# ESTIMATE OF DAMAGE OF BURIED GAS PIPELINES IN DHAKA CITY DUE TO AN EARTHQUAKE

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**DHAKA, BANGLADESH**

**December, 2013**

# ESTIMATE OF DAMAGE OF BURIED GAS PIPELINES IN DHAKA CITY DUE TO AN EARTHQUAKE

**BY**

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A project report submitted to Department of Civil Engineering of
Bangladesh University of Engineering and Technology, Dhaka
In partial fulfillment of the requirements for the degree

of

**MASTER IN CIVIL ENGINEERING (GEOTECHNICAL)**

December, 2013

# DECLARATION

I hereby certify that the research work embodied in this project report has been performed by the author under the supervision of Dr. Mehedi Ahmed Ansary, Professor of the Department of Civil Engineering, BUET. Neither this thesis nor any part of it has been submitted or being currently submitted elsewhere for any other purpose (except for publications).

December, 2013 Mohammad Sayeed Hossain

The thesis titled **"Estimate of Damage of Buried Gas Pipelines in Dhaka City Due to an Earthquake**" submitted by Mohammad Sayeed Hossain, Roll no: 040204217P, Session: April, 2002 to the Department of Civil Engineering, Bangladesh University of Engineering and Technology, has been accepted as satisfactory in partial fulfillment to the requirement for the degree Master in Civil Engineering (Geotechnical).

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## ACKNOWLEDGEMENT

All praises to Allah, the merciful and kind.

The author expresses his heartiest gratitude to his supervisor Dr. Mehedi Ahmed Ansary, Professor of Civil Engineering Department BUET for his guidance, invaluable suggestion and affectionate and continuing encouragement at different stages of the study.

The author also extends his gratefulness to Dr. Abu Siddique, Professor, Department of Civil Engineering, BUET and Dr. Md. Jahangir Alam, Associate Professor, Department of Civil Engineering, BUET for their constructive suggestions.

## ABSTRACT

This thesis focuses on the damage analysis of buried gas supply pipeline of Dhaka City subject to earthquake effects. Damage prediction of gas pipelines due to earthquake involves seismic microzonation of Dhaka city and determination of the length of water supply pipeline. In this process already developed seismic microzonation map of Dhaka city is used and the available map of gas supply pipeline network of "TITAS gas Transmission and Distribution Company Ltd" is digitized to get the length of pipelines with the help of GIS software.

On the basis of intensity the whole Dhaka city has been divided into three different zones. Out of total area of 135 sq.km 88 sq.km is (65%) of intensity VIII, 39 sq.km is (29%) of intensity IX and remaining 9 sq.km is (6%) of intensity X. From the digitized pipeline network, based on 1988 "TITAS gas Transmission and Distribution Company Ltd", the length of 20mm, 25mm, 50mm, 75mm, 100mm, 150mm, 200mm, 250mm and 300mm diameter pipe is found to be 11.3 km, 121.28 km, 175.16 km, 40.69 km, 28.35 km, 28.52 km, 42 km, 1.5 km, and 3.49 km respectively. Again from the intensity based pipeline network it is found that 376 km pipe falls in the zone of intensity VIII, 67.31 km falls in the zone of intensity IX and 22.23 km falls in the zone of intensity X irrespective of pipe diameter. A selection step is followed to estimate peak ground acceleration (PGA) and peak ground velocity (PGV) to determine the pipeline damage rate. Existing empirical relations based on peak ground acceleration such as Katayama (1975), Isoyama and Katayama (1982), O'Rourke (1998) and Isoyama (2000) for the prediction of earthquake-induced pipeline damage are reviewed. Similarly empirical relations based on peak ground velocity such as Eidinger et al. (1995, 1998), O'Rourke and Ayla (1993), Isoyama (2000) and O'Rourke (2001) also reviewed. Finally using above relations and selected peak ground acceleration and peak ground velocity damage rate of pipelines is determined.

Pipeline damage rate is expressed in number of repairs per unit length of pipe. In case of PGA based analysis total number of repairs for all intensities is 97 within a total pipe length of 465 km. Out of which 24 numbers of repairs required for 376 km pipelines, 28 numbers of repairs required for 67.31 km pipelines and 45 numbers of repairs for 22.23 km. For PGV base analysis total number of repairs for all intensities is 182 within total pipe length of 465 km. Out of which 87 numbers of repairs required for 376 km pipelines, 50 numbers of repairs required for 67.31 km pipelines and 45 numbers of repairs for 22.23 km.

# CHAPTER ONE
# INTRODUCTION

## 1.1 GENERAL

Earthquake can cause extensive damage to buried gas pipelines which is one of six categories of infrastructure grouped under the heading “lifelines”, resulting in disruption of essential services for the whole community. Since the mid-70s, there have been advances in the development of models to better understand how earthquakes affect buried pipelines. These natural events can cause damage due to two phenomena: seismic wave propagation and permanent ground deformation. The combined effect of both phenomena in pipeline damage estimation is a subject still complex to address, especially if the objective is to estimate damage due to future earthquakes. In this work, the damage assessment methods only consider the impact of seismic wave propagation. The effects of permanent ground deformation phenomena, like ground subsidence, landslides, and ground rupture are omitted.

Gas distribution systems are one of six broad categories of infrastructure grouped under the heading 'lifelines' (O'Rourke, 1998). Together with water distribution, electric power, liquid fuels, telecommunications, transportation and wastewater facilities they provide the basic services and resources upon which modem communities have come to rely, particularly in the urban context. Disruption of these lifelines through earthquake damage can therefore have a devastating impact, threatening life in the short term and a region's economic and social stability in the long term.

Fire following earthquakes due to leakage of gas has resulted in a considerable loss of property during the history. In San Francisco earthquake (1906) the estimated loss due to a widespread fire was about 250 million dollar of its time. The case of Tokyo earthquake of 1923 was much worse where the fire-induced loss shared 77% of the total. In recent times, Loma Prieta (1989), Northridge (1994) and Kobe (1995) earthquakes are new experiences in which ignition of fire was generally because of damages in old gas networks. It is therefore justifiable to concentrate on the performance of city gas networks, among other lifelines, when assessing vulnerability of urban areas to fire following an earthquake.

Like other field of earthquake engineering lifeline earthquake engineering is not so old. Its formal recognition came in the 1970's with the establishment in the United States of ASCE's Technical Council on Lifeline Earthquake Engineering (Duke & Matthiesen, 1973). In 1975, Council Members, C .M. Duke and D.F. Moran commented that the state-of-the-art for lifeline earthquake engineering was l0 to 20 years behind that of buildings (Duke & Moran‘1975). A concerted research effort since then has made up much of the lost ground, but many challenges remain.

Fragility functions are typically the tools most used to assess seismic damage in buried pipelines. These functions relate pipeline damage with seismic intensity. Pipeline damage is generally expressed as a linear pipe repair density. Seismic intensity is usually quantified through a seismic parameter. There are many seismic parameters used as arguments of fragility functions.

## 1.2 OBJECTIVES OF STUDY

The major objective of this study is as follows:

1. To develop a database of buried gas pipeline (based on gas network map of “TITAS Gas Transmission and Distribution Company Ltd”) of Dhaka City using GIS.
2. To assess the vulnerability of buried gas pipelines due to earthquake.

## 1.3 THESIS OUTLINE

In **Chapter Two** different earthquake effects, seismic response of buried pipelines, factors affecting earthquake vulnerability of pipelines are studied. Different existing empirical fragility relations for buried pipelines are reviewed. Background information of the seismic environment prevailing in Bangladesh as a part of the evaluation of seismic hazard has also been reviewed. Important tectonic features of Bangladesh seismic zoning map, geotechnical characteristics and seismic microzonation map of Dhaka city are described.

**Chapter Three** deals with the development of pipeline damage database with the GIS software, selection of peak ground acceleration (PGA) and peak ground velocity (PGV) values from intensity. Estimation of damage based on existing methods were done and presented in this chapter. Monetary loss estimation is also presented in this chapter.

In **Chapter Four** conclusions from this study and recommendations for further areas of study are presented.

# CHAPTER TWO
# LITERATURE REVIEW

## 2.0 GENERAL

Earthquake is the trembling or shaking movement of the earth's surface. Most earthquakes are minor tremors, while larger earthquakes usually begin with slight tremors, rapidly take the form of one or more violent shocks, and end in vibrations of gradually diminishing force called aftershocks. Earthquake is a form of energy of wave motion, which originates in a limited region and then spreads out in all directions from the source of disturbance. It usually lasts for a few seconds to a minute. The point within the earth where earthquake waves originate is called the focus, from where the vibrations spread in all directions. They reach the surface first at the point immediately above the focus and this point is called the epicenter. It is at the epicenter where the shock of the earthquake is first experienced. On the basis of the depth of focus, an earthquake may be termed as shallow focus (0-70 km), intermediate focus (70-300 km), and deep focus (>300 km). The most common measure of earthquake size is the Richter‘s magnitude. The Richter scale uses the maximum surface wave amplitude in the seismogram and the difference in the arrival times of primary and secondary waves for determining magnitude. The magnitude is related to roughly logarithm of energy. Earthquakes originate due to various reasons, which fall into two major categories non-tectonic and tectonic. The origin of tectonic earthquakes is explained with the help of ‘elastic rebound theory‘. Earthquakes are distributed unevenly on the globe. However, it has been observed that most of the destructive earthquakes originate within two well-defined zones or belts namely, ‘the circum-Pacific belt‘and ‘the Mediterranean-Himalayan seismic belt‘(Banglapedia, 2004).

## 2.1 EARTHQUAKE EFFECTS

The direct effects of earthquakes are surface faulting and ground shaking. Secondary or "collateral" effects include liquefaction, landslides, densification and tsunami. Earthquake effects on buried pipelines are best understood by considering the displacements induced in the surrounding soil. Damage may he caused by transient ground deformation (GDt), or permanent ground deformation (GDp), or a combination of the two. O'Rourke (1998) defines the distinction between these two effects *“GDp involves the irrecoverable movement of the ground that often is the result of ground failure, but also may result from modest levels of* *volumetric strain and shear distortion. GDt involves ground waves and soil strains associated with strong shaking. Although ground crack and fissures may result from GDt during ground shaking*." All of the collateral earthquake effects, plus faulting, can give rise to permanent ground deformation.

The relative impact of different effects on buried lifelines varies from earthquake to earthquake. Transient effects are common to all earthquakes and are felt over a wide geographical area and associated pipeline damage tends to be spread over the whole of a gas supply system. Resulting damage rates (in terms of breaks per unit length of pipe) are relatively low but the total number of pipe breaks can be high. Surface fault rupture and collateral earthquake effects can give rise to very high ground strains.

## 2.2 FAULTING

Most earthquakes occur as a result of the buildup of stresses at tectonic plate boundaries. When these stresses exceed the rock's ability to resist them, rupture occurs along a fault, releasing the stored strain energy in the form of seismic waves and heat. The fault rupture usually coincides with pre-existing discontinuity in the Earth's crust. The extent of faulting is linked closely with earthquake magnitude. Large earthquakes can produce faults of several hundred kilometers length with widths of tens of kilometers and offsets of several meters.

In most earthquakes, the fault rupture plane does not have a surface expression (blind faulting) (Reiter, 1990). A surface fault trace is usually only observed for large earthquakes occurring at shallow depth. The extent of surface faulting depends chiefly on the length and amount of offset of the subsurface faulting, the attitude of the fault plane, the direction of the fault movement and the type and thickness of the surface geology (Taylor and Cluff, 1977). Faults can be classified according to the movement of the two sides of the fault relative to each other (Figure 2.1). Faulting is termed strike-slip when the movement is predominantly horizontal. It is known as dip-slip when the movement is predominantly in the direction of dip of the fault plane. Dip-slip movement where the horizontal component is in compression is called reverse faulting. Where the horizontal component is extensional, the faulting is termed normal. A combination of dip-slip and strike-slip movement is referred to as oblique faulting.

Not all fault-like features observed at the surface are related to tectonic rupture. Fractures may be formed by ground shaking, landslides. As illustrated in Figure 2.1, fault-induced ground-strain is most severe at the intersection between the fault plane and the ground surface. However, the crustal deformation that accompanies earthquake faulting can be significant at considerable distances from the surface rupture.

The large permanent ground deformations associated with faulting can present a very severe hazard to structures on or near to active faults. Where potentially active fault can be identified “no build'" zones can he designated to avoid unnecessary damage in the event of an earthquake. In the ease of water pipelines crossing active fault is often unavoidable, since pipeline location is dictated by the locations of supply and demand areas. It is therefore useful to be able to estimate the amount of permanent ground displacement that might occur in the event of an earthquake of a given magnitude on a particular fault.

Numerous studies have been carried out to investigate the connection between earthquake magnitude and various characteristics of the fault rupture. Wells and Coppersmith (1994) compiled a worldwide database of 244 earthquakes covering the moment magnitude range $5.6 \leq M_W \leq 8.1$ .Observed fault displacements ranged from 0.05 - 8.0 m for strike-slip faults, 0.08 - 2.1 m for normal faults and 0.06 - 1.5 m for reverse faults. From this database empirical relationships were derived among magnitude, rupture length, rupture width, rupture area and surface displacement. These expressions can be used to predict likely fault rupture characteristics given a specific magnitude of event. Of most interest for the prediction of pipeline damage are expressions for expected surface fault displacement as a function of magnitude:

$$\text{Log } D = C_1 + C_2 M_w \qquad (2.1)$$

Where:

D is the average surface fault displacement (m),

$M_w$ is the moment magnitude,

$C_1$ and $C_2$ are coefficients derived from the regression; Values for different categories of fault slip type are presented in Table 2.1.

**Table 2.1: Regression coefficients for different categories of fault slip type for use in Equation 2.1 (After Wells and Coppersmith, 1994)**

| **Fault slip type** | **$C_1$** | **$C_2$** | **Standard Deviation** | **Correlation coefficient** | **Magnitude range** |
|---|---|---|---|---|---|
| Strike-slip | -6.32 | 0.90 | 0.28 | 0.89 | 5.6 – 8.1 |
| Reverse | -0.74 | 0.08 | 0.38 | 1.10 | 5.8 – 7.4 |
| Normal | -4.45 | 0.63 | 0.33 | 0.64 | 6.0 – 7.3 |
| All | -4.80 | 0.69 | 0.36 | 0.75 | 5.6 – 8.1 |

Even for earthquakes without a surface fault expression, co seismic strains induced in the epicenter region may still be large enough to cause damage to buried pipelines. The response of a buried pipe to surface faulting depends to a large extent on its orientation with respect to the fault. Bending, buckling due to axial compression or pull-out due to axial extension are all possible responses.

### 2.3 GROUND SHAKING

Ground shaking is caused by two different kinds of seismic waves: body waves and surface waves. Body waves are generated by earthquake faulting and are responsible for the radiation of seismic energy from the rupture zone at depth to the surface of the Earth. Body wave disturbances are of two types: P-waves (primary waves) and S-waves (secondary) (Figure 2.1). P-waves (compression waves) are characterized by disturbance parallel to the direction Of wave propagation whereas waves (shear waves) cause a disturbance perpendicular to the direction of travel. The direction of particle movement can be used to divide S-waves into two components: SV (vertical) and SH (horizontal).

