteretic formulation that was proposed by Groholski et al. (2016) was used to fit modulus reduction and material damping curves of the sandy and clayey soils of the profiles with the reference curves of Darendeli (2001) based on the specific shear strength. The water table was considered at the top of first soil layer (ground surface). Therefore, the effective shear strength was used for all calculations. The GQ/H model has the capacity to capture the large strain shear stress behavior of the soils to accurately estimate the surface ground motion at large strains (Groholski et al. 2016). The frequency independent viscous/small strain damping formulation option was used for time-domain nonlinear response analysis. The horizontal acceleration time histories of the matched bedrock response spectra were used for ground motion (e.g., Figure 5.12).
5.5.2.1 DEEPSOIL Software for Site Response Analysis

The DEEPSOIL was developed by Hashash et al. (2017) for one-dimensional site response analysis. One-dimensional linear, equivalent-linear, and nonlinear site response analysis can be performed using this software. Time- and frequency-domain linear, frequency-domain equivalent-linear, and time-domain nonlinear site response analysis options are available in DEEPSOIL. The nonlinear site response analysis with and without pore pressure generation can be performed using DEEPSOIL. The software is being regularly updated introducing new options and fixing problems to improve the accuracy of the results. For details of the software, the readers are referred to the “DEEPSOIL 7.0, User Manual” (Hashash et al. 2017).

5.5.2.2 Linear Site Response Analysis

The linear site response analysis has been performed in the frequency-domain. During the linear response analysis, the maximum stiffness of the soils and a constant damping ratio were applied throughout the duration of loading. Using the linear response analysis, the surface uniform hazard spectrum (UHS) was calculated for 10% probability of exceedance in 50 years (e.g., Figure 5.17). The linear response analysis was performed to observe the maximum amplification of seismic waves within the thick and soft sedimentary deposits with respect to the amplification of the equivalent-linear and nonlinear analyses.
Figure 5.17 Uniform hazard spectra (UHS) at ground surface using $V_s^{30}$-based site coefficients and UHS at ground surface using linear, equivalent-linear, and nonlinear ground response analysis at BH-03 site using the soil profile down to a depth of 303m at which the $V_s = 760$ m/s.

5.5.2.3 Equivalent-linear Site Response Analysis

The soft sedimentary deposits do not behave linearly during seismic loading. The dynamic properties of the soils, such as shear modulus and material damping, are changed throughout the seismic loading. In the equivalent-linear analysis, the dynamic properties of the soils are iteratively adjusted for each layer to approximate the nonlinear properties of the soils. The equivalent-linear analysis has been performed in the frequency-domain. The surface uniform hazard spectrum (UHS) of equivalent-linear response analysis for 10% probability of exceedance in 50 years is shown in Figure 5.17 as an example.
5.5.2.4 Nonlinear Site Response Analysis

The estimation of the surface ground motion using the equivalent-linear response analysis becomes inaccurate at high strain. As the dynamic properties of soils are nonlinear at high strain, the surface ground motion can be estimated more accurately using the nonlinear response analysis. Therefore, the nonlinear response analysis was performed at the same site where linear and equivalent-linear site response analysis were performed (e.g., Figure 5.17). The calculations of the nonlinear model were implemented in time-domain.

5.5.3 Comparisons of the UHS of Different Site Response Models

At a site, the uniform hazard spectra (UHS) that were predicted using the four different site response models were not similar (Figure 5.17). The spectral acceleration were amplified for both short and long period waves in case of the $V_s^{30}$-based site coefficient and linear response models, whereas the spectral acceleration were de-amplified for short period waves (from 0.01s to 0.35s) and amplified for long period waves (0.35s and 10s) in case of equivalent-linear and nonlinear analysis models. The spectral period of the maximum acceleration was shifted towards the long periods in case of the linear, equivalent-linear, and nonlinear models compared to the $V_s^{30}$-based model.

The spectral period of the maximum acceleration of the linear model was higher than that of the $V_s$-based model. The UHS of the equivalent-linear model was always higher than that of the nonlinear model and the spectral period of the maximum acceleration was same for both the models. As the bedrock ($V_s = 760$ m/s) in the study area exists at a depth of more than 195 m, the UHS of the $V_s^{30}$-based model is not appropriate to estimate the site effects of the thick and soft sedimentary deposits in the study area. Kaklamanos et al. (2015) found that at peak shear strain
from 0.01 to 0.1 %, the linear model fails to accurately estimate the ground motion at short periods.

It was also observed that the equivalent-linear model becomes inaccurate at shear strain of approximately 0.1 to 0.4% and the nonlinear model improves the prediction of the ground motion at high shear strain greater than 0.05%. During one-dimensional response analysis, the maximum shear strain was estimated greater than 0.05% in most of the soil layers of the profiles at 10 sites for the rock ground motion for 10% and 2% probability of exceedance in 50 years. Therefore, the nonlinear site response analysis was performed at 10 sites to accurately estimate the site effects.

5.5.4 Evaluation of Surface UHS using Nonlinear Response Analysis

In the study area, the near-surface time-averaged shear wave velocity in the top 30 m ($V_{s}^{30}$) is less than 760 m/s. The bedrock ($V_{s} \geq 760$ m/s) also does not exist within the depth of 30 m. It exists at a depth of 195 to 328 m at 10 sites in the study area. Therefore, the nonlinear analysis was performed using different depths of soil profiles setting bedrock ground motion ($V_{s} = 760$ m/s) at the bottom of each profile to observe the depth dependency of the ground motion in a very thick unconsolidated sedimentary deposits of Dhaka City, where the bedrock ($V_{s} = 760$ m/s) exists at a depth of more than 195 m.

The bedrock UHS, $V_{s}^{30}$-based surface UHS, and surface UHS of the nonlinear model for 30 m, 100 m, 200 m, and 303 m depths of soil profiles were prepared at BH-03 and shown in Figure 5.18. At BH-03, the bedrock ($V_{s} = 760$ m/s) exists at a depth of 303 m. In Figure 5.18, it is observed that the surface UHS calculated using the $V_{s}^{30}$-based model is higher at low spectral periods than that of nonlinear model. The PGA and spectral acceleration are decreasing with increasing the depth of the profile in case of nonlinear analysis. Park and Hashash (2005) identified that the amplitude of the PGA and spectral acceleration are decreasing with increasing profile depth due
to low strain levels and viscous damping at higher profile depth. They observed that accurate estimation of surface ground motion at a very thick soft sedimentary rock (1000 m) depends on the thickness of the profile. In the study area, similar characteristics are observed for the UHS of different depths of soil profiles. Therefore, in this study, the nonlinear response analysis has been performed at 10 sites using the soil profile to the depth of the bedrock to estimate the surface UHS for 10 % and 2 % probability of exceedance in 50 years (Figure 5.19 and Figure 5.20).

![Figure 5.18 Uniform hazard spectra (UHS) at bedrock condition (V_{s30} = 760 m/s) using probabilistic seismic hazard analysis, UHS at ground surface using V_{s30}-based site coefficients, nonlinear ground response analysis using different depths of soil profiles at BH-03 site for 10 % probability of exceedance in 50 years.](image)
Figure 5.19 Uniform hazard spectra (UHS) at ground surface for 10% probability of exceedance in 50 years using nonlinear (NL) ground response analysis at 10 borehole (BH) sites. The depth where shear wave velocity ($V_s$) is equal to or more than 760 m/s is used as soil profile depth. The UHS for bedrock condition ($V_{s30} = 760$ m/s) is at BH-03 using probabilistic seismic hazard analysis (PSHA) for 10% probability of exceedance in 50 years.

Figure 5.20 Uniform hazard spectra (UHS) at ground surface for 2% probability of exceedance in 50 years using nonlinear (NL) ground response analysis at 10 borehole (BH) sites. The depth, where shear wave velocity ($V_s$) is equal to or more than 760 m/s, is used as soil profile depth. The UHS for bedrock condition ($V_{s30} = 760$ m/s) is at BH-03 using probabilistic seismic hazard analysis (PSHA) for 2% probability of exceedance in 50 years.
From the surface UHS for 10% probability of exceedance in 50 years (Figure 5.19), it is observed that the highest amplification occurs at BH-09, where the highest impedance contrast occurs at a depth of 3 m at the boundary between the artificial filling ($V_s = 96$ m/s) and the Pleistocene deposits ($V_s = 259$ m/s) (Table 5.3). The highest amplification are gradually decreasing and shifting towards higher spectral periods with increasing the depth of the impedance contrast boundary, such as BH-05 (5 m), BH-03 (8 m), BH-06 (17 m), and BH-10 (19 m). The amplification of BH-08 (5 m) and BH-04 (10 m) are relatively lower than that of BH-05 (5 m) and BH-03 (8 m) due to low impedance contrast boundary at BH-08 (5 m) and BH-04 (10 m).