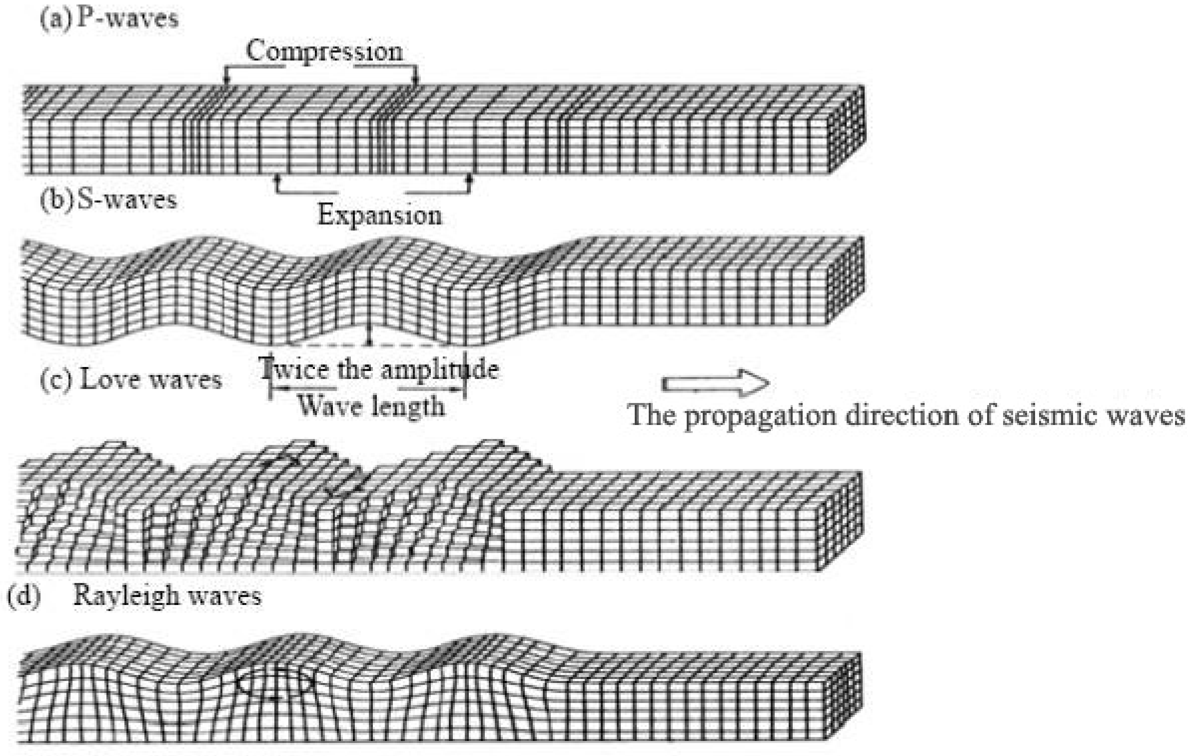



**Figure 2.1 Deformation produced by body waves (after Bolt, 1993)**

The interaction of body waves with the surface of the Earth causes surface waves, the most important of which for engineering purposes, are R-waves (Rayleigh waves) and L-waves (Love waves) (Figure 2.1). For R-waves, the particle motion traces an ellipse in a vertical plane, the size of the ellipse decreasing with depth below the ground surface. R-waves also have a horizontal component, which is parallel to the direction of propagation. For L-waves, the particle motion is in the horizontal plane, perpendicular to the direction of propagation, with the amplitude decreasing with depth below the ground.

Both types of waves are of interest when considering the response of buried pipelines to seismic ground shaking. For body waves, only S-waves are normally considered as they carry more energy than P-waves. In the case of surface waves, it is R-waves which are most important, inducing axial strains in buried pipelines of much more significance than the bending strains induced by L-waves (O'Rourke & Liu, 1999). Seismic wave propagation theory indicates significant differences between the transient ground motions associated with body waves and those associated with surface waves.

In order to predict earthquake damage to pipeline systems or design a new pipeline for earthquake resistance, it is therefore important to define the predominant effects at the site or region of interest.

O'Rourke (1998) identifies four distinct categories of transient ground shaking effects of relevance to pipelines and other lifelines:

a) Travelling ground waves.
b) Surface-wave generation in large sedimentary basins (typically several kilometers wide with depths less than 1 km). Significant long-period motions are caused by surface waves generated by the trapping and focusing of obliquely incident S-waves in large sedimentary basins.
c) Vibration of sediments in relatively narrow valleys (several hundreds of meters wide by several tens of meters deep). For smaller basins, mass shear deformation in the valley sediments is more important than wave scattering effects. In such cases, large strains are induced near valley margins.
d) Liquefaction-induced ground oscillation. The last three phenomena are examples of long- period ground motion. it is only large earthquakes, with extended fault ruptures that give sufficiently strong excitation in the long-period range to be of engineering interest.

### 2.3.1 EFFECTS OF SURFACE TOPOGRAPHY

Destructive earthquakes have often caused higher concentrations of building damage on the tops of hills than at their bases. Instrumental and theoretical evidence supports the hypothesis that surface topography can significantly modify the amplitude and frequency content of ground motion. However, few systematic investigations have been conducted into this phenomenon and there is, as yet, no general consensus.

Geli et al. (1988) made a compilation of eleven individual studies of topographic effects, including both instrumental and theoretical results. Their conclusions are summarized below:

a) The amplification of ground motions on a hilltop and its de-amplification at the root of a hill is supported, at least qualitatively, by observations and theory. In general, amplification is more pronounced for the horizontal components of ground motion than for the vertical component.
b) Amplification on a hilltop is roughly related to the sharpness of the topography. The steeper the terrain, the greater the amplification at the peak.
c) The frequencies most significantly modified by surface topography are those which correspond to wavelengths comparable to the horizontal dimension of the topographic feature.

In view of the current lack of understanding of topographic modification of earthquake ground motion, Bard & Riepl-Thomas (2000) suggest the need for more detailed studies of this phenomenon involving dense arrays of strong-motion instruments and detailed geotechnical characterization of the study area.

**2.3.2 Effects of Soft Surface Layers**

It is well recognized that earthquake induced ground motions are strongly influenced by the nature of near-surface geological materials. Earthquake damage to structure situated on soft Soil is consistently greater than damage to structures on firm soil or bedrock outcrops.

The amplification of ground motion in sot soils is caused by the trapping of seismic waves within the soft layers because of the contrast in properties between the soft overlying material and the firmer underlying bedrock. In the simplest case of horizontally layered sediments, this trapping affects only the vertical propagation of body waves. However any real soil structure will also have lateral heterogeneities which trap horizontally propagating surface waves. The trapped waves interfere with each other, giving rise to resonance effects whose spatial distribution and frequency content depend on the characteristics of the incident seismic wave form and the geometrical and mechanical characteristics of the geological structure.

Resonance effects at a given strong-motion measurement location can he identified by considering frequency domain representation of the ground motion. Fourier or response spectral plots will peak at resonant frequencies. The location of these peaks will depend on the thickness and seismic velocities of the soil layers. For a simplified single layer 1-D structure, the fundamental frequency, $f_0$ and its harmonics, $f_n$ are given by the expressions below:

$$f_0 = \frac{v_z}{4H} \tag{2.2}$$

$$f_n = \frac{2n+1}{f_0} \tag{2.3}$$

Where: $v_z$ is shear wave velocity

H is the layer thickness and

n is integer.

Very thick deposits or very soft soils (of low-shear wave velocity) are therefore characterized by low fundamental frequencies (~0.2 Hz), whereas very thin or stiff layers have much higher fundamental frequencies (~10Hz).

## 2.4 Seismic parameters related to damage in buried pipelines

An historical revision of all the seismic parameters employed to represent seismic intensity in fragility functions is summarized in this section. The seismic parameters described in detail are Mercalli modified intensity (MMI), peak ground acceleration (PGA), peak ground velocity (PGV), maximum ground strain, and a recently proposed composite parameter (Section 2.5). Other parameters used as fragility function arguments are not included here because there is not enough evidence of their relationship with pipeline damage; among them are permanent ground displacement, Arias intensity, spectral acceleration, and spectral intensity.

### 2.4.1 Mercalli modified intensity

Though it is a parameter of subjective nature, MMI was used as damage indicator for pipelines in the 80s and 90s (Eguchi 1983 and 1991; Ballantyne et al. 1990; and, O'Rourke T. et al. 1998). A likely reason for the development of MMI-based fragility relations in the past was the extended use of that parameter to describe damage to aboveground structures. Lately, the installation of seismic stations and the availability of seismic records have made it easier to estimate parameters like PGA and PGV, which are better related to buried pipeline damage.

### 2.4.2 Peak Ground Acceleration (PGA)

PGA was largely employed as a damage indicator for pipelines during 25 years, from the study of Katayama et al. (1975), to the PGA-based fragility function of Isoyama et al. (2000). Though it has been largely demonstrated that PGV is related more closely to pipeline damage than PGA, as it is further explained in the following paragraphs, there are several reasons to explain why PGA, instead of PGV, was used to create some fragility functions before 2000. Most seismic stations record time histories of acceleration instead of velocity; then, PGA can be directly obtained from seismic records without involving the integration process needed for computing PGV. Most attenuations laws provide estimates of PGA (before 2000, PGV attenuation laws were limited); thus, for practical purposes, PGA was the ideal parameter for analyzing pipeline damage, and therefore, creating pipeline fragility relations.

### 2.4.3 Peak Ground Velocity (PGV)

PGV is by far the most widely used seismic parameter for pipeline seismic fragility functions. Generally, PGV shows good correlation with pipeline damage; although some studies (Sections 2.4 and 2.5) have shown that, for pipeline located in soft soils, there are some complications, mainly due to the assumptions related to PGV's use as a damage indicator. PGV is better related to pipeline damage than PGA mainly due to two reasons: 1) PGV is related to ground strain –the main cause of pipeline damage due to seismic wave propagation (Section 2.3.2)–; and, 2) PGA is more related to inertia forces –forces that do not affect buried structures like pipelines–. Many studies have empirically demonstrated that PGV is better pipeline damage predictor than PGA (O'Rourke T. et al. 1998; Isoyama et al. 2000; and, Pineda 2002).

GV has been extensively used as damage indicator for pipelines considering two assumptions: 1) PGV is directly related with maximum ground strain $\varepsilon_g$; and 2) transient ground strain is the main cause of pipeline damage due to seismic wave propagation. The relationship between PGV and $\varepsilon_g$ can be analyzed in Equation 1 (Newmark 1967), where C is seismic wave velocity. From Equation 2.4, PGV is directly related to $\varepsilon_g$ only if C is constant. Since $\varepsilon_g$ is non-dimensional, PGV and C must be expressed with the same velocity units.

$$\varepsilon_g = \frac{PGV}{C} \tag{2.4}$$

### 2.4.4 Maximum ground strain ($\varepsilon_g$)

Because transient ground strain is the assumed main cause of pipeline damage due to seismic wave propagation, $\varepsilon_g$ is straightforwardly the optimum parameter for analyzing the relationship between pipeline damage and seismic intensity. Rigorously, $\varepsilon_g$ can be estimated from displacement time histories D(t) (Equation 2.5). In Equation 2.5, x is a space variable, ε(t) is ground strain time history, and max represents the maximum of the expression between absolute value brackets | |.

$$\varepsilon_g = \max\left|\varepsilon(t)\right| = \max\left|\frac{\partial D(t)}{\partial x}\right| \tag{2.5}$$

There are three major problems for estimating $\varepsilon_g$ through Equation 2. 5. First, $\varepsilon_g$ is generally obtained through the double integration of acceleration time histories; this process causes loss of information due to the involved mathematical operations. Procedures like correction of base line, filtering and tapering could generate ambiguous results if the parameters used in those operations are modified. Second, the derivation process of $\varepsilon_g$ with respect to a space variable (x) implies that the seismic records used in the analysis need to be referenced to an absolute time scale; this is a very significant limitation because only ground motion information from seismic arrays using the same time reference, and preferably located in the place of interest (e.g., the zone covered by a pipeline system), would be useful. The third and probably the most important problem is the high cost involved in installing and operating seismic arrays covering large extensions (e.g., area covered by a pipeline network).

In order to avoid the above-mentioned problems of Equation 2.5, Equation 2.1 has been used to obtain conservative estimates of $\varepsilon_g$. PGV can be easily obtained from seismic records or other sources (e.g., attenuation laws); on the contrary, C is far from being easy to obtain, which complicates the estimation of $\varepsilon_g$.

PGV is a more convenient parameter than $\varepsilon_g$ for analyzing pipeline damage due to seismic wave propagation for three reasons. First, PGV is easier to estimate than $\varepsilon_g$. Second, many studies have proved that PGV is well correlated with pipeline damage. Third, theoretically, there is a direct relationship between PGV and pipeline damage considering two assumptions already mentioned in this section. Notwithstanding these three points, there is evidence of a case in which PGV is not the best parameter for relating pipeline damage with seismic intensity.

## 2.5 MICROZONATION OF DHAKA CITY

Seismic microzonation is important for hazard assessment of an area due to earthquake. Seismic hazards due to local site effects such as soil amplification and liquefaction can be estimated by combining the available soil parameter data with the current hazard models or by making use of existing maps showing estimated models of levels of these collateral hazards. Due to recent improvement in the availability and quality of GIS technology, tabular database software, as well as computer hardware, a significant amount of current research is devoted incorporating GIS technology in seismic microzonation for Dhaka city. In this chapter geotechnical characteristics of Dhaka city (Bashar, 2004) is reviewed and by reviewing the outcome of an extensive research work done by Rahman, Gazi Md. Ferooz (2000) a seismic microzonation map of Dhaka city is adopted.

### 2.5.1 GEOGROPHICAL SETTING OF THE CITY

Dhaka the capital city of Bangladesh, was founded about 400 years ago by the side of the river Buriganga. The earliest available map shows Dhaka extending over an area of about 1.5 sq km near the junction of the Dholai Khal and Burigonga river. Large scale urbanization was initiated by the British Raj in 1904 when Dhaka was made the capital of East Bengal, a newly created province of British India. Dhaka gained city status in 1947 when it was made the capital of East Pakistan and by that time stretched over an area of about 40 sq km. The importance of Dhaka increased exponentially after 1971, phenomenally and according to the census of 1991 the area and population of Dhaka Megacity or Dhaka Statistical Metropolitan Area (DSMA) of were 1600 sq km and 6.83 million respectively. According to the same census the area under the Dhaka city corporation was 360 sq km, with a population 3.39 million. The present population of DSMA is about 13.0 million (2008).

Dhaka is situated between latitudes $23^0 42'$ and $23^0 54'$N and longitude $90^0 20'$ and $90^0 28'$E. The city is bounded by the rivers Burigonga to the south, Turag to the West, Balu to the East, and Tongi Khal to the North. The city has three distinct seasons: winter (November-February), dry with temperatures ranging from 100 to 200 C; the pre-monsoon season (March-May), with some rain and hot temperature reaching up to 400C; and the monsoon (June-October), which is very wet with temperatures around 300C. Dhaka experiences about 2,000 mm of rain annually, of which about 80% falls during the monsoon.

Urbanization in Dhaka is restricted mostly to the north bank of the river Buriganga. The four hundred year history of Dhaka city can be divided into five different stages of development: Pre-Mughal period, Mughal period, British period, Pakistan period and Bangladesh period.

### 2.5.2 GEOLOGY OF THE STUDY AREA

Quaternary sediments consisting of deltaic and alluvial deposits of the Ganges, Brahamaputra and Meghna rivers and their numerous tributaries underlie more than 80% of Bangladesh. According to the study of Morgan and McIntire (1959), there are two major areas of Pleistocene sediments, commonly known as Madhupur tract and Barind tract. The Madhupur block lies between the Jamuna and Old Brahmaputra rivers and 6 to 30 m above the mean sea level. Madhupur tract is bounded by faults; they appear to be uplifted and structurally complex; the Madhupur block has been titled eastward (Morgan and McIntire, 1959). The study area is situated on the southern tip of the Madhupur tract. Two characteristic units cover the and its surroundings, i.e. the Madhupur clay of Pleistocene age and alluvial deposits of recent age. The Madhupur clay is the oldest sediment exposed in and around the city area. The alluvial deposits are characterized by flood plains, depression and abandoned channels. The geological map of Dhaka metropolitan area is presented in Figure 2.2

The subsurface sedimentary sequence, up to the explored depth of 300m, shows three distinct entities; one is the Madhupur clay formation of Pleistocene age and is characterized by raddish plastic clay with silt and very fine sand particles. This Madhupur clay formation uncomfortably overlies the Dupi Tila formation of Pleistocene age composed of medium to coarse yellowish brown sand and occasional gravel. The incised channels and depression within the city are floored by recent alluvial flood plain deposits and is further subdivided into lowland Alluvium and high Alluvium (WASA 1991).

Geotechnical characteristics of the Madhupur Clay in Dhaka city and its surroundings vary significantly both aerially and vertically. The evaluated parameters, particularly its low strength and high compressibility values indicate that the clay, to some extent, is problematic for engineering construction. The moisture content and plastic limit results show that Madhupur Clay is normally consolidated to over consolidate. The clay is normal to active and has intermediate to high plasticity. The compressibility values suggest that the clay ranges from very low to highly compressible at different locations.

The Dupi Tila sand aquifer is the main source of water in Dhaka city. Madhupur Clay overlies the aquifer with a thickness of 8 to 45 m (averaging 10 m). the aquifer varies in thickness from 100 to 200 m (averaging 140 m). Ground water occurs at a depth of 25 to 30 , in the central part of the city. In the periphery the ground water lies at a depth of 15 to 20 m. Under the present conditions the peripheral rivers act as sources of recharge where the Dupi Tila sands are exposed along the riverbeds. Other sources of recharge are vertical percolation of rain and flood water, leakage from water mains and sewer system and seepage from standing water bodies within the city.

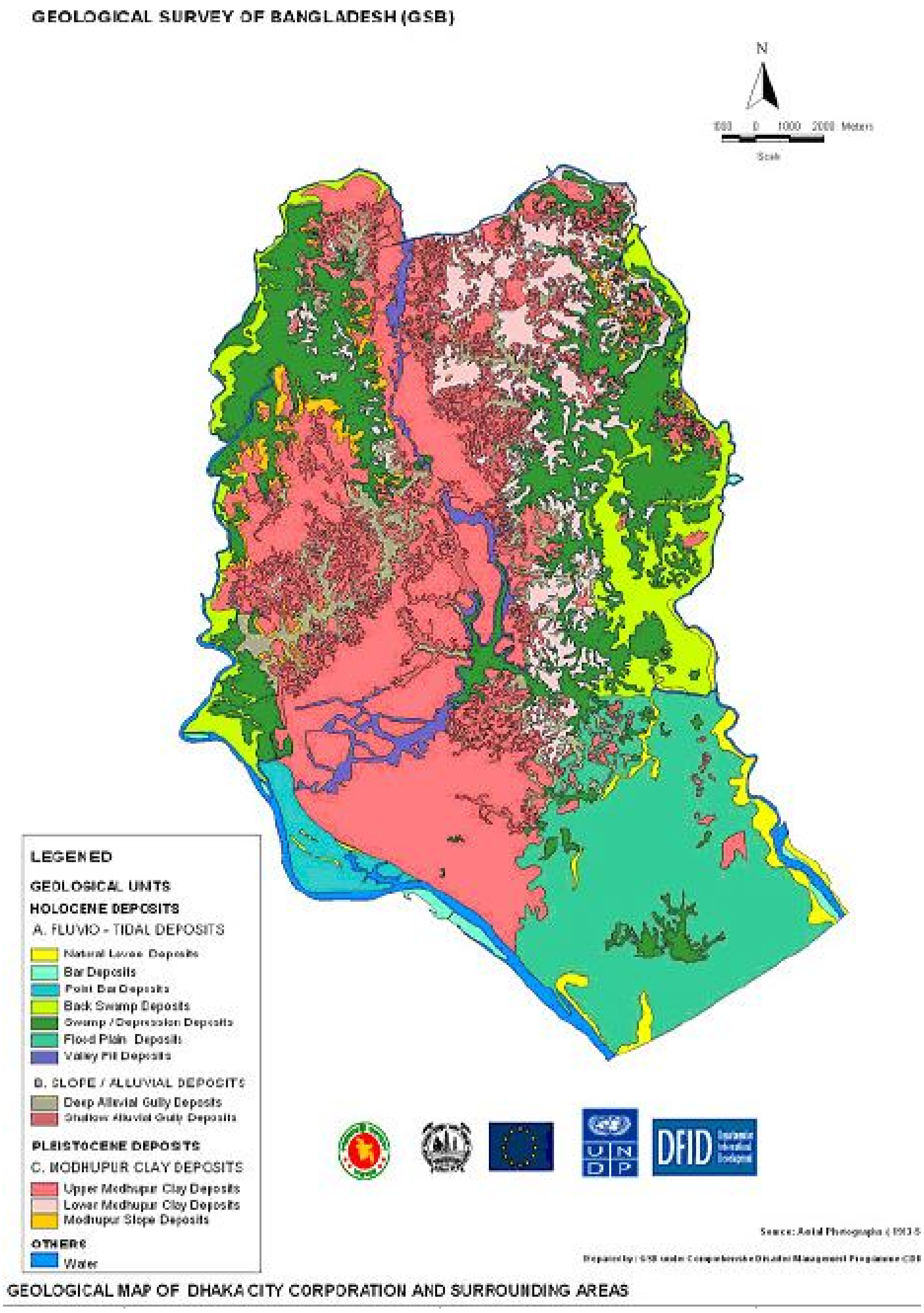



**Figure 2.2 GEOLOGICAL MAP OF DHAKA CITY (after GSB, 1990)**

### 2.5.3 REGIONAL TECHTONICS

Bangladesh lies in the Burma basin, which was formed by the continent collision of India to the north and subduction of ocean crust beneath the Burma continental crust to the east. Bangladesh is surrounded by regions of high seismicity, which include the Himalayan Arc and Shilong Plateau in the north, the Burmese Arc, Arakan Yoma anticlinorium in the east and complex Naga-Disand-Haflong thrust zone in the northeast shown in figure 2.3

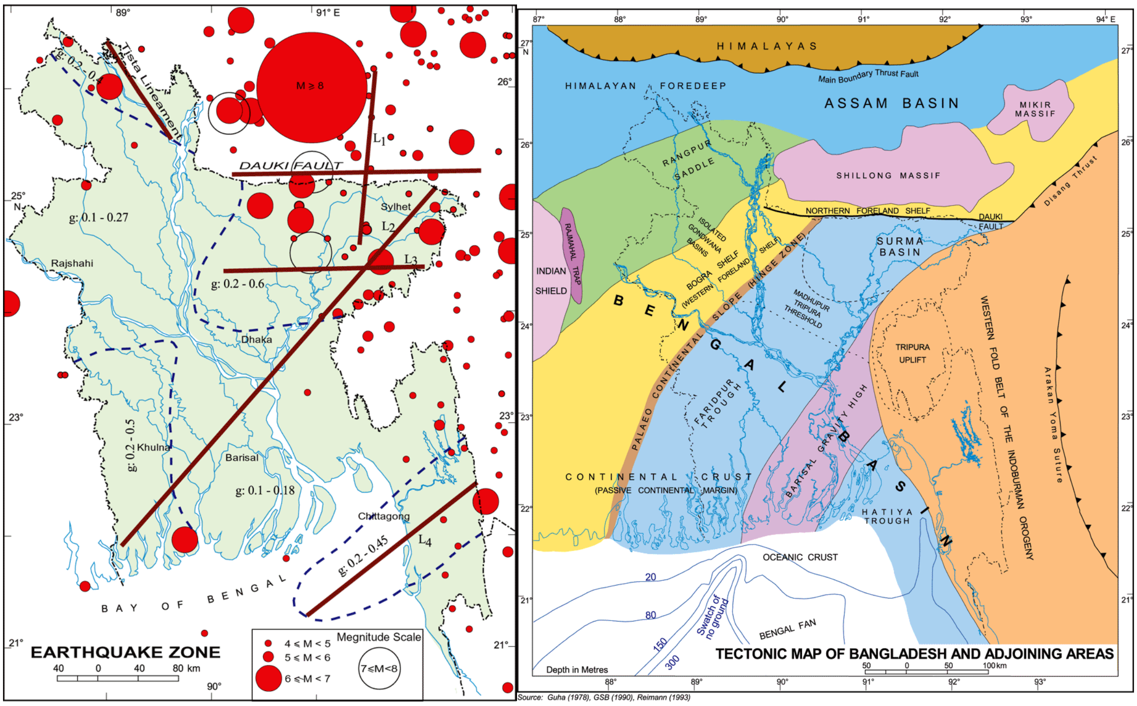



**Figure 2.3 Seismo-tectonic lineaments capable of producing damaging earthquake (after Banglapedia, 2004)**

The Dhaka city area does not show any surface folding. However, a large number of faults and lineaments have N-S, E-W, NE-SW, NW-SE trends recognized from air photo interpretation and the nature of the stream courses. All four sides of the city are bounded by major faults.

The country has a long history of seismic activity related to its proximity to the Himalayas. Three great earthquakes of magnitudes exceeding 8 were felt in 1897, 1934 and 1950 and another four earthquakes exceeding magnitude 7 were felt between 1869 and 1950. Major seismic sources are the Meghalaya (8.0), Tripura (7.0), Sub-Dauki (7.3) and Bogra (7.0), all of them with associated earthquakes of expected magnitudes higher or equal to 7.0.

The major earthquakes that have affected Bangladesh since the middle of the last century is presented in Table 2.2

**Table 2.2: Great historical earthquakes in and around Bangladesh (after Department of Disaster management, Bangladesh)**

| Date | Name | Epicentre | Magnitude (M) |
|---|---|---|---|
| 10-01-1869 | Cachar Earthquake | Jantia Hill, Assam | 7.5 |
| 14-07-1885 | Bangal Earthquake | Sirajgong, Bangladesh | 7.0 |
| 12-06-1897 | Great Indian Earthquake | Shillong plateau | 8.7* |
| 18-07-1918 | Srimangol Eathquake | Srimangal, Sylet | 7.6 |
| 02-07-1930 | Dhubri  Eathquake | Dhubri, Assam | 7.1 |
| 15-01-1934 | Bihar – Nepal Earthquake | Bihar , India | 8.3 |

* Recently modified as 8.1(M) (Ambraseys, 2001)

Bolt (1987) analyzed different seismic sources in and Bangladesh and arrived at conclusions related to maximum likely earthquake magnitude (Bolt, 1987). Bolt identified the following four major sources:

*(i) Assam fault zone*

*(ii) Tripura fault zone*

*(iii)Sub-Dauki fault zone*

*(iv) Bogra fault zone*

Reliable historical date for seismic activity affecting Indian subcontinent is available only for the last 450 year (Gupta et al, 1982). Recently developed earthquake catalogue for Bangladesh and surrounding area (Sharfuddin, 2001) showed that 66 earthquakes with Ms ≥ 4.0 occurred from 1885 to 1995 within a 200 km radius of Dhaka City. The most prominent historical earthquake affecting Dhaka is listed in Table 2.3.

**Table 2.3: Magnitude, EMS Intensities and distances of some major historical earthquakes around Dhaka (after Ansary, 2001 & 3CD City Profiles Series)**

| Date | Name of Earthquake | Magnitude (Richter) | Intensity at Dhaka (EMS) | Epicentral distance from Dhaka (km) |
|---|---|---|---|---|
| 10 January, 1869 | Cachar Earthquake | 7.5 | V | 250 |
| 14 July, 1885 | Bengal Earthquake | 7.0 | VII | 170 |
| 12 June, 1897 | Great Indian Earthquake | 8.7 | VIII+ | 230 |
| 8 July, 1918 | Srimangal Earthquake | 7.6 | VI | 150 |
| 2 July, 1930 | Dhubri Earthquake | 7.1 | V+ | 250 |
| 15 January, 1934 | Bihar-Nepal Earthquake | 8.3 | IV | 510 |
| 15 August, 1950 | Assam Earthquake | 8.5 | IV | 780 |

## 2.6 SEISMIC ZONING MAP OF BANGLADESH

The seismic zones and zone coefficients may be determined from the earthquake magnitude for the various return periods and the acceleration attention relationship. It is required that for the design or ordinary structures, seismic ground motion having 10% probability of being exceeded in design life of a structure (50 Year) is considered critical. An earthquake having 200 year return period originating in Sub- Dauki zone have epicentral acceleration of more than 1.0g but at 50 kilometers the acceleration Shall be reduced to as low as 0.3g. In the Bogra fault system, earthquake having 200 year return period have a value of only 7.3 and at

50 kilometers distance, the acceleration shall be reduced to a value of less than 0.1g. Ali (1998) presented the earthquake base and seismic zoning map of Bangladesh. Tectonic frame work of Bangladesh and adjoining areas indicate that Bangladesh is situated adjacent to the plate margins of India and Eurasia Where devastating earthquakes have occurred in the past. Non-availability of earthquake, geologic and tectonic date posed great problem in earthquake hazard mapping of Bangladesh in the past. The first seismic map which was prepared in 1979 was developed considering only the epicentral location of past earthquakes and isoseismal map of very few of them. During preparation of National Building Code of Bangladesh in 1993, substantial effort was given in revising the existing seismic zoning map using geophysical and tectonic data, earthquake data, ground motion attenuation data and strong motion data available from Geological Survey, of Bangladesh. Earthquake data were collected from NOAA date files and Geodetic Survey, U.S. Dept, of Commerce.

Seismic zoning map for Bangladesh has been presented in Bangladesh National Building Code (BNBC) Published in I993. l he pattern of ground surface acceleration contours having 200 year return period is the basis of this seismic zoning map. There are three zones in the map-- Zone l. Zone 2 and Zone 3. The seismic coefficients of the zones are 0.075g, 0.15g and 0.25g for Zone l, Zone 2 and Zone 3 respectively. Bangladesh National Building Code (I993) placed Dhaka City area in Seismic Zone 2 as shown in Figure 2.4. The Seismic Zones in the code are not based on the analytical assessment of seismic hazard and are mainly based on the location of historical data. An updated seismic zoning map as shown in Figure 2.5 based on analytical studies was recently developed by Sharfuddin, (2001). This zoning was based on consistent ground motion criterion such as equal peak ground acceleration levels. In this map also Dhaka City has been placed Zone 2. This map also has been three zones namely— Zone 1, Zone 2 and Zone 3. The seismic coefficients are also the same as in the map presented by BNBC (I993). The only modifications are is the zone areas. From both maps, it is seen that Dhaka city belongs to Zone 2 where the seismic coefficient is 0.15g.