In the study area, the high impedance contrast exits at the boundary between the artificial filling or alluvium and the Pleistocene deposits. In the artificial filling and alluvium, the shear wave velocity ($V_s$) varies from 78 m/s to 137 m/s above the boundary of the shallowest impedance contrast. Below the boundary the $V_s$ varies from 188 m/s to 239 m/s in the Pleistocene deposits (Table 5.3). At BH-01, BH-02, and BH-07, the Pleistocene deposits are exposed and the $V_s$ values at these sites are higher than the $V_s$ values of other sites. At BH-07, BH-01, and BH-02, the relatively low impedance contrast boundary exists at a depth of 3 m, 17 m, and 24 m. The highest amplifications at these sites are also shifting towards higher spectral periods with increasing the depth of the impedance contrast boundary. As the $V_s$ of the Pleistocene deposit is higher than that of the Holocene and artificial filling, the resonance frequency of the Pleistocene deposit is higher than that of the Holocene and artificial filling (Figure 5.19).

From the UHS for 2% probability of exceedance in 50 years (Figure 5.20), it is observed that the amplifications at different sites are relatively low due to high amplitude of the rock ground motion for long return period. The high frequency waves are attenuated more than the low
frequency waves when passing through thick column of soft and loose soils that are mainly composed of sandy soils below 30 m depth. As the resonance frequency in the Pleistocene deposit is high, the high frequency wave will amplify more in the Pleistocene deposit than that of the thick and soft Holocene deposit. The resonance frequency of the thick Holocene deposit is low. Therefore, the low frequency wave will amplify more in the thick Holocene deposit than the Pleistocene deposit.

The high frequency waves, that carry more energy than low frequency waves, arrive in the site from the near-field earthquakes. Therefore, the high frequency waves will amplify in the Pleistocene deposit, as the resonance frequency of the Pleistocene deposit is high. The low frequency waves arrive in the site from the far-field earthquake and high frequency waves are attenuated due to long distance from source to site and during passing through thick soil column. As the resonance frequency of the Holocene deposit is low, the low frequency waves will amplify in the thick Holocene deposit.

In the study area where the high impedance contrast boundary between the artificial filling or the Holocene deposit and the Pleistocene deposit exists at shallow depth (3 m to 8 m), the amplification is exceptionally very high (BH-03, BH-05, and BH-09 in Figure 5.19). The ground motion amplification for the UHS of 2% probability of exceedance in 50 years is lower than that of 10% probability of exceedance in 50 years due to more attenuation of the high amplitude ground motion when passing through thick and soft sedimentary deposits (Figure 5.20).
5.5.5 Nonlinear Analysis using the Acceleration Time History Recorded at the Station of KiK-net

The KiK-net (Kiban Kyoshin network) is a network of strong ground motion seismographs operated by the National Research Institute for Earth Science and Disaster Prevention (NIED), Japan. At KiK-net stations, high sensitivity seismographs were installed at the ground surface as well as in the deep borehole. As a result, acceleration time history of an earthquake can be recorded in the subsurface as well as on the ground surface. The site effects can be estimated from the acceleration time histories of the surface and subsurface seismographs.

In this study, the acceleration time history of the 2011 Tohoku Earthquake ($M_w = 9.0$) was used for nonlinear response analysis. The acceleration time history was taken from the seismograph that was installed at a depth of 200 m in the borehole at FKSH10 station of KiK-net. The station was location 266 km away from the epicenter in the southwest direction. The response spectrum of this acceleration time history was similar to the target response spectrum of the study area for 10% probability of exceedance in 50 years. As the seismograph was located at a depth of 200 m, the soil profile of 200 m depth with the soil properties of BH-09 was used for this analysis. The shear wave velocity profiles at BH-09 and FKSH10 station are shown in Figure 5.21 (a). The uniform hazard spectrum (UHS) of this analysis is plotted with the response spectra of the surface and borehole seismographs at FKSH10 station in Figure 5.21 (b).
In Figure 5.21 (b), it is observed that the site amplification at FKSH10 station is very high compared to the site amplification at BH-09 in Dhaka. This is due to the very high impedance contrast at shallow depth and different material properties (Figure 5.21(a)) at FKSH10 station. From this observation, it is clear that if very high impedance contrast exists at shadow depth, the amplification will be very high. In Dhaka City, relatively high impedance contrast exists at the boundary between the artificially filled materials and the Pleistocene deposits. Therefore, high amplification will occur at this boundary. In these soils, the spectral period of high amplification is close to the natural period of the low-rise buildings. Therefore, the damage to the low-rise buildings will be increased in these soils.
5.5.6 Developing Site Amplification Function

After completing the site response analysis, it is necessary to interpret the results of the site response analysis in the form of site amplification functions that are conditioned on the intensity of the ground motion (Stewart et al. 2014). The site amplification function is used to merge the results of the site response analysis with the ground motion hazard of a reference site condition. Seyhan and Stewart (2014) expressed the mean site amplification function as

\[ \ln \bar{Y}(f) = f_1 + f_2 \ln \left( \frac{x_{I M ref} + f_3}{f_3} \right) \]  
\text{Eq. 5.1}

where, \( \ln \bar{Y}(f) \) represents mean amplification, \( x_{I M ref} \) is the ground motion hazard at the reference site condition used as the driver of nonlinearity, \( f_1 \) represents weak-motion (linear) amplification, \( f_2 \) represents nonlinearity, and \( f_3 \) represents the level of reference site ground motion below which the amplification converges towards a linear (constant) upper limit.

The site coefficients \( f_1, f_2 \) and \( f_3 \) were derived using the spreadsheets that were provided by Stewart et al. (2014) as electronic supplement. The spreadsheets includes the regression of the mean site amplification function, interpolation of the site coefficients for a range of periods, and different approaches to merge the results of the site response analysis with the results of the PSHA.

The mean site amplification function was determined for each site using the reference ground motion for 10% and 2% probability of exceedance in 50 years. For an example, the site amplification function for the reference PGA and spectral acceleration at 0.05, 0.1, 0.2, 0.5, and 1.0 s periods are shown in Figure 5.22 and Figure 5.23.
5.5.7 Interpolation of Site coefficients for a Range of Periods

It is often not practical to perform the fitting operation for the site coefficients $f_1$, $f_2$, and $f_3$ for all of the periods that are used to construct a response spectrum. Therefore, the site coefficients $f_1$, $f_2$, and $f_3$ were calculated for the periods of 0.01 (PGA), 0.05, 0.1, 0.2, 0.5, and 1.0 s. Then, the site coefficients were interpolated for a range of periods from 0.01 to 10 s to predict the coefficients for intermediate periods. For an example, the interpolation of the site coefficients ($f_1$, $f_2$, and $f_3$) for a range of periods from 0.01 to 10 s are shown in Figure 5.24. Using these site coefficients and the intensity of the reference ground motion, the mean site amplification for spectral periods from 0.01 to 10 s can be predicted from Eq. 1.
Figure 5.22 Site amplification of the reference ground motion at borehole site BH-10 for (a) peak ground acceleration (PGA), and for ground acceleration at spectral periods of (b) 0.05 s, (c) 0.1 s, and (d) 0.2 s. The GRA and SS14 stand for ground response analysis and model of Seyhan and Stewart (2014), respectively.
Figure 5.23 Site amplification of the reference ground motion at borehole site BH-10 and for ground acceleration at spectral periods of (e) 0.5 s and (f) 1.0 s. The GRA and SS14 stand for ground response analysis and model of Seyhan and Stewart (2014), respectively.
Figure 5.24 Interpolation site coefficients at borehole site BH-10 for a period range of 0.01 to 10 s. The GRA and SS14 stand for ground response analysis and model of Seyhan and Stewart (2014), respectively.
5.5.8 Simplified Approaches to Merge Site effects with PSHA

The site response analysis provides the deterministic estimation of site effects using some input parameters. The results of this analysis are typically merged with the probabilistically-determined rock ground motion in some ways (Stewart et al. 2014). In this study, three simplified approaches were used for merging the site amplification function of the site response analysis with the reference ground motion of the probabilistic seismic hazard analysis (PSHA). The approaches are hybrid, modified hybrid, and convolution.

In hybrid approach, the rock ground motion of the PSHA is multiplied with the mean site amplification to estimate the surface ground motion (Cramer 2003). It is referred to as hybrid, because it merges the probabilistic rock ground motion with the deterministic site amplification. This approach is widely used in both engineering practice and research community. Goulet and Stewart (2009) proposed a modified hybrid approach in which the reference rock ground motion that is used in estimating mean site amplification is taken as a mean value. Bazzurro and Cornell (2004b) suggested a convolution approach to merge the nonlinear site amplification function with the rock ground motion to estimate the surface ground motion. Stewart et al. (2014) discussed the advantages and disadvantages of these approaches.

The nonlinear site amplification was merged with rock ground motion of the PSHA using the spreadsheet that was provided by Stewart et al. (2014) as electronic supplement. For an example, the nonlinear site amplifications for the reference PGA and spectral acceleration at 0.2 and 1.0 s were merged with the reference PGA and spectral acceleration at 0.2 and 1.0 s using hybrid, modified hybrid, and convolution approaches at borehole site BH-10 (Figure 5.25).
Figure 5.25 Merging the nonlinear site amplification with the rock ground motion using hybrid, modified hybrid, and convolution approaches at borehole site BH-10 for (a) peak ground acceleration (PGA), (b) spectral acceleration at 0.2 s, and (c) spectral acceleration at 1.0 s.
5.6 Discussions

The seismic waves can be influenced by the near-surface soils, surface topography, and basin structure of the site. In this study, only soil effects of the site were predicted using the $V_s^{30}$-based site coefficients and one-dimensional linear, equivalent-linear, and nonlinear site response analysis.

In the study area, the thickness of the soft sedimentary deposits ($V_s \leq 760$ m/s) is more than 195 m, whereas the $V_s^{30}$-based site coefficients are recommended based on the thickness of the soil down to a depth of 30 m. The soils encountered below 30 m depth are not considered for the estimation of the $V_s^{30}$-based site coefficients. The $V_s^{30}$-based site coefficients overpredict the surface ground motion at short spectral periods and underpredict at long spectral periods compared to the surface ground motion that has been predicted using nonlinear site response analysis taking soil thickness of 30 m (Figure 5.18). The maximum site amplification of the nonlinear UHS is shifted towards the higher spectral period compared to the $V_s^{30}$-based ones. When the thickness of the soil profile (30, 100, 200, and 303 m) is increased in nonlinear model, the amplification of the UHS is deceased up to the spectral period of 0.6 s with increasing the thickness of the soil profile. As the depth of the bedrock ($V_s = 760$ m/s) in this site is 303 m, the surface UHS that was estimated using the soil profile depth of 303 m was considered to estimate the site amplification of this site. The UHS of the linear and equivalent-linear analyses is also high compared to that of the nonlinear analysis (Figure 5.17). As the surface ground motion can be accurately predicted using nonlinear site response analysis, in this study, the surface UHS at the 10 sites is estimated using the nonlinear analysis taking the soil profile down to the depth of bedrock for 10% and 2% probability of exceedance in 50 years (Figure 5.19 and Figure 5.20).
The mean site amplification function can be easily estimated for different spectral periods using the spreadsheet of regression analysis that was provided by Stewart et al. (2014) as electronic supplement (Figure 5.22 and Figure 5.23). The site coefficients of the site amplification function (Eq. 1) can be interpolated for a range of periods from 0.01 and 10 s (Figure 5.24) using the spread sheet of electronic supplement. Then, the results of the site response analysis can be merged with the rock ground motion of the probabilistic seismic hazard analysis (PSHA) to estimate the surface ground motion at different spectral periods using hybrid (Cramer 2003), modified hybrid (Goulet and Stewart 2009), and convolution (Bazzurro and Cornell 2004b) approaches (Figure 5.25).

From the results of the nonlinear site response, it is observed that the rock ground motion is de-amplified at short spectral periods and amplified at long spectral periods (Figure 5.19 and Figure 5.20). The de-amplification is increased with increasing ground motion intensities. The short period seismic waves arrive from near-field earthquake sources and the long period seismic waves arrive from far-field earthquake sources. In the study area, the short period seismic waves are de-amplified and the long period seismic waves are amplified due to the presence of very thick and soft sedimentary deposits (sand and clay) above the bedrock $(V_s = 760 \text{ m/s})$.

There is no known source of large magnitude earthquake ($M_w > 5$) within a radius of 50 km from the city center of Dhaka. The closest source of large magnitude earthquake was the 1885 Bengal Earthquake ($M_w = 6.9$). The epicenter of this earthquake was determined by Middlemiss (1885) at 50 km away in the northwest direction from the city center. In Dhaka City, the damage of this earthquake was very small compared to other places equally near the center of disturbance (Middlemiss 1885). This evidence indicated that the source of this earthquake was farther away from Dhaka City. The epicenter of this earthquake might be located 100 km away in the northwest
from Dhaka, which is near to the highest damage areas (Bogra, Sirajgonj, Jamalpur, Sherpur, and Mymensigh).

Singh et al. (2016) predicted that the thickness of the sediments increases dramatically across the Hinge Zone of the Early Cretaceous passive margin from 3 to 17 km south of Madhupur Terrace (Figure 3.2). The areas of the highest damage were located close to the Hinge Zone. In the areas of the highest damage, the crystalline basement rock exists at shallower depth than Dhaka City. Therefore, the short period seismic waves caused damage near to the epicenter. As these areas were located close to the basin boundary, the seismic waves were also influenced by the basin structure. On the other hand, Dhaka City is located on the thick sedimentary deposits in the middle of the Bengal Basin, where the crystalline basement rocks exists at a depth of more than 16 km (Singh et al. 2016). Due to very thick and soft sedimentary deposits, the short period seismic waves were de-amplified and the long period seismic waves were amplified in Dhaka City. At that time, one to three story buildings were present in Dhaka. As the short period seismic waves were de-amplified in Dhaka, the low-rise buildings (1-3 story) were not damaged during this earthquake.

Similar phenomena might occur during the 1918 Srimangal Earthquake (M\textsubscript{w} = 7.1) that was located about 150 km away in the northeast direction from Dhaka. No damage was reported in Dhaka during this earthquake (Stuart 1920). The 1897 Great Assam Earthquake (M\textsubscript{w} = 8.1) that was located more than 200 km away in the north direction from Dhaka. In Dhaka, almost all the building were badly damaged, some buildings were completely collapsed, and many were uninhabitable (Oldham 1899). At that time, the unreinforced masonry buildings were present in Dhaka. Due to large magnitude earthquake, the seismic waves carried sufficient energy that generated high peak ground acceleration and spectral acceleration in Dhaka. Although the short
period seismic waves were de-amplified and the long period seismic waves were amplified due to the presence of very thick and soft sedimentary deposits, their accelerations were still sufficient to cause damage to the unreinforced masonry buildings in Dhaka.

At that time, the city was confined to the southern part of the Pleistocene Madhupur Terrace. Later, the city has been expanded to the surrounding Holocene floodplains and many high-rise buildings have been constructed on both the Pleistocene Terrace and Holocene floodplains after elevating the ground using sand dredging from the river. As the sources of large crustal and subduction earthquakes are not present within a radius of 50 km and 100 km, respectively, from the city, mostly the long period seismic waves will amplify after propagation through the thick and soft sedimentary deposits. The spectral periods of these waves will match with the natural periods of the high-rise buildings to cause resonance (e.g. BH-06, BH-10 in Figure 5.19 and Figure 5.20). Therefore, the damage to the high-rise buildings that are located on the Holocene floodplains will be increased. The short period seismic waves will amplify at the sites, where the Pleistocene deposits are overlain by a thin layer of artificially filled sand and clay or the Holocene sand and clay (e.g. BH-05, BH-09 in Figure 5.19 and Figure 5.20). The spectral periods of these waves will match with the natural periods of the low-rise buildings to cause resonance. Therefore, the damage to the low-rise buildings that are located on the valleys of the Pleistocene Terrace will be increased.

5.7 Summary

One-dimensional nonlinear site response analysis is required to accurately estimate the surface ground motion in the areas where the bedrock (\(V_s = 760 \text{ m/s}\)) is overlain by the soft sedimentary deposits of more than 30 m thick. In these areas, the \(V_s^{30}\)-based site amplifications overestimate the ground motion at short spectral periods and underestimate the ground motion at long periods.
As the properties of the soils are nonlinear, the nonlinear site response analysis better predicts the ground motion than the linear or equivalent-linear ones.

In areas of thick and soft Holocene deposits, the long period seismic waves of the far-field earthquakes will amplify and the resonance will occur with the natural periods of the high-rise building, and consequently, the damage to high-rise buildings will be increased in these areas. In areas where the shallow valleys of the Pleistocene Terrace were filled artificially with sand and clay (3-5 m thick), the short period seismic waves will amplify and the resonance will occur with the natural periods of the low-rise buildings, and consequently, the damage to the low-rise building will be increased in these areas.
Chapter 6: Liquefaction Potential Evaluation

6.1 Background

A number of studies on liquefaction and liquefaction-induced ground failures were performed by many researchers after the devastating earthquakes of Alaska and Niigata which occurred in 1964, where slope, bridge and foundation failures were observed as a result of soil liquefaction (Sonmez and Gokceoglu 2005). Historical incidents of earthquakes indicate that destructive earthquakes may occur around Bangladesh (Bilham and England 2001, Ambraseys and Bilham 2003, Bilham and Wallace 2005). The occurred historical earthquakes in Bangladesh and NE India are listed in Table 6.1 from Choudhury (2005). Some of these earthquakes such as the 1885 Bengal Earthquake (Middlemiss 1885), 1897 Great Indian Earthquake (Oldham 1899) and 1918 Srimangal Earthquake (Stuart 1920), caused serious damage to buildings and other infrastructures of Bangladesh. Although significant damage was reported in Dhaka City during the 1897 Great Indian Earthquake and 1885 Bengal Earthquake, there was no document on the extent of the damage in Dhaka during the 1918 Srimangal earthquake. Therefore, moderate to large earthquake magnitudes may occur in this region due to continuing tectonic deformation along the plate boundaries and active faults (CDMP 2009). When earthquakes hit the developing countries, millions of fatalities may occur (Bilham 2009). Such earthquakes may also increase in damage to buildings, bridges, industrial and port facilities, etc. The soil liquefaction is one of the major reasons for the increase in damage to infrastructures.
Table 6.1 List of historical earthquakes occurred in Bangladesh and NE India (Rahman et al. 2015c). Don’t require permission of Elsevier as the author of this article.