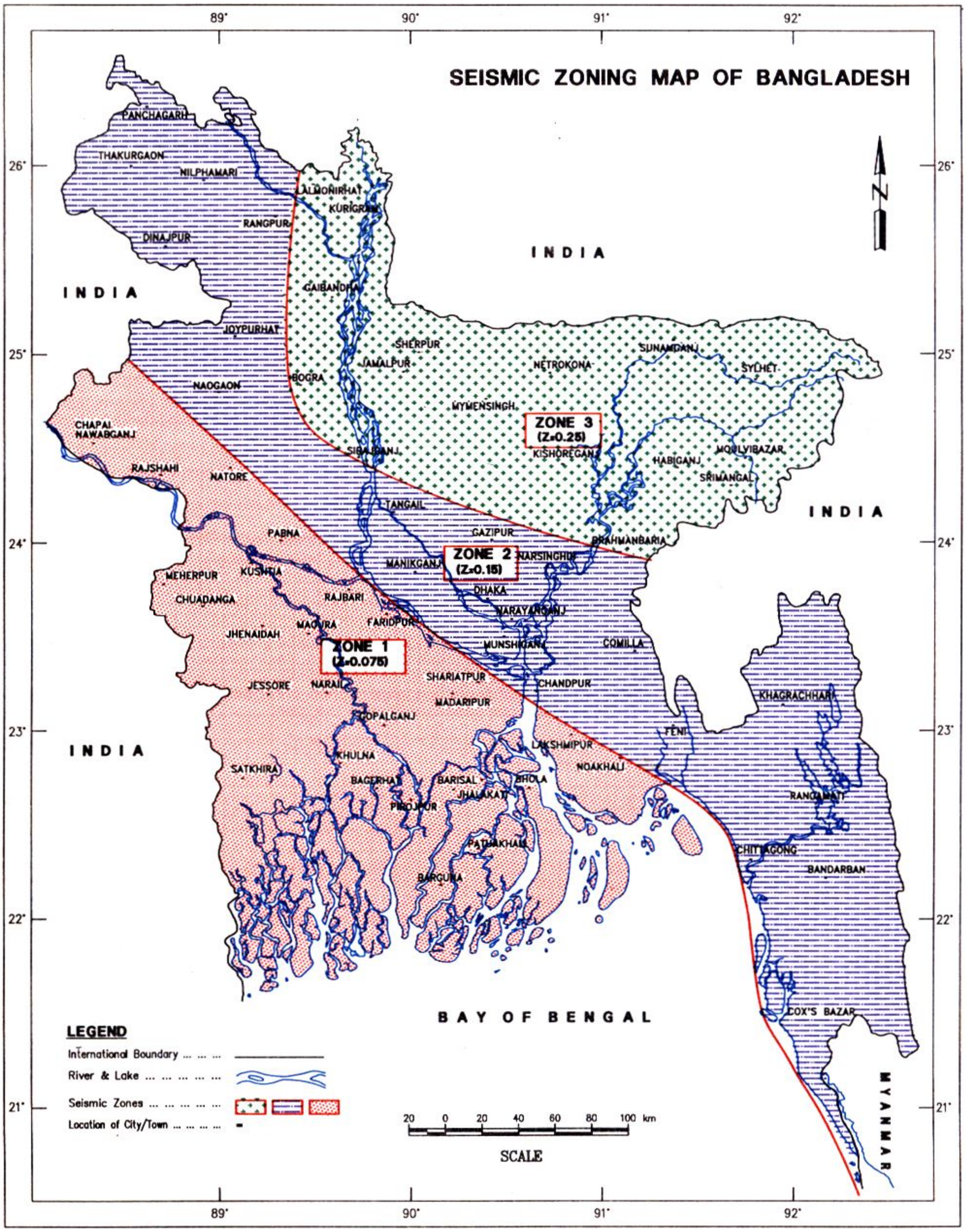



**Figure 2.4 Seismic Zoning Map of Bangladesh (after BNBC, 1993)**

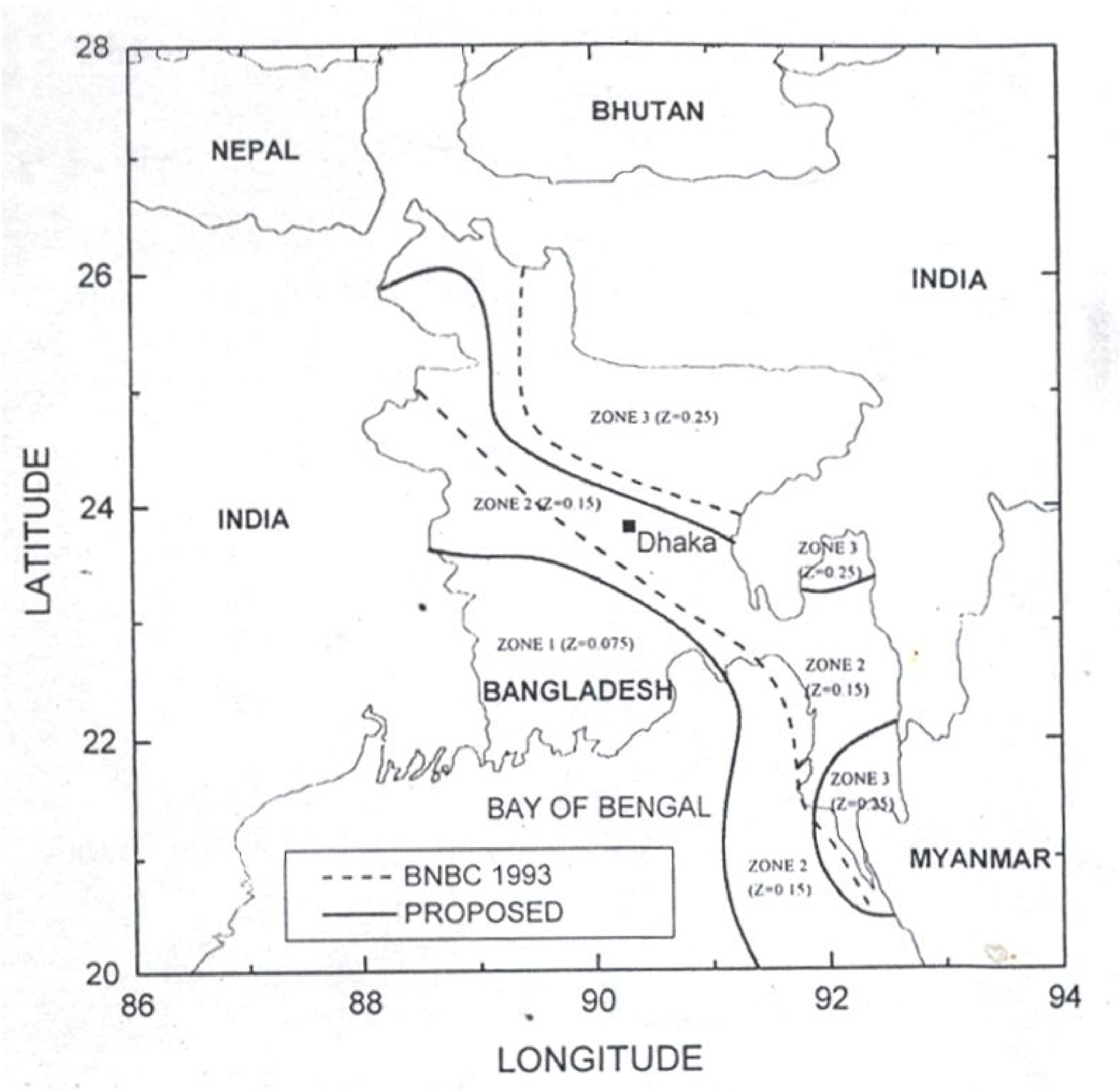



**Figure 2.5 Proposed Seismic Zoning Map of Bangladesh (after Sharfuddin, 2001)**

## 2.7 MICROTREMOR INVESTIGATION OF SITE EFFECTS

Micro tremor observation was carried out at different locations (120 in all) in Dhaka city during 2002 (Ansary, 2003). The equipment used was Tokyo Buttan Services GEODAS-10-24DA system connected to a tri axial accelerometer with a natural period of 1 second. In that experiment, the recording System operated continuously for about 6 minutes, with a sampling rate of 100 Hz. For the analysis of micro tremors, base line corrections were done and then a Butterworth band pass filter (0.40 to 25 Hz) was applied to the data. From the processed data sixteen 2048 point windows were selected and Fourier Spectra for NS, EW and UD Components were computed with a Parzen window. Then the mean curve for sixteen spectra both for NS and EW components were calculated. Finally, the Nakamura Spectral ratio as suggested by Equation (2.6) was obtained as follows:

$$HV = \frac{\sqrt{NS}\sqrt{EW}}{UD} \tag{2.6}$$

To validate the results obtained from micro tremor observations, H/V spectral ratios were compared with the transfer functions obtained from a one-dimensional numerical simulation using the computer program SHAKE, which consists of the response analysis of horizontally layered soils under seismic excitation, with linear equivalent soil behavior. Similar transfer functions from soil column using SHAKE were also estimated for area where no microtremor observations were made.

Use of geotechnical data for each of the sites and a synthesis of drilling data extracted from the existing subsurface database of Dhaka enabled to determine soil columns representative of each site. In most of the soil columns, a dense Sand layer was encountered at a depth of 30 m and in some cases silty clay layer was found. Soil columns of eight sites for which H/V spectral ratio and SHAKE transfer functions were compared.

Using the soil configurations a transfer function was calculated for each site using SHAKE numerical code. In addition, recordings of background noise by microtremor observations for each site were used to calculate average H/V spectral ratios. The amplification and the fundamental frequency obtained by the two methods are almost similar for all sites Studied. Figure 2.6 shows map of amplification at fundamental frequencies of Dhaka City.

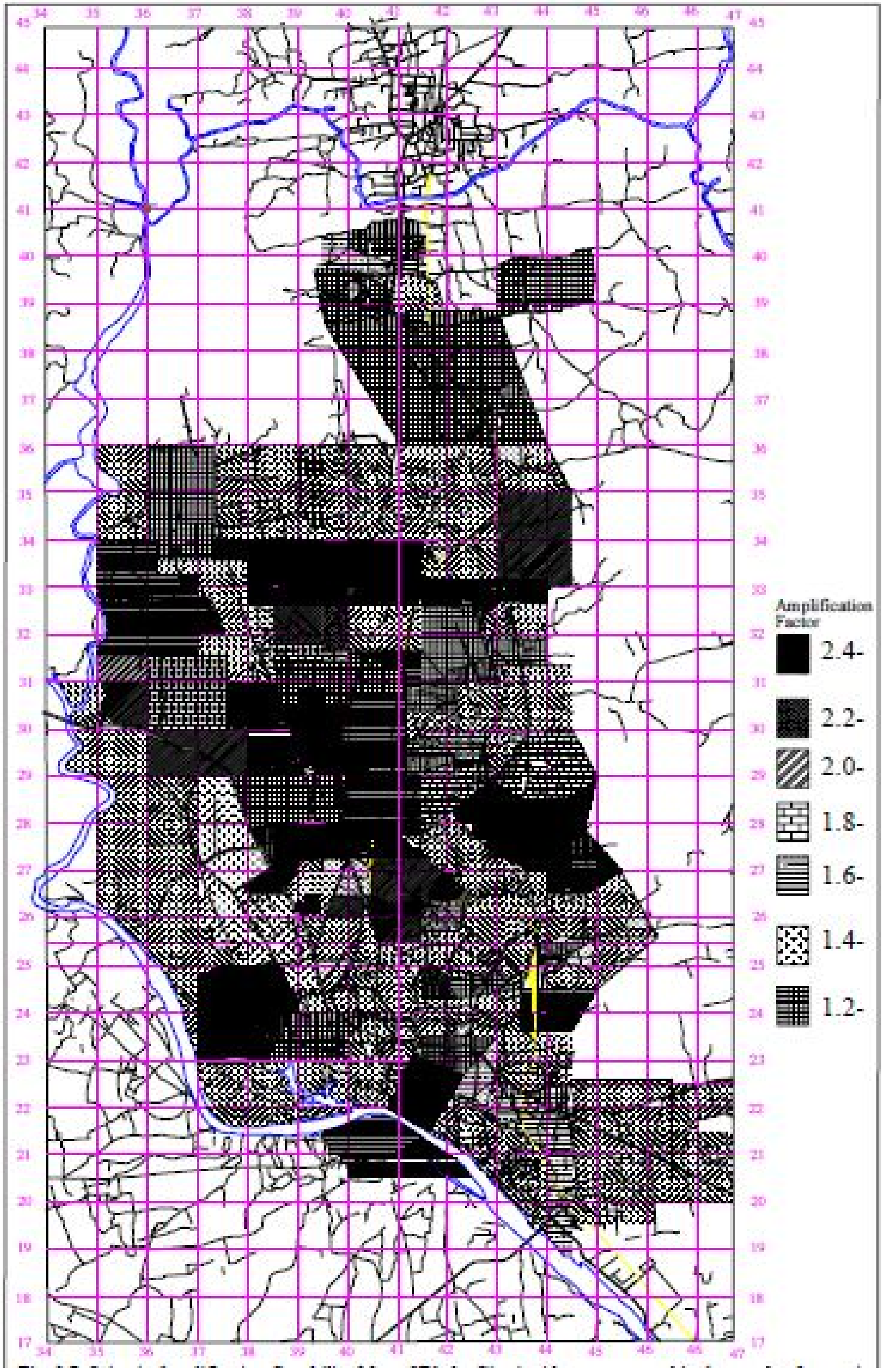



**Figure 2.6 Seismic amplification capability of Dhaka City (after Ansary, M.A. et. al. 2004)**

Bangladesh including Dhaka is largely an alluvial plain consisting of loose fine sand and silt deposits. Although the older alluvium consisting of mainly silty clay with deeper ground water table is less susceptible to liquefaction, the recent deposits consisting of loose fine sand with shallower water table along the river flood plains may liquefy during a severe earthquake. The ground water table is quite deep (20 to 25 m) in most places except the areas near the river. Clearly liquefaction is a serious component of the earthquake hazard in certain parts of Dhaka as indicated by Ansary and Rashid (2000) and needs to be considered.

In total area of Dhaka city are classed into two categories, one is liquefiable area and another is non- liquefiable. Figure 2.7 shows the map of liquefied areas and not liquefied areas.

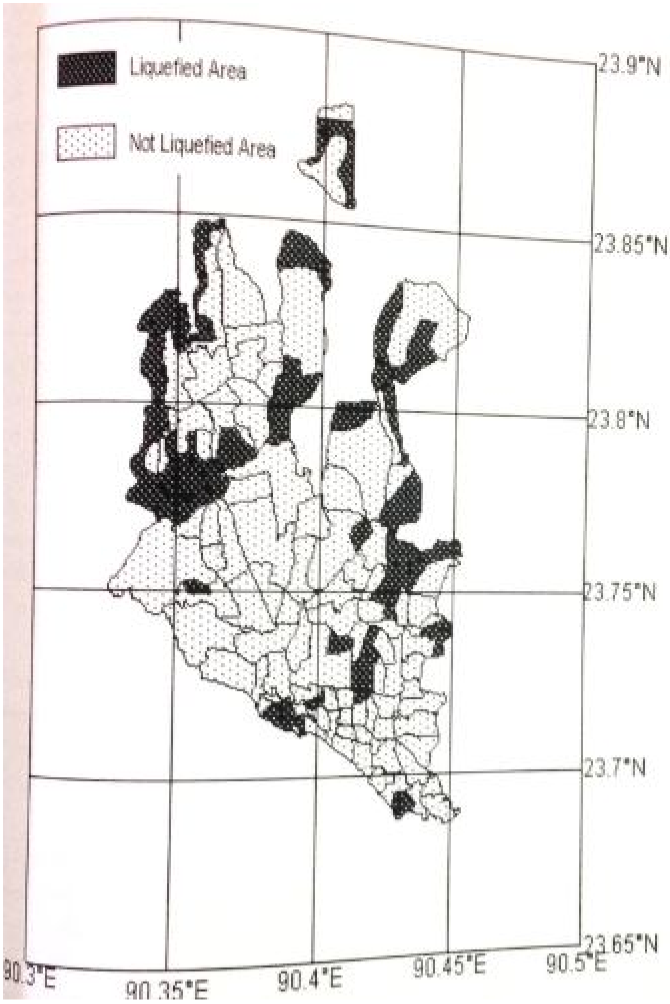



**Figure 2.7 Map showing liquefied and not liquefied areas of Dhaka City (after Rahman, 2004)**

## 2.8 SEISMIC MICROZONATION MAP OF DHAKA CITY

To mitigate/understand the seismic hazard a map of hazard assessment is required in which locations or zones with different level of hazard potential are identified. Seismic hazards due to local site effects such as soil amplification and liquefaction can be estimated by combining the available soil parameter data with the current hazard models or by making use of existing maps showing estimated models of levels of these collateral hazards. In order to establish such seismic microzonation map an extensive work was carried out by Rahman, Md. Gazi Ferooz (2000) where a soil database of 253 boreholes is developed. The soil data are used to develop site amplification and soil liquefaction assessment. Both of these site effects are integrated in Geographical Information System (GIS) platform for combined hazard assessment. Three past historical earthquakes are used as scenario events namely 1885 Bengal earthquake, 1897 Great Indian earthquake and 1918 Srimangal earthquake intensity value obtained for these events is calibrated against attenuation laws to check the applicability of the laws for this study. Using these laws, bedrock Peak Ground Acceleration (PGA) Values are obtained. Finally, a bedrock PGA value for scenario events is selected. PGA value is also converted into intensity values to integrate the effect of site amplification as well as liquefaction.

Every analysis region is different; therefore the quantification of the secondary site effects and the weighting scheme for combining the various seismic hazards is heuristic, based on judgment and expert opinion about the influence of local site conditions in the region and the exactness of the available geologic and geotechnical information.

At first the bedrock- level ground shaking in the region was ascertained. The shaking was depicted in terms of peak ground motion values. It is decided that the final combined seismic hazard would be quantified in terms of Modified Marcelli Intensity (MMI). There are several relationships for converting PGA to MMI. The equation used here is developed by Trifunac and Brady (1975). The following heuristic rules are used to quantity the seismic hazard attributable to liquefaction;

For Regions with liquefiable soil with high liquefaction potential

$$MMI_{LIQ}=MMI_{GS}+2 \tag{2.7}$$

For Regions with liquefiable soil with moderate liquefaction potential

$MMI_{LIQ}=MMI_{GS}+1$ (2.8)

And otherwise;

$MMI_{LIQ}=0$ (2.9)

The rules for combining the assorted hazards are based on expert option (after Stephanie and kiremidjian, 1994) about the comparative precision of the hazard information and the behavior of the local geology. For this study, two potential combinations were considered and their assumed weights are shown in Table 2.4. The final combines hazard ($MMI_F$) is computed as a weighted sum of the various hazards. By over-laying the regional maps for each hazard as shown in Figures 2.6 and Figure 2.7 in GIS environment, the Dhaka city had been separated into four groups as areas of 1.8 times amplifications, areas of 2.5 times amplifications, areas of 1.8 times amplifications plus liquefaction and areas of 2.5 times amplification plus liquefaction. And lastly figure 2.8 the regional distribution of the final combined seismic hazard ($MMI_F$) was produced.