<table>
<thead>
<tr>
<th>Date</th>
<th>Name of earthquake</th>
<th>Magnitude (Richter)</th>
<th>Intensity at Dhaka (EMS)</th>
<th>Epicentral distance from Dhaka (km)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10/01/1869</td>
<td>Cacher Earthquake</td>
<td>7.5</td>
<td>V</td>
<td>250</td>
</tr>
<tr>
<td>14/07/1885</td>
<td>Bengal Earthquake</td>
<td>7.0</td>
<td>VII</td>
<td>170 (50*)</td>
</tr>
<tr>
<td>12/06/1897</td>
<td>Great Indian Earthquake</td>
<td>8.7</td>
<td>VIII+</td>
<td>230</td>
</tr>
<tr>
<td>8/07/1918</td>
<td>Srimangal Earthquake</td>
<td>7.6</td>
<td>VI</td>
<td>150</td>
</tr>
<tr>
<td>02/07/1930</td>
<td>Dhubri Earthquake</td>
<td>7.1</td>
<td>V+</td>
<td>250</td>
</tr>
<tr>
<td>15/01/1934</td>
<td>Bihar-Nepal Earthquake</td>
<td>8.3</td>
<td>IV</td>
<td>510</td>
</tr>
<tr>
<td>15/08/1950</td>
<td>Assam Earthquake</td>
<td>8.5</td>
<td>IV</td>
<td>780</td>
</tr>
</tbody>
</table>

* According to Middlemiss (1885), epicenter was located near Atia, Manikganj which is about 50 km northwest from Dhaka City.

Earthquake induced liquefaction phenomena have been recorded and developed in many parts of the world (Seed and Idriss 1971, 1982, Iwasaki et al. 1978, 1982, Seed et al. 1984, 1985, 2003, Robertson and Wride 1998, Youd and Idriss 2001, Youd et al. 2001). The methods of liquefaction susceptibility analysis and mapping have further been modified, improved, calibrated and validated by many researchers (Chen and Juang 2000, Juang et al. 2003, 2008, 2009, Lee et al. 2004, Papanathanassiou et al. 2005, Sonmez and Gokceoglu 2005, Cox et al. 2007, Papanathanassiou 2008, Sonmez et al. 2008, Holzer 2008, Jha and Suzuki 2009, Heidari and Andrus 2010, Valverde-Palacios et al. 2014, Kang et al. 2014). Dhaka City is located close to seismically active zone. The eastern, western, southeastern parts of the city are covered by the Holocene sand, silty sand, silty clay, sandy- and clayey-silt up to more than 20 m depth from the ground surface. However, there are few studies about the potential of liquefaction in Dhaka City, and a limited number of literature is available on seismically induced liquefaction hazard assessment of the city. Therefore, in this study, an attempt was taken to prepare a seismically induced liquefaction hazard map of Dhaka City Corporation area. The objectives of this research were to compute liquefaction potential of
the subsurface geological materials of Dhaka City using Simplified Procedure to estimate Liquefaction Potential Index (LPI), and to prepare a liquefaction hazard map by using contour lines of equal LPI values of SPT profiles located at different places of the city and by cumulative frequency distribution of LPI of different geological materials.

6.2 Geomorphology and Geology

Dhaka, the capital city of Bangladesh, has become a member of the megacities of the world. The city is located at the bank of the Buriganga River in the central part of the country (Figure 6.1). It covers an area of 321 sq km having a population of around 14 million as the prime city in Bangladesh. The city is surrounded by the Tongi Khal in the north, the Buriganga River in the south and south-west, the Turag River in the west and the Balu River in the east. Due to the unplanned civilization starting more than 400 years ago, many buildings in the old part of the city are non-engineered structures. Most of the buildings, which may be engineered and non-engineered, have also been constructed on the artificial sand fillings of the recent floodplains of the Buriganga, Turag, Balu and Sitalaykhah Rivers.

The city is almost flat with many depressions, and regional elevations of the area gradually decline towards the south and west. Although the slopes of the western and eastern parts of the city are towards the west and east respectively, but the general slope of the city is from the north to south and south-east. The elevations of Dhaka City vary from 2 to 14 m above main sea level (MSL) with an average of 6.5 m (JICA 1987).

Bangladesh constitutes the major part of the Bengal Basin bounded by the Precambrian Indian Shield in the west, by the Shillong Massif in the north and by the Frontal Folded Belt of the Indo-Burman Ranges in the east. It is open to the Bay of Bengal in the south (Alam 1989, Reimann
1993). The city covers the southern part of the Pleistocene Madhupur Terrace and the surrounding Holocene floodplains.

Dhaka City may be divided into six geological units based on geomorphology, stratigraphy and geotechnical properties of the geological materials. These units are Pleistocene terrace deposit (Qpty), Holocene Alluvial valley fill deposit (Qhav), Holocene Alluvial channel deposit (Qhc), Holocene Alluvium (Qha), Holocene terrace deposit (Qhty) and Artificial fill (af) (Figure 6.2). Some samples from the geological materials of the six units are classified according to the Unified Soil Classification System, and are given in Table 5.1. The Pleistocene terrace deposit in the central part of the city is generally composed of yellowish brown to reddish brown stiff to very stiff clayey

Figure 6.1 Location map of the study area (Rahman et al. 2015c). Don’t require permission of Elsevier as the author of this article.
silt, silty clay and medium dense to dense silty sand and sand. The Holocene Alluvial valley fill deposit is located in the depressions or valleys of the Pleistocene terrace deposit, consisting of dark grey to grey very soft to soft silty clay, clayey silt and grey to yellowish brown, very loose to medium dense silty sand. The Holocene Alluvial channel deposit in the present river channels is formed of grey very loose to loose silty sand and sand. The Holocene terrace deposit is present in the point bars, channel bars consisting of grey loose to medium dense silty sand, sand and very soft to stiff clayey silt. The Holocene Alluvium located in the eastern, southeastern and northwestern parts of the city is composed of grey very soft to medium stiff silty clay, clayey silt and very loose to loose silty sand. The artificial fills in the western and eastern parts of the city are composed of grey clayey silt, silty sand and sand. Most of the artificial fills in the western and eastern parts of the city were emplaced both by hydraulic dredging from the river and by truck from land. The ground was not improved during or after artificial filling considering the liquefaction potentiality of the soils.

6.3 Seismotectonics

Bangladesh covers a major part of the Bengal Basin, one of the largest sedimentary basins of the world (Alam 1972). The northward collision of the Indian Plate with the Eurasian Plate created the Himalayan Ranges between the Indian Plate and Eurasian Plate and the Bengal Basin in the eastern part of the Indian Plate (Curray and Moore 1974, Curray et al. 1982, Alam et al. 2003, Aitchison et al. 2007). As the plate boundaries are considered the main sources of earthquake, the major part of the Bengal Basin is seismically active. Several large earthquakes have occurred along plate boundaries and active faults of this landmass in the past (Table 6.1).
Figure 6.2 Surface geological map of Dhaka City with borehole locations (Rahman et al. 2015c). Don’t require permission of Elsevier as the author of this article.
According to the report of the Comprehensive Disaster Management Programme (CDMP 2009), the most critical earthquake for Dhaka City is the 1885 Bengal Earthquake. The epicenter of this seismic event was located near Atia in Manikganj District (Middlemiss 1885). The location of the epicenter was about 50 km NW from the city center. At that time the epicenter was determined using an intensity map of this earthquake. The intensity of this earthquake at Dhaka City was VII in European Microseismic Scale (Choudhury 2005). A recent study indicated that the epicenter of this earthquake was along the Madhupur Fault, which is located about 60 km NNW from Dhaka City (CDMP 2009). Therefore, the Madhupur Fault is the most important seismic source for Dhaka City. In the seismic zoning map, Bangladesh is divided into three seismic zones based on peak ground acceleration (BNBC 1993). The zones are Zone I, Zone II and Zone III, where the values are 0.075g, 0.15g and 0.25g. Dhaka City is situated in Zone II where peak ground acceleration is 0.15g.

6.4 Evaluation of Liquefaction Potential of Geological Units

The Standard Penetration Test (SPT) data is widely used to evaluate the liquefaction potential of geological units up to a depth of 20 m from the ground surface (Seed and Idriss 1971, 1982, Seed et al. 1985, 2001, Juang et al. 2000, Youd et al. 2001, Idriss and Boulanger 2004, Sonmez and Gokceoglu 2005, Sonmez et al. 2008). In this study, SPT N-values, along with other necessary geotechnical properties of 53 borehole profiles located in different geological units of Dhaka City, were used to assess the liquefaction potential of the geological materials of the city using Simplified Procedure to determine the factor of safety against liquefaction. Then the Liquefaction Potential Index (LPI) of each SPT profile was estimated to produce a liquefaction hazard map of the city.
6.4.1 Establishment of Database

The data of the subsurface geological materials of Dhaka City was collected from existing literature and Comprehensive Disaster Management Programme (CDMP). Boreholes data having sufficient information, such as SPT N-values, geotechnical properties and geological information for determination of Liquefaction Potential Index (LPI) was selected for this study. Under a project on seismic hazard and vulnerability assessment of Dhaka, Chittagong and Sylhet City Corporation areas, which was implemented in 2009 by the Comprehensive Disaster Management Programme (CDMP), Ministry of Food and Disaster Management, Bangladesh.

The borehole locations have been classified based on the surface geological units of the city. The location of the boreholes has been shown in the surface geological map of the city (Figure 6.2). Twenty three (23) boreholes on the Pleistocene terrace deposit (Qpty), seven (7) boreholes on the Holocene Alluvial valley fill deposit (Qhav), twelve (12) boreholes on the Holocene Alluvium (Qha), four (4) boreholes on the Holocene terrace deposit (Qhty) and seven boreholes on artificial fill (af) were located. The depths of the boreholes, numbers of SPT N-values at each borehole and the LPI value for each borehole have been summarized in Table 6.2.
Table 6.2 Liquefaction potential index (LPI) calculated for each SPT profile for an earthquake scenario of $M_w = 7$ having peak horizontal ground acceleration ($a_{max}$) of 0.15g (Rahman et al. 2015c). Don’t require permission of Elsevier as the author of this article.

<table>
<thead>
<tr>
<th>Borehole No.</th>
<th>Coordinates</th>
<th>Elevation (m)</th>
<th>Depth of GWT (m)</th>
<th>Drilling Depth (m)</th>
<th>SPT (Times) (each 1.5 m interval)</th>
<th>Liquefaction Potential Index (LPI)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Dhk-01</td>
<td>23°52'21.20&quot; N, 90°25'22.00&quot; E</td>
<td>6.8</td>
<td>7.0</td>
<td>21.0</td>
<td>11</td>
<td>0.00</td>
</tr>
<tr>
<td>Dhk-02</td>
<td>23°42'12.00&quot; N, 90°25'21.80&quot; E</td>
<td>6.0</td>
<td>5.0</td>
<td>21.0</td>
<td>11</td>
<td>0.00</td>
</tr>
<tr>
<td>Dhk-03</td>
<td>23°43'15.50&quot; N, 90°29'40.20&quot; E</td>
<td>6.0</td>
<td>3.0</td>
<td>21.0</td>
<td>11</td>
<td>10.88</td>
</tr>
<tr>
<td>Dhk-04</td>
<td>23°50'59.90&quot; N, 90°25'28.90&quot; E</td>
<td>4.5</td>
<td>5.0</td>
<td>21.0</td>
<td>11</td>
<td>4.30</td>
</tr>
<tr>
<td>Dhk-05</td>
<td>23°51'42.70&quot; N, 90°26'42.30&quot; E</td>
<td>2.0</td>
<td>3.0</td>
<td>21.0</td>
<td>11</td>
<td>0.00</td>
</tr>
<tr>
<td>Dhk-06</td>
<td>23°53'05.10&quot; N, 90°21'39.40&quot; E</td>
<td>1.8</td>
<td>5.0</td>
<td>21.0</td>
<td>11</td>
<td>7.88</td>
</tr>
<tr>
<td>Dhk-07</td>
<td>23°48'06.60&quot; N, 90°20'56.58&quot; E</td>
<td>14.3</td>
<td>18.0</td>
<td>21.0</td>
<td>11</td>
<td>0.00</td>
</tr>
<tr>
<td>Dhk-08</td>
<td>23°45'57.84&quot; N, 90°20'43.68&quot; E</td>
<td>5.2</td>
<td>4.0</td>
<td>21.0</td>
<td>11</td>
<td>2.86</td>
</tr>
<tr>
<td>Dhk-09</td>
<td>23°44'42.12&quot; N, 90°27'13.74&quot; E</td>
<td>5.0</td>
<td>4.5</td>
<td>21.0</td>
<td>11</td>
<td>0.00</td>
</tr>
<tr>
<td>Dhk-10</td>
<td>23°48'02.00&quot; N, 90°27'14.90&quot; E</td>
<td>1.9</td>
<td>4.0</td>
<td>21.0</td>
<td>11</td>
<td>10.72</td>
</tr>
<tr>
<td>Dhk-11</td>
<td>23°42'58.00&quot; N, 90°22'08.00&quot; E</td>
<td>5.8</td>
<td>4.5</td>
<td>21.0</td>
<td>11</td>
<td>9.18</td>
</tr>
<tr>
<td>Dhk-12</td>
<td>23°43'26.00&quot; N, 90°22'28.00&quot; E</td>
<td>5.8</td>
<td>5.5</td>
<td>21.0</td>
<td>11</td>
<td>0.27</td>
</tr>
<tr>
<td>Dhk-13</td>
<td>23°43'48.00&quot; N, 90°22'55.00&quot; E</td>
<td>8.0</td>
<td>1.5</td>
<td>21.0</td>
<td>11</td>
<td>0.78</td>
</tr>
<tr>
<td>Dhk-14</td>
<td>23°45'53.00&quot; N, 90°25'37.00&quot; E</td>
<td>5.8</td>
<td>1.5</td>
<td>21.0</td>
<td>11</td>
<td>13.35</td>
</tr>
<tr>
<td>Dhk-15</td>
<td>23°47'36.00&quot; N, 90°27'05.00&quot; E</td>
<td>2.7</td>
<td>1.5</td>
<td>21.0</td>
<td>11</td>
<td>8.69</td>
</tr>
<tr>
<td>Dhk-16</td>
<td>23°47'54.00&quot; N, 90°28'24.00&quot; E</td>
<td>3.0</td>
<td>4.5</td>
<td>21.0</td>
<td>11</td>
<td>1.09</td>
</tr>
<tr>
<td>Dhk-17</td>
<td>23°49'59.00&quot; N, 90°27'06.00&quot; E</td>
<td>1.5</td>
<td>1.5</td>
<td>21.0</td>
<td>11</td>
<td>17.86</td>
</tr>
<tr>
<td>Dhk-18</td>
<td>23°49'34.00&quot; N, 90°29'06.00&quot; E</td>
<td>2.6</td>
<td>4.0</td>
<td>21.0</td>
<td>11</td>
<td>13.88</td>
</tr>
<tr>
<td>Dhk-19</td>
<td>23°42'15.00&quot; N, 90°30'04.00&quot; E</td>
<td>3.5</td>
<td>4.0</td>
<td>21.0</td>
<td>11</td>
<td>13.33</td>
</tr>
<tr>
<td>Dhk-20</td>
<td>23°43'02.00&quot; N, 90°27'35.00&quot; E</td>
<td>2.8</td>
<td>1.5</td>
<td>21.0</td>
<td>11</td>
<td>0.00</td>
</tr>
<tr>
<td>Dhk-21</td>
<td>23°44'26.00&quot; N, 90°25'46.00&quot; E</td>
<td>6.0</td>
<td>6.0</td>
<td>21.0</td>
<td>11</td>
<td>0.00</td>
</tr>
<tr>
<td>Dhk-22</td>
<td>23°47'45.00&quot; N, 90°24'28.00&quot; E</td>
<td>7.0</td>
<td>2.0</td>
<td>21.0</td>
<td>11</td>
<td>8.66</td>
</tr>
<tr>
<td>Dhk-23</td>
<td>23°49'20.00&quot; N, 90°25'01.00&quot; E</td>
<td>6.3</td>
<td>2.5</td>
<td>21.0</td>
<td>11</td>
<td>3.00</td>
</tr>
<tr>
<td>Dhk-24</td>
<td>23°51'24.00&quot; N, 90°23'02.00&quot; E</td>
<td>4.0</td>
<td>3.0</td>
<td>21.0</td>
<td>11</td>
<td>1.85</td>
</tr>
<tr>
<td>Dhk-25</td>
<td>23°52'36.00&quot; N, 90°23'30.00&quot; E</td>
<td>7.0</td>
<td>4.0</td>
<td>21.0</td>
<td>11</td>
<td>4.60</td>
</tr>
<tr>
<td>Dhk-26</td>
<td>23°49'42.00&quot; N, 90°22'06.00&quot; E</td>
<td>7.7</td>
<td>4.0</td>
<td>21.0</td>
<td>11</td>
<td>0.00</td>
</tr>
<tr>
<td>Dhk-27</td>
<td>23°46'37.00&quot; N, 90°21'42.00&quot; E</td>
<td>5.0</td>
<td>3.5</td>
<td>21.0</td>
<td>11</td>
<td>5.63</td>
</tr>
<tr>
<td>Dhk-28</td>
<td>23°46'29.00&quot; N, 90°26'13.00&quot; E</td>
<td>2.0</td>
<td>3.0</td>
<td>21.0</td>
<td>11</td>
<td>0.58</td>
</tr>
<tr>
<td>Dhk-29</td>
<td>23°42'36.25&quot; N, 90°23'23.06&quot; E</td>
<td>6.7</td>
<td>6.7</td>
<td>21.0</td>
<td>11</td>
<td>2.43</td>
</tr>
<tr>
<td>Dhk-30</td>
<td>23°43'52.10&quot; N, 90°21'45.76&quot; E</td>
<td>5.5</td>
<td>5.5</td>
<td>21.0</td>
<td>11</td>
<td>0.00</td>
</tr>
<tr>
<td>Dhk-31</td>
<td>23°46'52.61&quot; N, 90°22'26.33&quot; E</td>
<td>0.8</td>
<td>4.0</td>
<td>21.0</td>
<td>11</td>
<td>1.40</td>
</tr>
<tr>
<td>Dhk-32</td>
<td>23°49'51.89&quot; N, 90°22'34.25&quot; E</td>
<td>1.2</td>
<td>1.5</td>
<td>21.0</td>
<td>11</td>
<td>18.77</td>
</tr>
<tr>
<td>Dhk-33</td>
<td>23°49'20.28&quot; N, 90°21'12.06&quot; E</td>
<td>1.8</td>
<td>1.8</td>
<td>21.0</td>
<td>11</td>
<td>18.90</td>
</tr>
<tr>
<td>Dhk-34</td>
<td>23°40'58.37&quot; N, 90°26'55.68&quot; E</td>
<td>0.8</td>
<td>1.5</td>
<td>21.0</td>
<td>11</td>
<td>11.18</td>
</tr>
<tr>
<td>Dhk-35</td>
<td>23°42'21.85&quot; N, 90°26'39.77&quot; E</td>
<td>3.7</td>
<td>3.7</td>
<td>21.0</td>
<td>11</td>
<td>2.05</td>
</tr>
<tr>
<td>Dhk-36</td>
<td>23°44'52.66&quot; N, 90°26'26.48&quot; E</td>
<td>0.6</td>
<td>1.5</td>
<td>21.0</td>
<td>11</td>
<td>19.33</td>
</tr>
<tr>
<td>Dhk-37</td>
<td>23°43'32.95&quot; N, 90°24'18.72&quot; E</td>
<td>3.7</td>
<td>3.7</td>
<td>21.0</td>
<td>11</td>
<td>3.35</td>
</tr>
<tr>
<td>Dhk-38</td>
<td>23°44'40.96&quot; N, 90°22'31.33&quot; E</td>
<td>7.8</td>
<td>2.3</td>
<td>21.0</td>
<td>11</td>
<td>1.86</td>
</tr>
<tr>
<td>Dhk-39</td>
<td>23°53'16.04&quot; N, 90°23'14.39&quot; E</td>
<td>7.3</td>
<td>1.5</td>
<td>21.0</td>
<td>11</td>
<td>0.81</td>
</tr>
<tr>
<td>Dhk-40</td>
<td>23°41'37.00&quot; N, 90°27'31.20&quot; E</td>
<td>2.6</td>
<td>1.5</td>
<td>21.0</td>
<td>11</td>
<td>0.00</td>
</tr>
</tbody>
</table>
178