**Table 2.4: Quantification rules for seismic hazard (after Stephanie and Kiremidjian 1994)**

| Rule | Possible hazards | Weighting scheme for final combined hazard $MMI_F$ |
|---|---|---|
| (a) | Ground shaking | $MMI_F = MMI_{GS}$ |
| (b) | Ground shaking + Liquefaction | $MMI_F = 0.55MMI_{GS}+0.45MMI_{LIQ} +5$ |

Notes

1. $MMI_F$ = Final Combined Hazard

2. $MMI_{GS}$ = Ground Shaking Hazard

3. $MMI_{LIQ}$ = Liquefaction Hazard

4. $MMI_F$ must be less than or equal 12

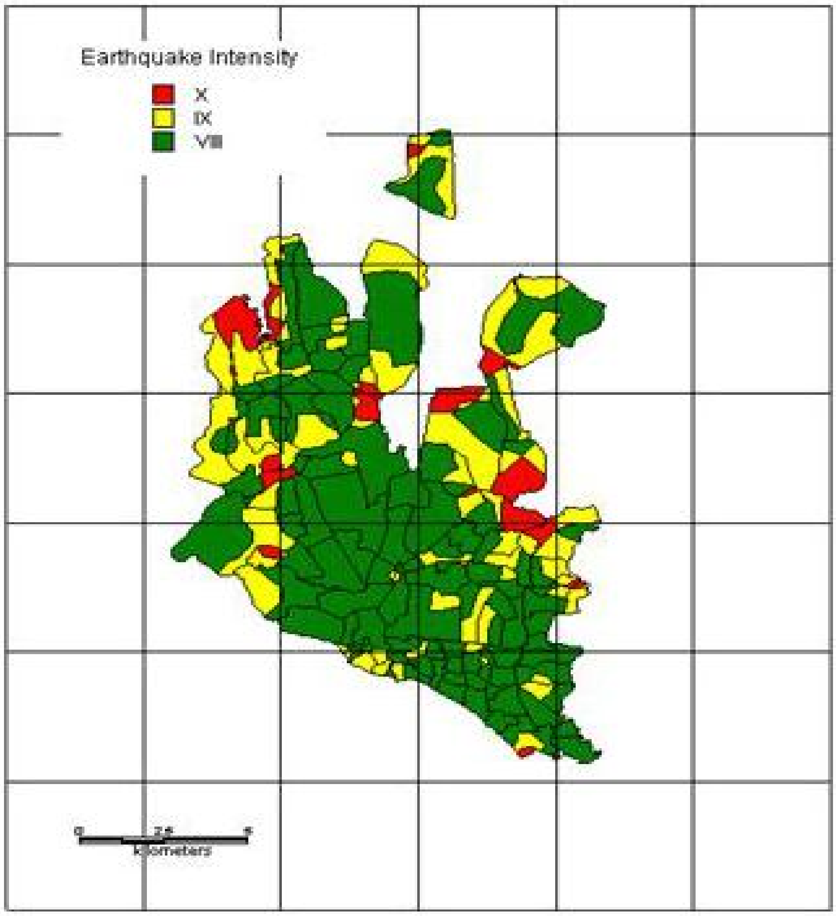



**Figure 2.8 Combined hazard intensity map (after Rahman, 2004)**

### 2.9 Seismic fragility functions for buried pipelines

The literature review has resulted in a total of 20 empirical studies (the list, in Table 2.5, may not be exhaustive) that addressed the issue of fragility relations for pipeline components subjected to transient ground shaking. We display these studies in the table below, along with the earthquake descriptor used, the typology of pipes, and the quality of the empirical data (i.e. number of earthquakes used):

**Table 2.5: Summary of fragility functions from literature**

| Referrence | Typology | Intensity index | No. of earthquakes studies |
|---|---|---|---|
| Katayama et al., 1975 | - mainly cast-iron pipes<br>- poor, average or good conditions | PGA | 6 |
| lsoyama & Katayama, 1982 | - mainly cast-iron pipes | PGA | 1 |
| Eguchi, 1983 | - WSGWJ (welded-steel gas-welded joints), WSAWJ (welded-steel arc-welded joints), AC (asbestos cement), WSCJ (welded-steel caulked joints), Cl | MMI | 4 |
| Barenberg, 1988 | - mainly cast-iron pipes | PGV | 3 |
| Eguchi, 1991 | - WSGWJ (welded-steel gas-welded joints), WSAWJ (welded-steel arc-welded joints), AC (asbestos cement), WSCJ (welded-steel caulked joints), CI (cast iron), DI (ductile iron), PVC, PE (polyethylene) | MMI | 4 |
| O'Rourke et aI., 1991 | | MMI | 7 |
| Hamada, 1991 | | PGA | 2 |
| O'Rourke & Ayala, 1993, HAZUS (NlBS, 2004) | - brittle or flexible pipes | PGV | 6 |

| | | | |
|---|---|---|---|
| Eidinger et al., 1995<br>Eidinger, 1998 | - material type<br>- joint type<br>- diameter<br>- soil type | PGV | 7 |
| O'Rourke et al.,1998 | - mainly cast-iron pipes | PGV,<br>PGA, MMI | 4 |
| lsoyama, 1998 | - material type,<br>- diameter | PGV | 1 |
| Toprak, 1998 | - no distinction | PGV | 1 |
| O'Rourke & Jeon, 1999 | - mainly cast-iron pipes<br>- diameter | PGV | 1 |
| Eidinger et Avila, 1999 | - material type<br>- joint type<br>- diameter<br>- soil type | PGV | - |
| lsoyama et al., 2000 | - DI, Cl, PVC, steel, AC<br>- diameter<br>- soil type | PGA,<br>PGV | 1 |
| ALA, 2001 | - material<br>- joint type<br>- soil type<br>- diameter | PGV | 18 |
| Pineda & Ordaz, 2003 | - mainly brittle pipes (Cl, AC) | PGV | 1 |
| O'Rourke & Deyoe, 2004 | - mainly cast-iron pipes | PGV, PGS | 5 |
| Pineda & Ordaz, 2007 | - mainly brittle pipes (Cl, AC) | $PGV^2/PGA$ | 1 |

Note:

MMI = Modified Mercali Intensity

PGV = Peak Ground Velocity

PGA = Peak Ground Acceleration

### 2.9.1 Fragility functions recommended by Katayama et al., 1975

This is one of the very first studies trying to establish a correlation between observe seismic damage and a strong-motion parameter (Figure 2.9). It is based on pipe failure rates obtained for six earthquakes (4 of them Japanese): 1923 Kanto, 1948 Fukui, 1964 Niigata, 1968 Tokachi-oki, 1971 San Femando and 1972 Managua earthquakes. A large scatter in the data is observed, probably due to larger damage rates induced in certain cases by permanent ground deformation. Most of the data used concerns cast-iron pipes, although the 1968 Tokachi-oki earthquake includes also damage to asbestos-cement pipes. No distinction is made on pipe diameter, joint types or pipe material. However the authors introduce a parameter b (Equation 2.10), which depends on several factors like soil condition or pipe age. Depending on the "poor", “average” or “good" conditions, this constant can take the respective values of 4.75, 3.65 or 2.0 (Ayala & O’Rourke, 1989).

$$RR = 10^{B+6.39\log PGA} \qquad (2.10)$$

### 2.9.2 Fragility functions recommended by Eguchi 1991

This study is an update of the earlier work of Eguchi (1983), based on damage date from four earthquakes: 1969 Santa Rosa, 1971 San Fernando, 1972 Managua and 1979 Imperial Valley earthquakes. This study (Figure 2.10) explicitly separates wave propagation damages from those induced by permanent ground deformation, and a distinction is made between different pipe materials and joint types. Bilinear relations with respect to macroseismic intensity (Modified Mercalli Intensity) are proposed for each of the pipe types.

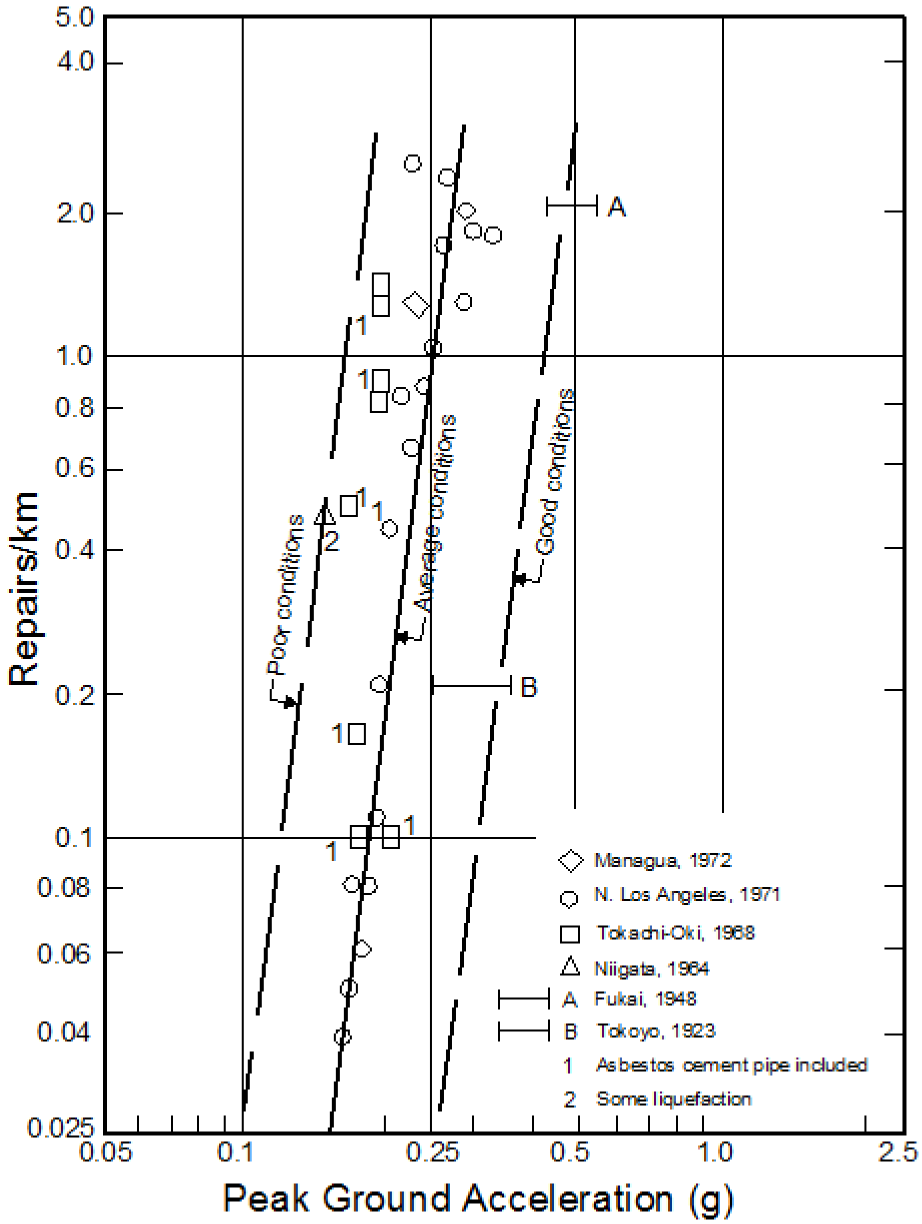



**Figure 2.9 Pipeline fragility data of Katayama et al., 1975 as presented by O'Rourke & Liu (1999)**

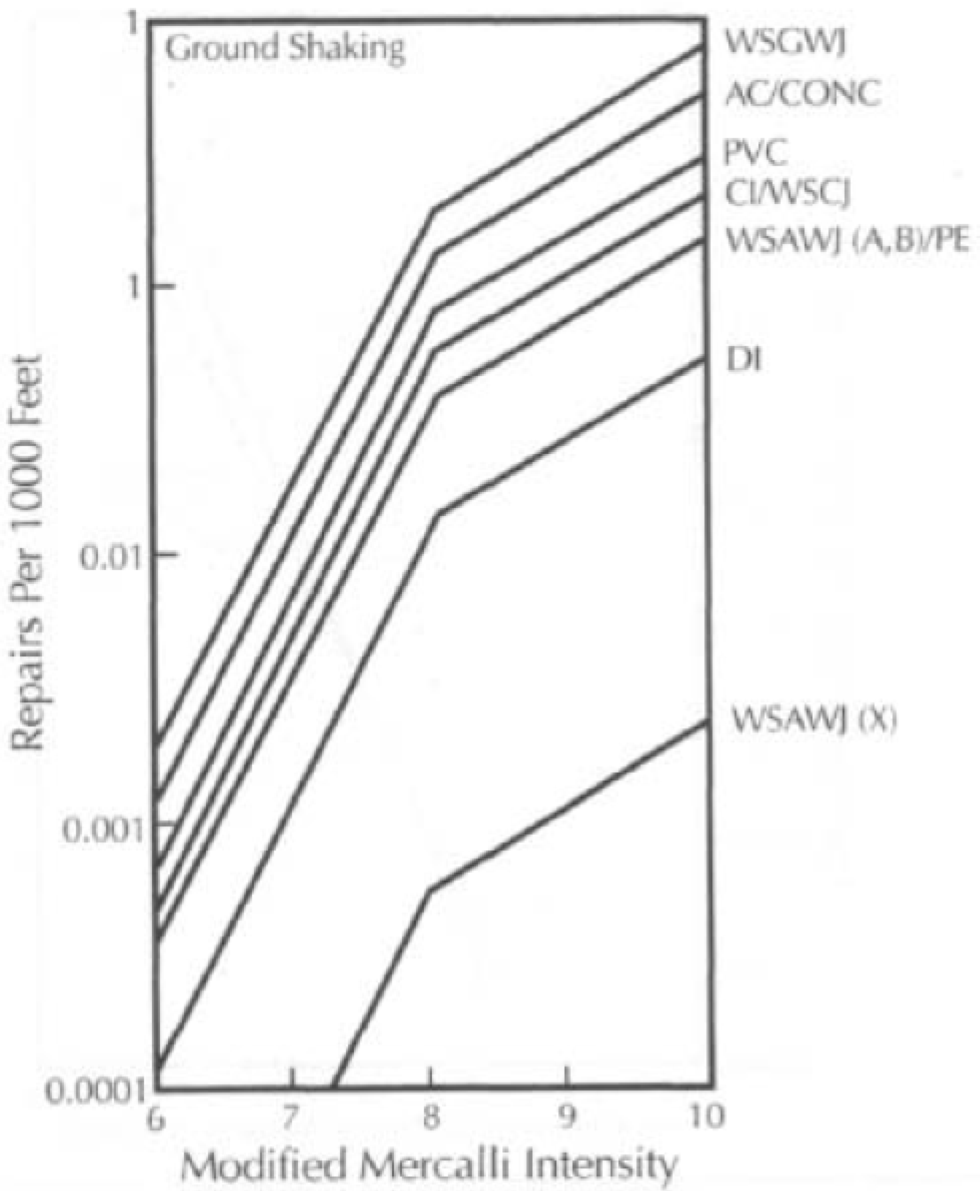



**Figure 2.10 Bilinear Pipeline fragility plot (after Eguchi 1991)**

### 2.9.3 Fragility functions recommended by O'Rourke & Ayala, 1993

This study uses the original data from Barenberg (1988), plus three additional earthquakes: 1983 Coalinga, 1985 Michoacan and 1989 Tlahuac earthquakes. A total of 11 data points are used to plot the trend line with respect to PGV (Figure 2.11), as it was found earlier by Katayama et al. (1975) and Barenberg (1988) that PGA is not the best earthquake descriptor for pipeline damage. The fragility equation is given by( Equation 2.11):

$$RR = 0.0001PGV^{2.25} \tag{2.11}$$

It appears to be only valid for brittle pipes, as the damage data is based on asbestos- cement, concrete and cast-iron pipes: for more ductile pipe material, it is recommended to multiply the repair rate by a corrective factor of 0.3.