### Simplified Procedure for Determination of \( F_L \) against Liquefaction

The most widely used earthquake induced liquefaction potential analysis method is the Simplified Procedure, originally proposed by Seed and Idriss (1971); then modified and improved by Seed and Idriss (1982), Seed et al. (1985), etc.; and later adjusted, modified and evaluated by Seed et al. (2001), Youd et al. (2001), Idriss and Boulanger (2004), etc. The updated Simplified Procedure summarized by Youd et al. (2001) from the recent research findings for evaluating liquefaction resistance of soils under level ground was used in this study.

Seed and Idriss (1971) compared the Cyclic Stress Ratio (CSR) to the liquefaction resistance of the soil represented by the Cyclic Resistance Ratio (CRR). The cyclic stress ratio is the proportional to the peak ground acceleration \( (a_{\text{max}}) \). The cyclic resistance ratio for \( M_w = 7.5 \) earthquake \( (\text{CRR}_{7.5}) \) can be determined from a standard penetration resistance \( (N_1)_{60\text{cs}} \) of equivalent clean sand. A Magnitude Scaling Factor (MSF) is used to adjust \( \text{CRR}_{7.5} \) to determine CRR for other earthquake magnitudes. Although the details of the procedure are available from
Youd et al. (2001) the factor of safety ($F_L$) against liquefaction is defined in terms of CRR, CSR and MSF as follows.

$$F_L = (CRR_{7.5}/CSR)MSF \quad Eq. 6.1$$

**6.4.3 Liquefaction Potential Index (LPI)**

The factor of safety ($F_L$) is not a sufficient parameter for evaluation of liquefaction and its damage potential at any site. However, the thickness and depth of the liquefiable layer and the factor of safety are very important inputs for damage potential based on liquefaction. Since it was proposed by Iwasaki et al. (1978), the Liquefaction Potential Index (LPI) has been a very popular tool due to the inclusion of the thickness and depth of the liquefiable layer and the factor of safety as inputs. LPI was originally proposed by Iwasaki et al. (1982, 1978) to evaluate the potential for liquefaction to cause foundation damage. LPI assumes that the severity of liquefaction is proportional to (1) the thickness of the liquefied layer; (2) proximity of the liquefied layer from the ground surface; and (3) amount by which the factor of safety ($F_L$) is less than 1.0.

LPI is defined as:

$$L_I = \int_0^{20} F(z)W(z)dz \quad Eq. 6.2$$

$$F(z) = 1 - F_L \quad for \ F_L < 1.0 \quad Eq. 6.3 (a)$$

$$F(z) = 0 \quad for \ F_L \geq 1.0 \quad Eq. 6.3 (b)$$

$$W(z) = 10 - 0.5z \quad for \ z < 20m \quad Eq. 6.3 (c)$$

$$W(z) = 0 \quad for \ z > 20m \quad Eq. 6.3 (d)$$
where, \( z \) is the depth from the ground surface in meters.

Iwasaki et al. (1982, 1978) defined the effect of the factor of safety on liquefaction potential as linear from zero to one. It is obviously known that the layer having the smallest factor of safety may cause more damage on the surface. However, some studies were performed to overcome the use of linear relation for impact of the factor of safety on liquefaction potential (Sonmez 2003, Sonmez and Gokceoglu 2005). For this purpose, Sonmez and Gokceoglu (2005) introduce the probability of liquefaction equation (Eq. 6.4) proposed by Juang et al. (2003) based on the factor of safety to the LPI concept.

\[
P_L = \frac{1}{1 + \left(\frac{F_L}{0.96}\right)^{4.5}}
\]

Eq. 6.4

In addition, Sonmez and Gokceoglu (2005) rearranged the classification of liquefaction severity by considering Chen and Juang's (2000) probability of liquefaction classes based on \( P_L \) values. The details of the studies are available in Sonmez and Gokceoglu (2005). Although the valuable studies on introducing probability of liquefaction to LPI are available, in this study the original form of the LPI was followed to produce the liquefaction hazard map of Dhaka City by considering some validation of the threshold values of LPI in literature.

Iwasaki et al. (1982) identified LPI values of 5 and 15 as the lower bounds of "moderate" and "major" liquefaction, respectively, from SPT measurements at 85 Japanese sites subjected to six earthquakes. Toprak and Holzer (2003) also found similar results using 50 CPT sounding at 20 sites affected by the 1989 Loma Prieta (\( M_w = 6.9 \)) earthquake to correlate with surface manifestation of liquefaction. They ascertained that median values of LPI equal to 5 and 12 corresponded to the occurrence of sand boils and lateral spreading, respectively. The San Simson
earthquake also support the use of $LPI = 5$ as the threshold for surface manifestations of liquefaction (Holzer et al. 2005b). As mentioned before, LPI has the capability of the use for the spatial analysis of the liquefaction hazards because it allows to develop a two-dimesional representation of a three-dimensional phenomenon (i. e., $F_L$ vs. depth), which is ideal for mapping (Luna and Frost 1998), and it correlates well with liquefaction effects (Toprak and Holzer 2003).

As stated before, the 1885 Bengal Earthquake ($M=7.0$), which occurred about 50 km northwest from Dhaka City, is the most important earthquake for the city. In addition, in seismic zoning map of Bangladesh the peak horizontal ground acceleration for the city is proposed as 0.15g by BNBC, 1993. Therefore the peak horizontal ground acceteraion and magnitude of the earthquake were used as 0.15g and 7.0 ($M_w$), respectively to calculate the factor of safety which is required for the calculation of LPI for each borehole. The fines content of each SPT profile was available in the grain size distribution data. The groundwater level in most of the boreholes varies from 1.5 m to 7 m, except two boreholes having the groundwater levels were 15 m and 18 m from the ground surface (Table 6.2). LPI values for whole borehole locations were determined for an earthquake senario having a magnitude of 7.0 ($M_w$) and a peak horizontal ground acceleration of 0.15g (Table 6.2).