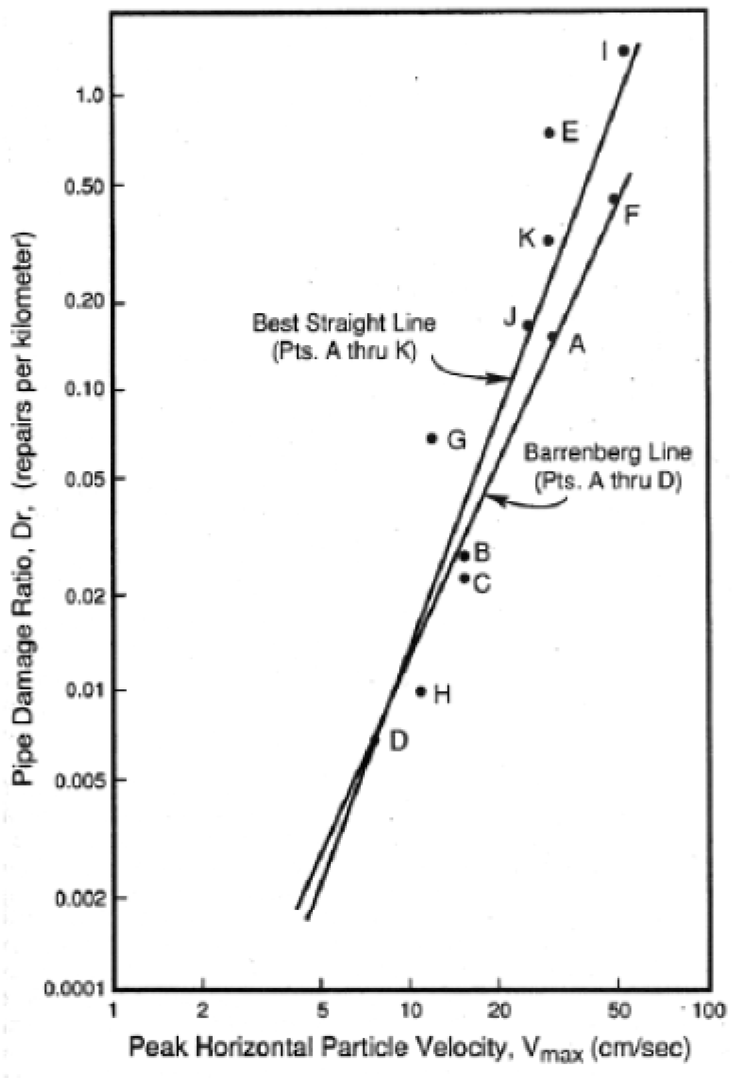



**Figure 2.11 Fragility relations proposed by Barenberg (1988) and O'Rourke & Ayala, (1993)**

### 2.9.4 Fragility functions recommended by Eidinger et al, 1995, 1998

The work of Eidinger et al. (1995) and Eidinger (1998) is based on the same data as O'Rourke & Ayala (1993), plus the 1989 Loma Prieta earthquake (i.e. seven US and Mexican earthquakes in total). Detailed pipeline data have allowed the authors to estimate different fragility relations based on various factors such as pipe material, diameter, joint type or soil corrosion. The "best-fit" relation (in this case regression with all data) is defined by the equation below (Equation 2.12) and displayed on Figure 2.12:

$$RR = K_1 0.0001658 PGV^{1.98} \tag{2.12}$$

Without any distinction on the pipe features, $K_1$=1. Otherwise, the values of this corrective factor are given in Table 2.6, for different pipe configurations.

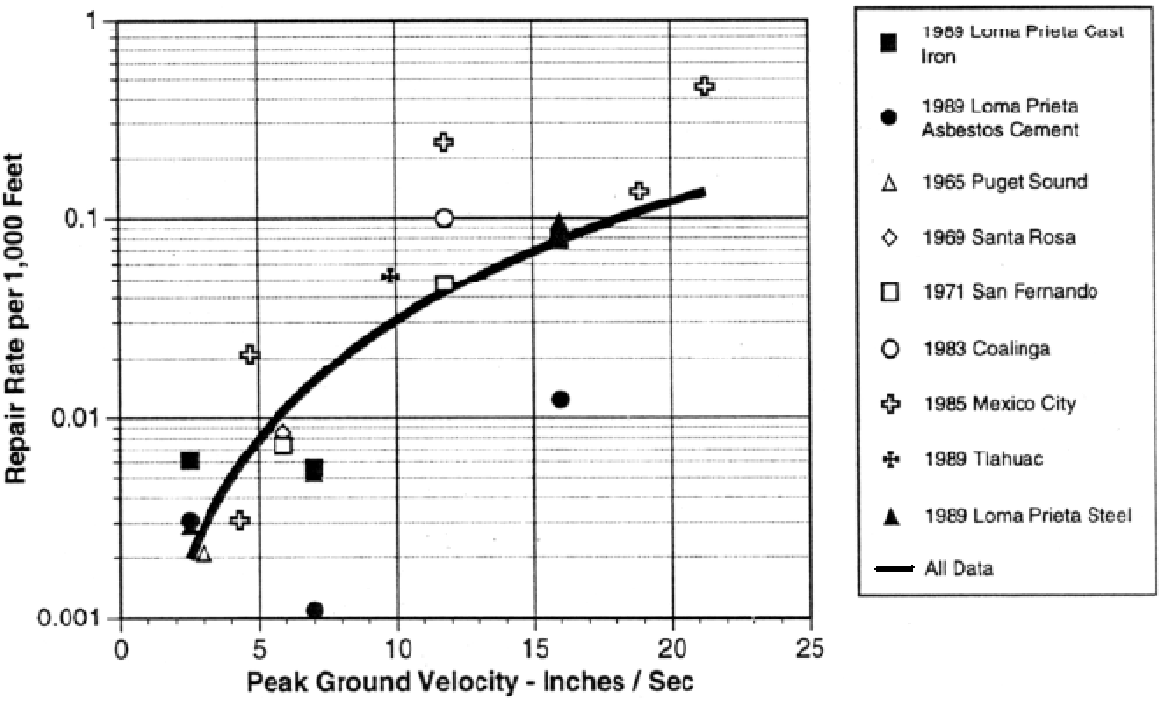



**Figure 2.12 Fragility relations proposed by Eidinger et al, 1995, 1998**

**Table 2.6: Values of corrective factor K1 (after Eidinger et al, 1995, 1998)**

| pipe material | joint type | soil | Diameter | K1 | quality index |
|---|---|---|---|---|---|
| CI | cement | unknown | Small | **0.8** | B |
| | cement | corrosive | Small | **1.1** | C |
| | cement | non corrosive | Small | **0.5** | B |
| | rubber gasket | unknown | Small | **0.5** | D |
| WS | arc welded | unknown | Small | **0.5** | C |
| | arc welded | corrosive | Small | **0.8** | D |
| | arc welded | non corrosive | Small | **0.3** | B |
| | arc welded | all | Large | **0.15** | B |
| | rubber gasket | unknown | Small | **0.7** | D |
| AC | rubber gasket | all | Small | **0.5** | C |
| | cement | all | Small | **1.0** | B |
| | cement | all | Large | **2.0** | D |
| C | welded | all | Large | **1.0** | D |
| | cement | all | Large | **2.0** | D |
| PVC | rubber gasket | all | Small | **0.5** | C |
| DI | rubber gasket | non corrosive | All | **0.3** | C |

Each of these $K_1$ values is linked to a quality index that gives the degree of confidence in the empirical data used:

- B: “there is a reasonable amount of background empirical data and study";

- C: “limited empirical data and study";

- D: “based largely on extrapolation and judgment, with very limited empirical data”

**2.9.5 Fragility functions recommended by Isoyama et al., 2000**

The work by lsoyama et al. (2000) is based on earlier studies of the 1995 Kobe earthquake. The damage data is concentrated on distribution pipes located in Kobe and two others cities nearby. The following functional forms (Equation 2.13) and (Equation 2.14) is adopted to represent the pipeline repair rate:

$$RR\ (IM) = B_p\, B_d\, B_g\, B_l\, R_0\ (IM) \tag{2.13}$$

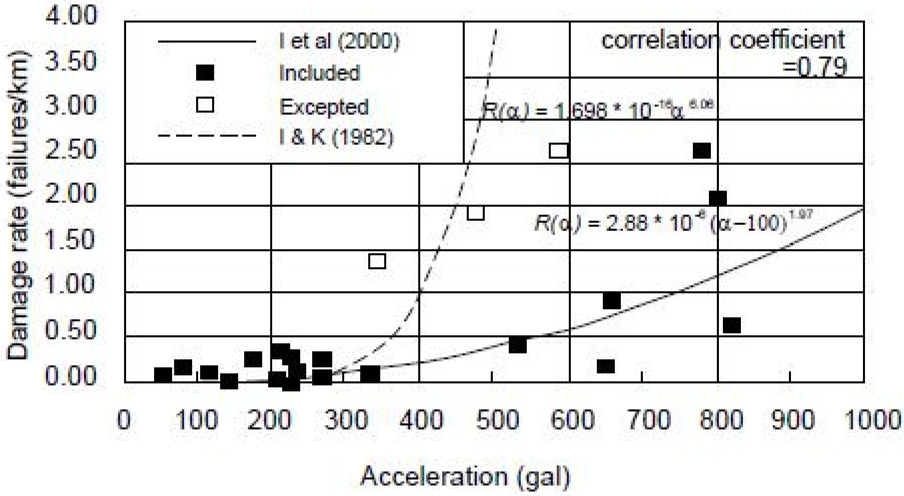
$$R_0\ (IM) = a\ (IM - IM_{min})^b \tag{2.14}$$

The intensity measure parameter, IM, is either PGA or PGV. $R_0$ represents the "standard" repair rate, for cast-iron pipes with medium diameter located in alluvial soil (Figure 2.13). The authors account for various typologies (pipe material, diameter, ground topography, liquefaction) by introducing corrective factors Bp, Ba, B9 and BL (see Table 2.7).

**Table 2.7: Values of corrective factors according to (lsoyama et al., 2000). Bracketed values are less reliable due to small sample size.**

| **pipe material, $B_p$** | | **pipe diameter (mm), $B_d$** | | **ground topography, $B_g$** | | **liquefaction, $B_L$** | |
|---|---|---|---|---|---|---|---|
| DI | **0.3** | 75 | **1.6** | disturbed hill | **1.1** | no liquefaction | **1.0** |
| CI | **1.0** | 100-150 | **1.0** | Terrace | **1.5** | partial liquefaction | **2.0** |
| PVC | **1.0** | 200-400 | **0.8** | narrow valley | **3.2** | total liquefaction | **2.4** |
| Steel | **(0.3)** | > 500 | **(0.5)** | Alluvial | **1.0** | | |
| AC | **(1.2)** | | | stiff alluvial | **0.4** | | |

For PGA, 19 data points were used and a relation was established with a=2.88x10$^{-6}$ and b=1.97. In the case of PGV, the values a=3.11$^{-3}$ and b=1.6 were found to best fit the 16 data points. The corresponding lines are displayed in the figure below:



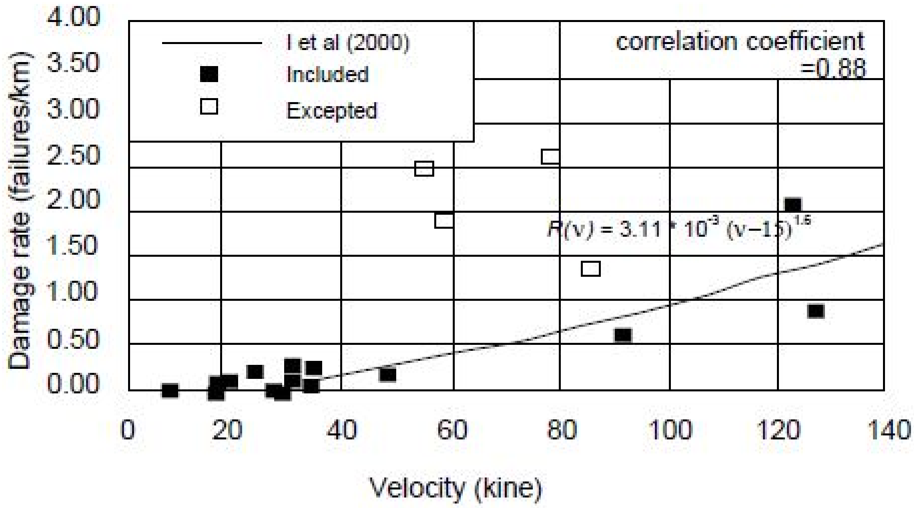




**Figure 2.13 Fragility relations by Isoyama et al. (2000), for both PGA (a) and PGV (b) parameters, without corrective factors.**

### 2.9.6 Fragility functions recommended by ALA, 2001

The work canted out in (ALA, 2001) is a compilation of several past studies, including a total of 18 earthquakes:

- Eidinger et al., 1995;
- Katayama et al., 1975;
- O'Rourke & Ayala, 1993;

- Shirozu et al., 1996;

- Toprak, 1998;

The study consists mainly in a homogenization of all available data and some data cleaning (some points were excluded due to an excessive influence of permanent ground deformation effects). This compilation gathers also a good sample of different material types, including ductile ones. A total of 81 data points are extracted and used to build a "backbone" curve, based on a single linear model, defining the median slope of all data points (Figure 2.14):

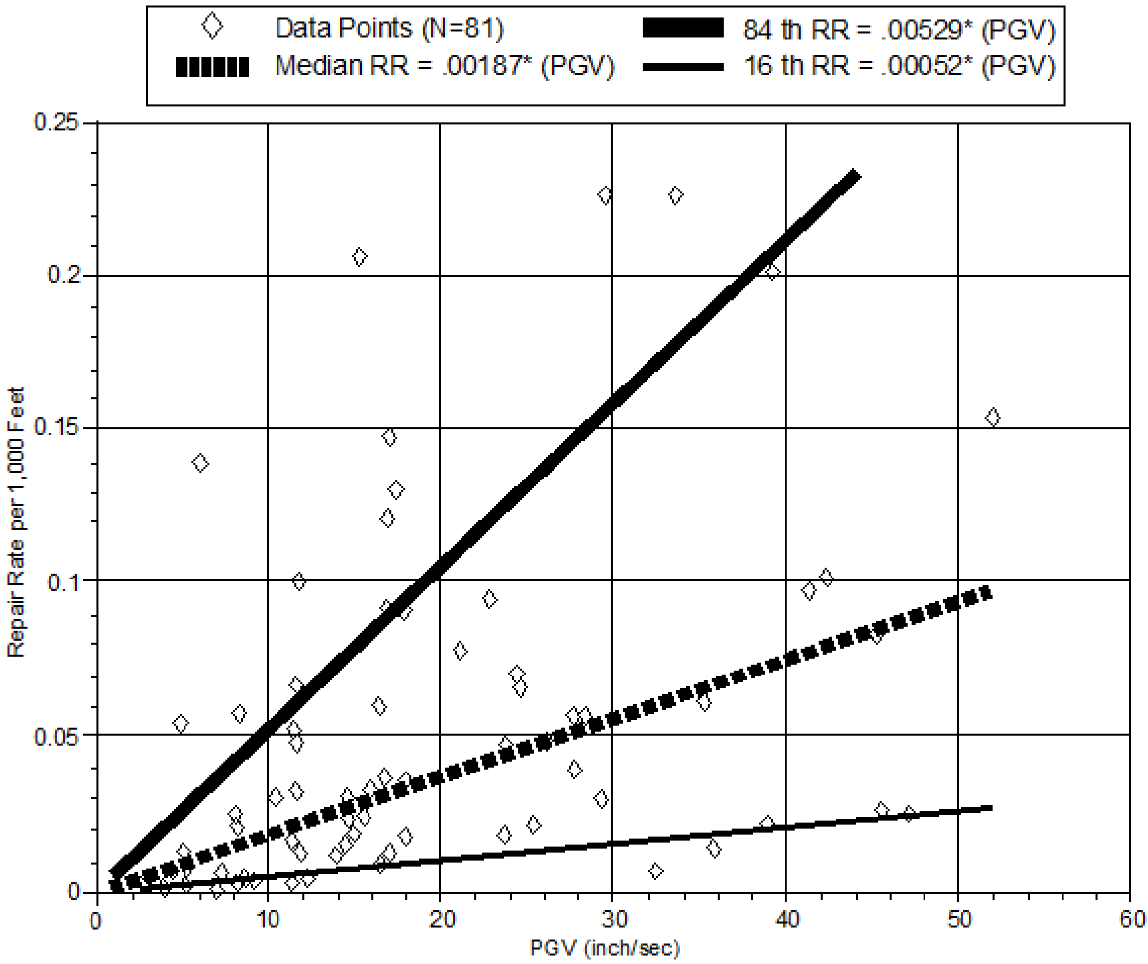



**Figure 2.14 "Backbone curve" proposed by ALA (2001), representing the median repair rate of all data points, and the corresponding 16th and 84th quantiles.**