There was no record of liquefaction at the ground surface of Dhaka City during historical earthquakes, such as the 1885 Bengal earthquake, 1897 Great Indian Earthquake and 1918 Srimangal earthquake. It is possible that although soil liquafaction may occurred during these earthquakes events but no historical accounts have been documented. Toprak and Holzer (2003) indicated that surface demonstration of liquefaction generally occur where $LPI \geq 5$. The cumulative percentage of the LPI valus having $LPI \geq 5$ for each zone may be evaluated as the approximate
percentage of the surface area underlain by that zone which will show a surface display of liquefaction (Holzer et al. 2006). For example, 72% of the areas underlain by Zone 3 (the Holocene Alluvium and artificial fill) will exhibit surface demonstration of liquefaction for a scenario earthquake having a magnitude of 7.0 ($M_w$) and a peak horizontal ground acceleration of 0.15g. The percentages of areas that will show surface effects for each zone of Dhaka City are shown in Figure 6.3.

![Cumulative frequency distributions of Liquefaction Potential Index (LPI) for three zones of Dhaka City. Number of SPT profiles used in each zone is shown in parentheses of the legend (Rahman et al. 2015c). Don’t require permission of Elsevier as the author of this article.]

Figure 6.3 Cumulative frequency distributions of Liquefaction Potential Index (LPI) for three zones of Dhaka City. Number of SPT profiles used in each zone is shown in parentheses of the legend (Rahman et al. 2015c). Don’t require permission of Elsevier as the author of this article.

### 6.5 Liquefaction Hazard Map

LPI values of fifty three (53) borehole locations are shown on the map and then contour lines of 0, 5, 10, 15 LPI value have been drawn on the map to evaluate the liquefaction hazard at a
specific location of the city. Liquefaction hazards are categorized based on LPI values, i.e., very low for LPI = 0, low for 0 < LPI ≤ 5, high for 5 < LPI ≤ 15 and very high for LPI >15 according to the method proposed by Iwasaki et al. (1982).

The surface geological units of Dhaka City have been divided into three liquefaction hazard zones based on the cumulative frequency distribution of LPI of each unit to define liquefaction probability (Error! Reference source not found. and Figure 6.4). The zones are Zone 1 (Pleistocene terrace deposit), Zone 2 (Holocene Alluvial valley fill deposit and Holocene terrace deposit) and Zone 3 (Holocene Alluvium and artificial fill). Zone 1 and Zone 3 occupy large areas that contain twenty three (23) and nineteen (19) LPI values, respectively. Zone 2 covers a small area having eleven (11) LPI values. Therefore, the distribution of LPI values in each zone was considered as uniform, which is important for cumulative frequency distribution analysis. The liquefaction probability is quantified as percentage of the cumulative frequency distribution at LPI = 5 for each zone. The map indicates that 8%, 50% and 72% areas of Zone 1, Zone 2 and Zone 3, respectively, will show surface manifestation of liquefaction.

6.6 Discussions

The liquefaction hazard map of Dhaka City offers a quantitative approach for mapping liquefaction susceptibility. The liquefaction potentiality of a specific location has been predicted by showing LPI values at each borehole location and by drawing contour lines of equal LPI values. The liquefaction probability has been evaluated in terms of cumulative percentage of LPI in each hazard zone. The probability map can either be used to estimate the area of coverage that is expected to show surface manifestations of liquefaction or the conditional probability of liquefaction at any specific zone (Holzer et al. 2006).
Figure 6.4 Liquefaction hazard map of Dhaka City. Liquefaction hazard have been categorized as very low for $LPI = 0$; low for $0 < LPI \leq 5$; high for $5 < LPI \leq 15$ and very high for $LPI > 15$. The 8%, 50% and 72% of areas in Zone 1, Zone 2 and Zone 3 respectively, will show surface effects of liquefaction for a scenario earthquake of $M=7$ having peak horizontal ground acceleration of 0.15g (Rahman et al. 2015c). Don’t require permission of Elsevier as the author of this article.
The subsurface soil of Zone 1 up to the depth of 20 m is formed of Pleistocene yellowish brown to reddish brown clayey soils and Plio-Pleistocene yellowish brown sandy soils. The clayey soils are stiff to hard and the sandy soils are medium dense to very dense in nature. The liquefaction potential of this zone varied from very low to low having LPI values from 0 to 4.6, except one borehole having LPI value of 8.66. Eight percent (8%) of the area of Zone 1 will show surface manifestations of liquefaction (Figure 6.4).

Zone 2 consists of Holocene alluvial valley fill and terrace deposits. The valley fill deposits are composed of dark grey to grey clayey and sandy soils deposited in the valleys and depressions of the Pleistocene terrace. The terrace deposits include natural levees, point bars and channel bars of the present rivers, which consist of grey silty and sandy soils. LPI values of this zone varied from 0 to 19.33 and the liquefaction potentials of this zone are categorized from very low to very high. Fifty percent (50%) of the area of Zone 2 will exhibit surface effects of liquefaction.

Zone 3 includes Alluvium and artificial fills consisting of grey clayey, silty and sandy soils. The LPI values of this zone varied from 0 to 18.9 having the liquefaction potentials from very low to very high. Seventy two percent (72%) of the area of Zone 3 will exhibit surface effects of liquefaction.

The groundwater level has a great influence in the calculation of LPI using Simplified Procedure. In the present study, most of the SPT were conducted during the dry season. The groundwater levels remained lower during the dry season than any other season of the year. It is most likely that the LPI value and the percentage of an area in each zone showing surface manifestation of liquefaction will increase if the groundwater levels are considered based on the monsoon of the year.
As the liquefaction records were not documented in Dhaka City as well in Bangladesh, LPI values were not calibrated with actual liquefaction phenomena. However, a large percentage of the area (72%) in Zone 3 (Alluvium and artificial fill) showing surface manifestation of liquefaction is consistent with the extensive liquefaction of loose fills during the 1995 earthquake in Kobe, Japan (Hamada et al. 1995). High liquefaction hazard in Alluvium and artificial fills and less liquefaction hazard in the Pleistocene deposit are also reported by Holzer et al. (2006).

6.7 Summary

Liquefaction hazard map of the subsurface geological materials of Dhaka City was prepared for a scenario earthquake having a magnitude of 7.0 (Mw) and a horizontal peak ground acceleration of 0.15g. The LPI value of each borehole location are shown in the map and then the LPI contour lines of 0, 5, 10, 15 have been drawn to define the liquefaction potential of a specific location. The liquefaction potential of the city varied from very low to very high. The geological units of Dhaka City have been grouped into three liquefaction hazard zones, such as Zone 1, Zone 2 and Zone 3 based on cumulative frequency distribution of LPI in each zone. The cumulative percentages of LPI indicate that 8%, 50% and 72% of the areas of Zone 1, Zone 2 and Zone 3, respectively, will show the surface effects of liquefaction. LPI value of each SPT profile has been calculated from the factor of safety estimated using the Simplified Procedure.

The uncertainties associated with this study are due to a small number of SPT profiles, groundwater level variation and delineation of surface geological unit boundaries. These uncertainties could be overcome by including a large number of SPT profiles, measuring groundwater levels in different seasons of the year and delineating the boundaries of the surface geological units accurately for future application of the method. Selection of appropriate peak
horizontal ground acceleration (PGA) for the scenario earthquake is also an uncertainty in the study. The PGA can be estimated by using appropriate ground motion prediction equation taking average distance of the study area from the fault of the scenario earthquake and average soil conditions. As a first time approach, the map prepared in the present study using LPI is considered as a preliminary liquefaction hazard map of Dhaka City. Finally, this type of regional hazard map can be used as additional guidelines for future planning and development of the city with the site specific seismic hazard analysis.
Chapter 7: Conclusions and Recommendations

7.1 Summary and Conclusions

In seismically active regions, the cities of developing countries, such as Dhaka in Bangladesh, are more vulnerable than those of the developed countries due to: (1) high population density, (2) unplanned urbanization, (3) non-engineered construction practices, (4) inadequate knowledge of the seismic design of structures, (5) ignorance of building codes, and (6) poor monitoring system of the concerned urban authorities during the construction of structures. To reduce the seismic risk, seismic hazard analysis is required to design structures, prepare regional seismic hazard maps to use in building codes, and develop emergency response system. Seismic hazard analysis includes deterministic or probabilistic seismic hazard analysis (DSHA or PSHA), site response analysis, and liquefaction potential evaluation. In the present study, the main objective of this research is to perform probabilistic seismic hazard analysis with nonlinear site response including liquefaction hazard evaluation for deep and soft sedimentary deposits in Bangladesh that is located close to the seismically active convergent plate boundary between the Eurasian and the Indian plates.

The seismic source characterization is one of the important components of seismic hazard analysis. A declustered and complete earthquake catalog is required to characterize the seismic sources. Therefore, a declustered and complete earthquake catalog was prepared in present study. The seismic sources in the study region were characterized as background seismicity, regional seismicity, crustal fault, and subduction zone sources using the earthquake catalog, and geodetic strain rate across different crustal faults and subduction zones, as geologic slip rate for the crustal faults and subduction zones are not available in this region.
Probabilistic seismic hazard analysis (PSHA) was performed to estimate the seismic ground motion at the bedrock condition \( (V_{s}^{30} = 760 \text{ m/s}) \) using the ground motion prediction equations (GMPEs) as a function of the earthquake magnitudes of the seismic sources and the distances from the sources. The GMPEs of the NGA project developed in 2014 for shallow crustal earthquakes were used for background seismicity and crustal fault sources. The GMPEs of the subduction zones were used for deep crustal and subduction sources. The PGA and SA at different spectral periods were estimated at bedrock condition for Bangladesh at a grid size of 0.25°. The PGA and SA maps for different spectral periods were prepared for Bangladesh including uniform hazard spectra (UHS) for 10% and 2% probability of exceedance in 50 years for nine major cities in Bangladesh.