The median curve is given by the following equation (with $K_1$=1):

$$RR = K_1 0.002416PGV \tag{2.15}$$

Like the work of Eidinger et al. (1995, 1998), a corrective factor K1 is introduced in order to account for various configurations such as pipe material, diameter, joint type, soil corrosion. The different values of this factor are given in the table 2.8 below:

**Table 2.8: Values of corrective factor K1, according to (ALA, 2001)**

| pipe material | joint type | soil | diameter | K1 |
|---|---|---|---|---|
| CI | cement | unknown | small | **1.0** |
| | cement | corrosive | small | **1.4** |
| | cement | non corrosive | small | **0.7** |
| | rubber gasket | unknown | small | **0.8** |
| WS | arc welded | unknown | small | **0.6** |
| | arc welded | corrosive | small | **0.9** |
| | arc welded | non corrosive | small | **0.3** |
| | arc welded | all | large | **0.15** |
| | rubber gasket | unknown | small | **0.7** |
| | screwed | all | small | **1.3** |
| | riveted | all | small | **1.3** |
| AC | rubber gasket | all | small | **0.5** |
| | cement | all | small | **1.0** |
| C | welded | all | large | **0.7** |
| | cement | all | large | **1.0** |
| | rubber gasket | all | large | **0.8** |
| PVC | rubber gasket | all | small | **0.5** |
| DI | rubber gasket | all | small | **0.5** |

### 2.9.7 Fragility functions recommended by O'Rourke & Deyoe, 2004

The work of (O'Rourke & Deyoe, 2004) includes data from three US and two Mexican events: 1965 Puget Sound, 1971 San Fernando, 1983 Coalinga, 1985 Michoacan and 1994 Northridge earthquakes. They introduce a criterion to select only statistically reliable data (Equation 2.12) where n is the minimum number of km of pipe data for a given repair rate:

$$n \approx 15.36 \frac{1-RR}{RR} \tag{2.16}$$

This led to the selection of 14 data points. Then, using the PGV value, the authors back-calculate the transient ground strain (based on the apparent wave propagation velocity, see equations in Table 2.9). A distinction is made between earthquakes generating surface waves (“shallow" earthquake and "distant“ basin) and events where body S-waves are dominant: the velocity of R-waves is assumed to be $C_R$=500 m/s, whereas $C_s$=3000 m/s. These assumptions are used to develop the following fragility relations Equations 2.17, 2.18 and 2.19, based on both transient ground strain and PGV:

$RR = 714\ \varepsilon^{0.92}$ (for all cases) (2.17)

$RR_R = 0.064PGV^{0.92}$ (if Rayleigh waves are dominant) (2.18)

$RR_R = 0.0035PGV^{0.92}$ (if Shear waves are dominant) (2.19)

This analysis has been performed on mainly segmented cast-iron pipelines.

**Table 2.9: Maximum longitudinal strains induced by seismic wave propagation along a pipeline (St John & Zahrah 1987)**

| | Ground motion direction | Maximum strain | |
|---|---|---|---|
| | | Incident angle | Value |
| R waves | Horizontal component parallel to the wave direction | 0° | $\varepsilon = \frac{PGV_R}{C_R}$ |
| S waves | Perpendicular to the wave propagation direction | 45° | $\varepsilon = \frac{PGV_S}{2C_S}$ |

## 2.10 Comparison of fragility relations

The majority of pipeline fragility relations use either PGA or PGV as the predictor parameter. A selection of available relations for PGA are given in Table and are plotted for comparison in Figure 2.15, along with an indication of the range of applicability of each relation, where this could be estimated.

The predictions of Bresko (1980), based on the data of Katayama et al. (1975) are significantly greater than any of the other predictions for PGA above about 200 cm/s$^2$. The high values predicted reflect both the influence of permanent ground deformation effects and large uncertainties in the derivation of repair rates. The curves of lsoyama & Katayama (1982) is based on the data of San Fernado earthquake, which according to Bresko (1980) yielded pipeline repair data for the PGA range 170-330 cm/s$^2$. Data from the Hyogoken-nanbu earthquake used by Isoyama et. al. (2000) included data for PGA up to about 800 cm/sec$^2$ and is more reliable in the range of $330 < PGA < 800$ cm/s$^2$. In any case Isoyama et. al. (2000) is based on much more reliable and comprehensive than that of earlier study.

**Table 2.10: Pipeline fragility relations for PGA derived by several investigators. RR denotes repair rate. PGA is measured in cm/s$^2$.**

| Investigators | $R_R$ = f(PGA) | Notes |
|---|---|---|
| Katayama *et al.* (1975) | $10^{b+6.39\log_{10}PGA}$ (3.28) | Mainly CI pipes. Data is from Katayama *et al.* (1975) Trend is as suggested by Bresko (1980) for "average conditions" ($b$ = 3.65) |
| Isoyama & Katayama (1982) | $1.698\times10^{-16}PGA^{6.06}$ (3.29) | CI pipes |
| O'Rourke *et al.* (1998) | $10^{1.25\log_{10}PGA-0.63}$ (3.30) | CI pipes |
| Isoyama *et al.* (2000) | $2.88\times10^{-6}(PGA-100)^{1.97}$ (3.31) | CI pipes |

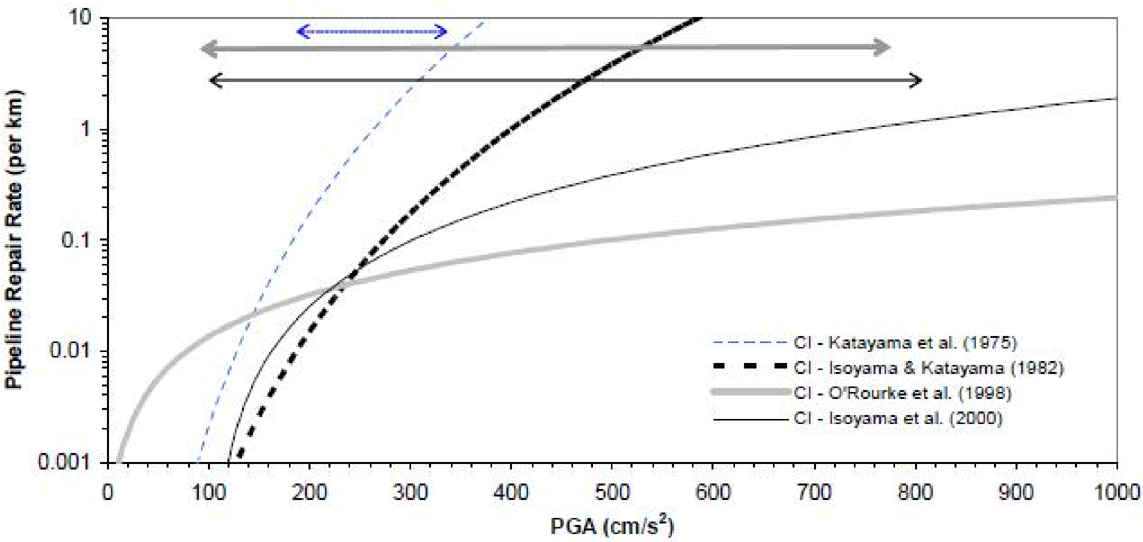



**Figure 2.15 Comparison of the pipeline fragility relations for PGA**

The relation derived by O'Rourke et. el. (1998) predicts high repair rates for low values of PGA. However, application of the relation to PGA values below about 90 cm/s$^2$ requires extrapolation beyond the limits of the dataset. For PGA greater than around 220 cm/s$^2$, the O'Rourke et al. (1998) relation predicts lower repair rate values than the Japanese study. The two curves diverge significantly: the ratio of repair rates for the two relations at 400 and 800 cm/s$^2$ are 2.9 and 6.4 respectively. The reasons for this difference are not clear without more information on how the relations were derived.

Various PGV fragility relations are expressed in Table 2.11 and compared graphically in Figure 2.16. The range of applicability for each relation is indicated in the Figure, estimated from the range of PGV values used to derive each study.

**Table 2.11: Pipeline fragility relations for PGV derived by several investigators. RR denotes repair rate. PGV is measured in cm/s.**

| Investigators | $R_R$ = f(PGV) | | Notes |
|---|---|---|---|
| Eidinger *et al.* (1995, 1998) | $K_1 0.0001658 PGV^{1.98}$ | (3.32) | "best-fit" fragility relation ($K_1$ = 1), converted from Imperial units to SI units |
| O'Rourke & Ayala (1993) HAZUS (FEMA, 1999) | $0.0001 PGV^{2.25}$ | (3.33) | "brittle pipes" fragility relation |
| Isoyama *et al.* (2000) | $3.11\times10^{-3}(PGV-15)^{1.30}$ | (3.34) | CI pipes "standard curve" |
| O'Rourke *et al.* (2001) | $e^{1.55\ln PGV-8.15}$ | (3.35) | CI pipes |
| ALA (2001) | $K_{1ALA} 0.002416 PGV$ | (3.36) | "backbone" fragility relation ($K_{1ALA}$ = 1), converted from Imperial units to SI units |

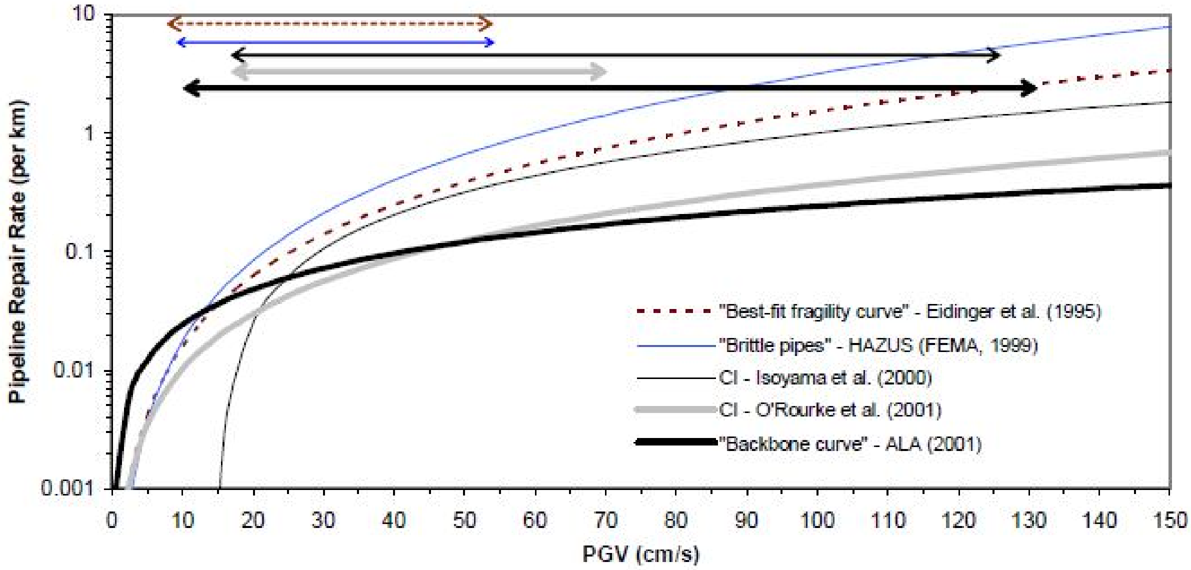



**Figure 2.16 Comparison of the pipeline fragility relations for PGV. (Arrows refer to the range of applicability of a given relation, approximated from knowledge of the dataset from which it was derived.)**

The HAZUS curve, based on the data of O'Rourke & Ayala (1993) gives the highest predictions of pipeline repair rate for PGV greater than 15 cm/s. O'Rourke (1999) considers this fragility relation 95 to be over-conservative, with pipeline repair rates being unduly affected by the long durations of ground shaking experienced during the Michoacan earthquake.

The Eidinger et al. (1995, 1998) and Isoyama (2000) relations predict repair rates within about a third of each other over the range 35 < PGV < 70 cm/s. These predictions are remarkably close for fragility relations, especially considering the fact that completely different data sets were used in each case. The disagreement at lower levels of PGV is largely due to the assumption by Isoyama et al. (2000) of a lower PGV threshold for pipeline damage. The Eidinger et al. (1995, 1998) relation has a much more limited range than that of Isoyama et al. (2000) and probably should not be extrapolated much beyond about 55 cm/s. The HAZUS relation is based on a dataset with a similarly restricted range.

For the range of strong-motion values typically associated with destructive earthquakes, the variation in repair rate obtainable using different fragility relations is generally less for PGV than PGA. This suggests that PGV may be a better predictor of earthquake induced pipeline damage than PGA. However, many factors have contributed to the scatter observed among the various fragility relations and a more quantitative investigation is required to draw more firm conclusions.

The investigations of O'Rourke et al. (1998, 2001) and Isoyama et al. (2000) suggest that PGV is more effective than PGA for the prediction of pipeline damage caused by earthquake-induced ground shaking. That this should be the case has been suspected for a long time. Newmark (1967) highlighted the close connection between ground strain and PGV and this served as the motivation for the first PGV fragility relation (Barenberg, 1983). Measures of ground acceleration (although not necessarily the peak ground acceleration) are of more relevance in predicting damage to aboveground structures, for which inertial forces are much more important.

Pipeline fragility relations have improved considerably over recent years and are useful for damage prediction. For general application, the PGV relation of ALA (2001) is recommended as it is derived from a global database.

Although it has been shown that PGV is a better predictor parameter for pipeline damage than PGA, it is nevertheless useful to have PGA fragility relations because of the widespread use of this parameter in earthquake risk assessment. It should be stressed, however, that wherever possible, predictions of pipeline damage should be made from PGV estimates.

## 2.11 SUMMARY

Major objective of this part of thesis was to introduce seismic microzonation of Dhaka city & to review the existing empirical pipe line fragility relations. Seismic microzonation map of Dhaka city has been introduced with a view to assess regional multi-hazard seismic risk. The response of buried water supply pipeline due to ground shaking and Site effects has been reviewed. Factors affecting earthquake vulnerability is also reviewed. A thorough review work is done on existing empirical relations such as Katayama (I975), O'Rourke (1982), lsoyama & Katayama (1998) and Isoyama (2000) for the prediction of earthquake – induced pipeline damage and finally a comparison is made among the selected fragility relationships.

**CHAPTER THREE**

## ANALYSIS OF DAMAGE IN BURRIED GAS PIPELINE

### 3.0 GENERAL

The damageability of buried gas supply pipelines in seismic Zone can be very serious and it is necessary to take preventive measures that eliminate, or at least decrease, that damageability. Pipelines that are the main source of gas distribution for important cities in seismic zone should be investigated and analyzed in terms vulnerability to earthquakes. Institutions and authorities responsible for the design, Construction and operation of buried pipelines located in seismic zones should demand that the seismic effects are correctly taken into consideration in order to assure the good behavior of such pipelines during their working life.

The damage Produced by breakage or disconnection of pipelines is quite variable and can be related to technical, economical and social aspects. The breakage of gas pipelines, for instance, besides representing a health hazard and fire risk causes leakage and the repairs in the pipeline represent an important cost

The damage algorithm for buried pipe is expressed as a repair rate per unit length of pipe, as a function of ground shaking or ground failure. The development of damage algorithms for buried pipe is primarily based on empirical evidence, tempered with engineering judgment and sometimes by analytical formulations. Empirical evidence means the following: after an earthquake, data is collected about how many miles of buried pipe experienced what levels of shaking and how many pipes were broken or leaking because of that level of shaking.

Repair rate of pipelines due to earthquake is related to either peak ground acceleration (PGA) or peak ground velocity (PGV). There exist a good number empirical relations such as Katayama (1975), O' Rourke (1982), Isoyama & Katayama (1998) and Iosyama (2000) for the Prediction of earthquake-induced pipeline damage analysis which are Presented in chapter two. In case of PGA O'Rourke (1982) and Isoyama (2000) relations are used. In case of PGV O'Rourke (2000) and Isoyama (2001) relations are used to predict the damage rate of pipelines. Finally an estimation of financial loss is presented.