Seismic site characterization is required for site response analysis of an area. For seismic site characterization of Dhaka City, the \( V_{s}^{30} \) of the geological materials was predicted using the relationship between the \( V_{s} \) of the downhole seismic (DS) and the standard penetration test blow count (SPT-N). The \( V_{s}^{30} \) was predicted using the relationship between the \( V_{s}^{30} \) and Holocene soil thickness. Then, the \( V_{s}^{30} \) map for Dhaka City was prepared from the \( V_{s}^{30} \) values at a grid size of 30 m. It is observed that the estimated and predicted \( V_{s}^{30} \) results have a good match. As the \( V_{s} \) estimations using direct methods, such as downhole seismic, surface waves, are costly and need expertise knowledge and sophisticated software, the \( V_{s}^{30} \) can easily be predicted using the correlation between the \( V_{s} \) and SPT-N. The \( V_{s}^{30} \) can also be predicted using the relationship between the \( V_{s}^{30} \) and the Holocene soil thickness. Therefore, the correlation between the \( V_{s} \) and SPT-N, and relationship between the \( V_{s}^{30} \) and Holocene soil thickness can be used for the \( V_{s}^{30} \) estimation in Dhaka City where most of the organizations and companies have no geophysical instruments and expertise personnel to conduct geophysical surveys for near-surface shear wave velocity estimations.
As a simplified procedure, the ground motion at the ground surface is predicted by multiplying the bedrock ground motion with the site amplification factor that is estimated using the $V_s^{30}$. The state-of-practice is to perform one-dimensional site response analysis for the prediction of surface ground motion using the dynamic properties of the soils above the bedrock and propagating the bedrock ground motion at the base of the soil profile. It was observed that the ground motion prediction at the ground surface using one-dimensional site response analysis improves the accuracy of the surface ground motion.

Spectral matching has been performed to modify the acceleration time history of real earthquake by matching its response spectrum with the target response spectrum (bedrock) that was generated using the PSHA. Twenty-eight horizontal acceleration time histories of nine earthquakes, whose response spectra were similar to the target response spectra of 10% and 2% probability of exceedance in 50 years, were downloaded from the PEER NGA WEST2 database to use in this study. The acceleration time histories of the matched response spectra were used for one-dimensional site response analysis.

The dynamic properties of the soils were modeled by using the normalized modulus reduction and material damping curves that were proposed by Darendeli (2001). The initial effective vertical stress, initial coefficient of effective lateral earth pressure, plasticity index, over consolidation ratio, loading frequency, and number of loading cycles were required to use the relationships of Darendeli (2001). The general quadratic/ hyperbolic model (GQ/H) with non-Masing reloading-unloading hysteretic formulation that was proposed by Groholski et al. (2016) was used to fit modulus reduction and material damping curves of the sandy and clayey soils of the profiles with the reference curves of Darendeli (2001) based on the specific shear strength. The GQ/H model
has the capacity to capture the large strain shear stress behavior of the soils to accurately estimate the surface ground motion at large strains.

One-dimensional site response analysis includes linear, equivalent-linear, and non-linear models. As the dynamic properties of soils are nonlinear at high strain, the surface ground motion can be estimated more accurately using the nonlinear response analysis. Therefore, one-dimensional nonlinear site response analysis was performed to accurately estimate the surface ground motion in Dhaka City where the bedrock \((V_s = 760 \text{ m/s})\) is overlain by the soft sedimentary deposits of more than 30 m thick. In Dhaka City, the \(V_s^{30}\)-based site amplifications overestimate the ground motion at short spectral periods and underestimate the ground motion at long periods.

In areas of thick and soft Holocene deposits, the long period seismic waves of the far-field earthquakes will amplify and the resonance will occur with the natural periods of the high-rise buildings, and consequently, the damage to high-rise buildings will be increased in these areas. In areas where the shallow valleys of the Pleistocene Terrace were filled artificially with sand and clay (3-5 m thick), the short period seismic waves will amplify and the resonance will occur with the natural periods of the low-rise buildings, and consequently, the damage to the low-rise buildings will be increased in these areas.

During earthquake, liquefaction can be a potential seismic hazard in the Holocene loose and poorly graded sands and low plastic silts existed at shallow depth \(< 20 \text{ m}\) below the water table. As more than 50% areas of Dhaka City are covered by thick layers of the Holocene sandy and silty soils, liquefaction hazard map for near-surface geological materials of Dhaka City was prepared for a scenario earthquake having a magnitude of 7.0 \((M_w)\) and a horizontal peak ground acceleration of 0.15g. The LPI value of each borehole location are shown in the map and then the
LPI contour lines of 0, 5, 10, 15 have been drawn to define the liquefaction potential of a specific location. The liquefaction potential of the city varied from very low to very high. The geological units of Dhaka City have been grouped into three liquefaction hazard zones, such as Zone 1, Zone 2 and Zone 3 based on cumulative frequency distribution of LPI in each zone. The cumulative percentages of LPI indicate that 8%, 50% and 72% of the areas of Zone 1, Zone 2 and Zone 3, respectively, will show the surface effects of liquefaction. The LPI value of each SPT profile was calculated from the factor of safety estimated using the simplified procedure.

7.2 Originality and Contributions

The specific contributions of this research are summarized below:

- A declustered and complete earthquake catalog is prepared from the earthquake records of the study regions for the period from 1762 to 2016.
- The seismic sources of the study regions are modeled as background seismicity, regional seismicity, crustal fault, and subduction zone sources.
- The uncertainties in estimating the ground motion using the source models and ground motion prediction equations (GMPEs) are accounted using the logic tree approach.
- The probabilistic ground motion maps for Bangladesh are prepared for 10% and 2% probability of exceedance in 50 years.
- The $V_s^{30}$ (time-averaged shear wave velocity in the top 30 m) map is prepared for Dhaka City using the relationship between the shear wave velocity and the Holocene soil thickness.
The site response analysis for deep and soft sedimentary deposits in Dhaka City is performed using $V_s^{30}$-based site coefficients and one-dimensional linear, equivalent-linear, and nonlinear approaches.

The liquefaction hazard map for Dhaka City is prepared using liquefaction potential index (LPI) and cumulative frequency distribution of LPI of different surface geological units, which is an excellent approach to evaluate the liquefaction hazard quantitatively and spatially.

These seismic hazard maps of the present study can be used for seismic risk management in Bangladesh.

7.3 Limitations and Recommendations

The following recommendations are suggested for future research:

- The seismic source models should be updated using geological slip rates that are estimated through paleoseismological studies to update the seismic hazard maps of the present study.
- The ground motion prediction equations (GMPEs) should be developed using the ground motion data of the study regions.
- The relationship between the shear wave velocity and the Holocene soil thickness should be updated using more shear wave velocity data of direct methods, such as downhole seismic investigation.
- Shear wave velocity should be measured up to the depth of the bedrock using direct methods to estimate the site effects of the deep and soft sedimentary deposits using one-dimensional nonlinear site response analysis.
- The results of the nonlinear site response for the deep and soft sedimentary deposits should be validated with the site response of the similar deposits during an earthquake in the similar seismotectonic setting.

- The software of the nonlinear site response analysis should be updated to capture the natural phenomena of the subsurface geological materials to improve the accuracy of the site effect estimations.

- The liquefaction hazard map should be updated using more boreholes of the standard penetration test (SPT).
**Bibliography**


BSSC. 1994. NEHRP recommended provisions for seismic regulations for new buildings. Part 1


Campbell, K.W., and Bozorgnia, Y. 2008. NGA ground motion model for the geometric mean horizontal component of PGA, PGV, PGD and 5% damped linear elastic response spectra for periods ranging from 0.01 to 10 s. Earthquake Spectra, 24(1): 139–171. doi:10.1193/1.2857546.

Campbell, K.W., and Bozorgnia, Y. 2014. NGA-West2 ground motion model for the average horizontal components of PGA, PGV, and 5% damped linear acceleration response spectra. Earthquake Spectra, 30(3): 1087–1114. doi:10.1193/062913EQS175M.


Fabbrocino, S., Lanzano, G., Forte, G., Santucci de Magistris, F., and Fabbrocino, G. 2015. SPT blow count vs. shear wave velocity relationship in the structurally complex formations of the


Kim, B., and Hashash, Y.M.A. 2013. Site response analysis using downhole array recordings during the March 2011 Tohoku-Oki earthquake and the effect of long-duration ground


Petersen, M.D., Zeng, Y., Haller, K.M., Mccaffrey, R., Hammond, W.C., Bird, P., Moschetti, M.,