### 3.1 PIPELINE DATABASE DEVELOPMENT OF DHAKA CITY

GIS provides an ideal tool for analyzing relationships amongst spatial datasets. G1S is increasingly used in lifeline engineering for post earthquake investigation of damage and for risk assessment.

The history of gas supply system of Dhaka city is very long. The only known maps available are those created by “TITAS gas T & D Company Ltd” which has been responsible for design, finance and construction of gas Supply System. A copy of maps of gas supply network of I988 covering the whole Dhaka city at a scale of 1:10000 is collected for this study. This map is scanned at first and then the whole gas pipeline networks are digitized which is illustrated in Figure 3.1.

No distinction is made between different pipe materials or diameters, although it is known from the authority that the network consists of cast iron pipes with diameter 75 mm to 300mm.

Microzonation maps of different intensities are grouped presented in chapter three, and pipeline networks of different diameter are laid in these maps on GIS platform which are shown in Figures 3.2 to 3.3. Finally, the lengths of pipelines are calculated according to intensity from these digitized intensity- pipe maps for analysis which are shown in Table 3.1

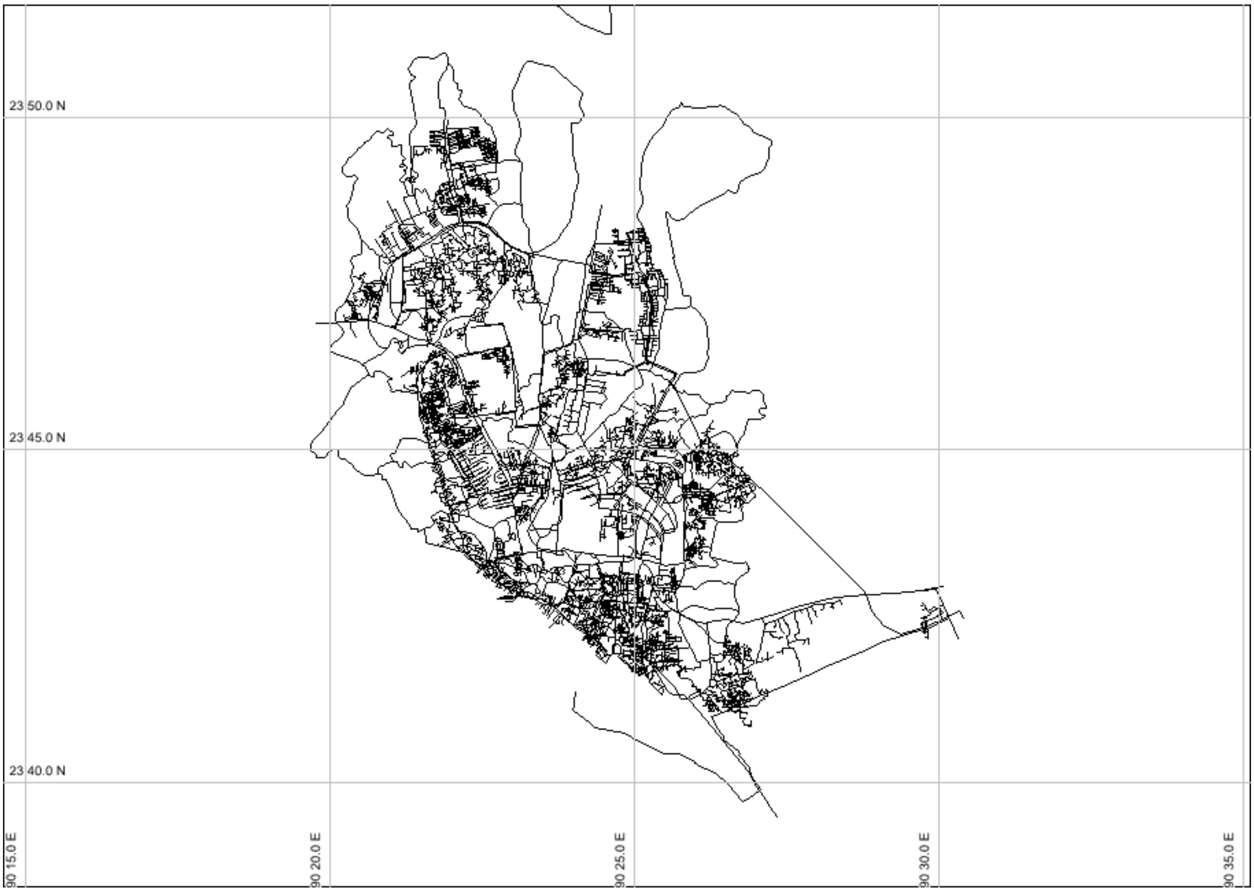



**Figure 3.1 Digitized layout of Gas pipe line network of Dhaka City.**

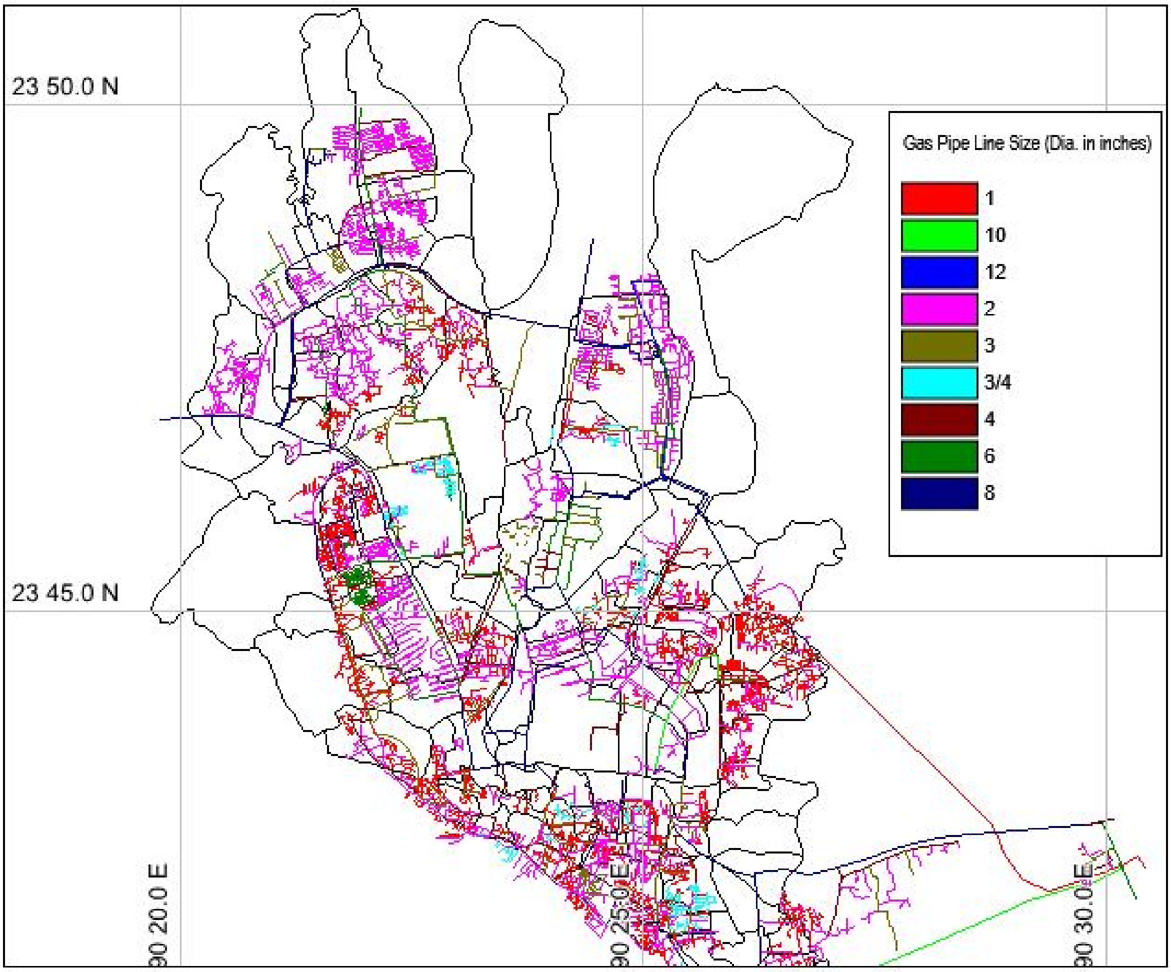



**Figure 3.2 Digitized layout of Gas pipe line network of Dhaka City showing diameter of pipe.**

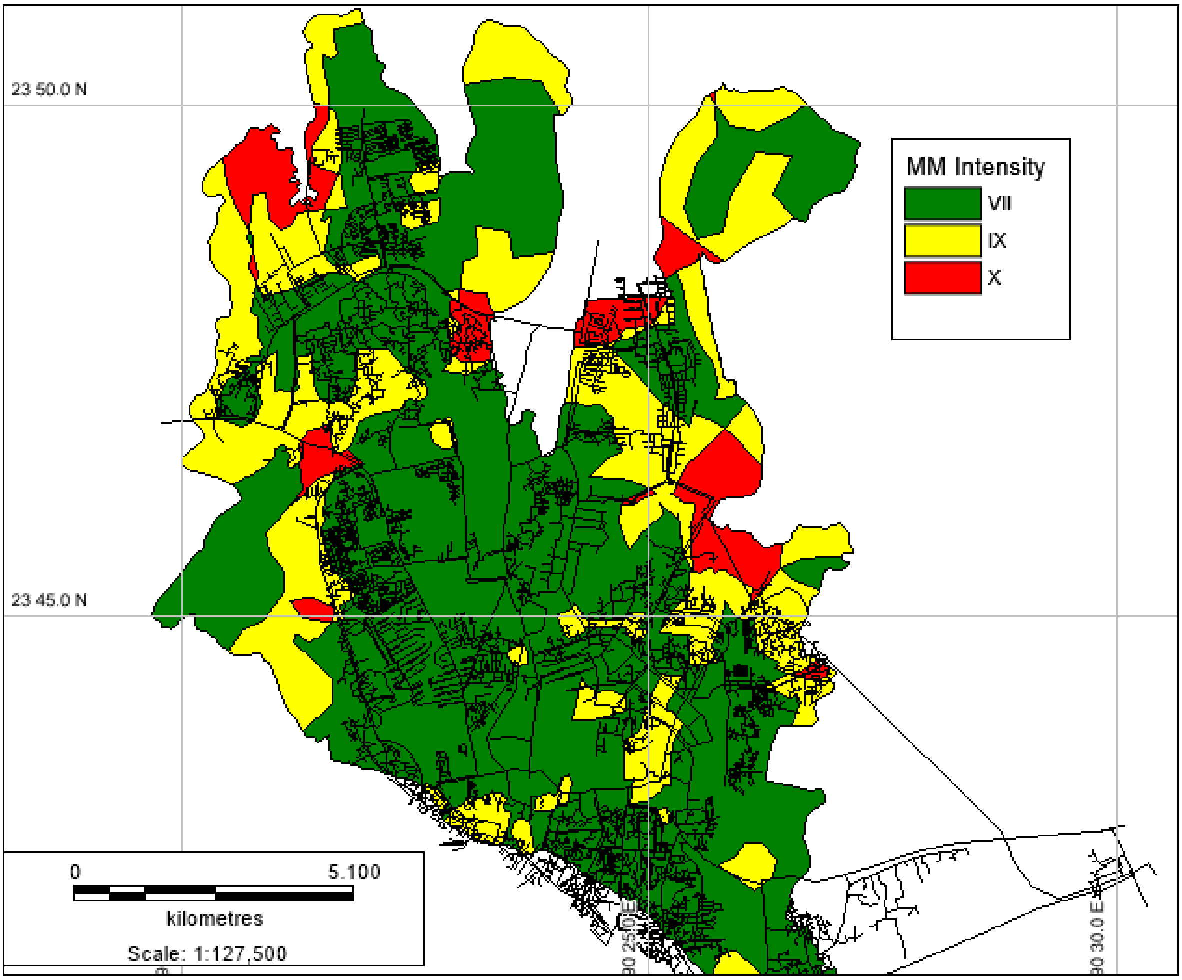



**Figure 3.3 Digitized layout of Gas pipe line network superimposed on MM intensity map of Dhaka city.**

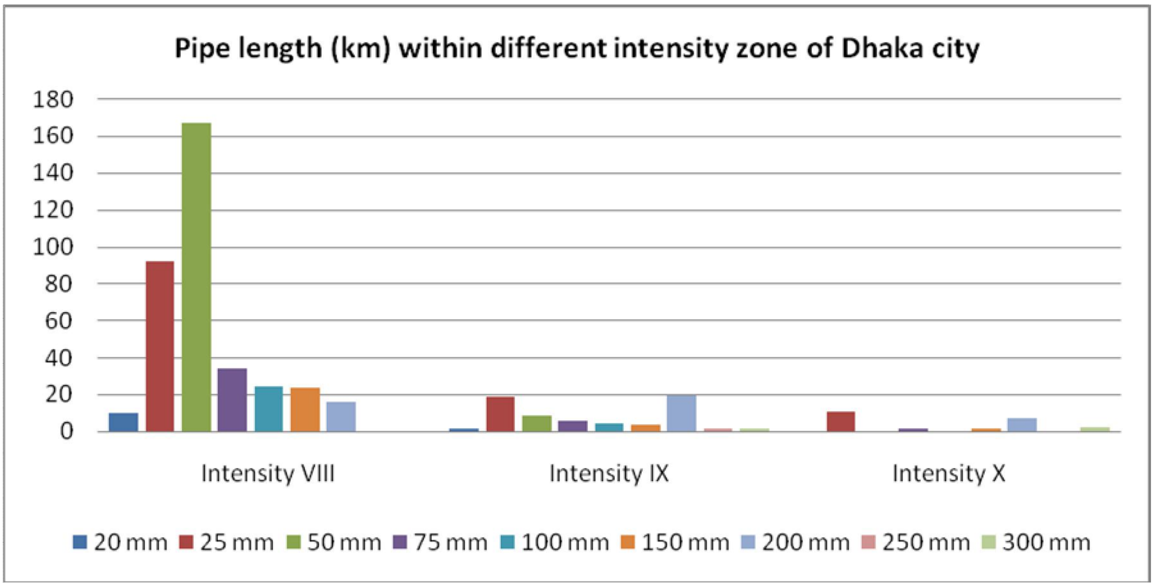



**Figure 3.4 Gas Pipe length (km) within different intensity zone of Dhaka city**

**Table 3.1: Gas pipe line length within different intensity of Dhaka city**

| Intensity (MMI) | Pipe length (km) | | | | | | | | | | Total Length (km) |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 20 mm | 25 mm | 50 mm | 75 mm | 100 mm | 150 mm | 200 mm | 250 mm | 300 mm | N/A | |
| 8 | 10.03 | 92.33 | 167.02 | 33.72 | 24.29 | 23.79 | 15.81 | 0.00 | 0.00 | 9.04 | 376.03 |
| 9 | 1.27 | 18.58 | 8.14 | 5.53 | 4.06 | 3.35 | 19.09 | 1.50 | 1.55 | 4.24 | 67.31 |
| 10 | 0.00 | 10.37 | 0.00 | 1.43 | 0.00 | 1.39 | 7.09 | 0.00 | 1.94 | 0.00 | 22.23 |

## 3.2 SELECTION OF PEAK GROUND ACCELERATION (PGA) VALUES FROM INTENSITY

In studies related to earthquake damage estimation and earthquake insurance, it has been observed that the Modified Mercalli intensity scale is the easiest and most convenient to work with. Most of the available damage statistics are related to the MM intensity at a site. However, for the recent instrumentally recorded data, the information on ground motion is usually in the form of peak ground motion parameter such as the PGA. Again, many empirical data base relationships are available in the literature to relate the MM intensity with the PGA. Peak ground acceleration is an instrumentally recorded continuous variable whereas modified Mercalli intensity is a subjectively assigned discrete Integer variable. Thus, it should be expected that there will be a range of PGA values corresponding to a given intensity level.

In the past, a number of researchers have developed PGA-MMI relationships. In each of the relationships given below, I is Modified Mercalli intensity and A is peak ground acceleration in $cm/sec^2$.

Gutenber and Richter (1942) $\log A = -0.5+0.33I$ (3.1)

Hersberger (1956) $\log A = -0.9+ 0.43I$ (3.2)

Ambraseys (1974) $\log A = -0.16+0.36I$ (3.3)

Trifunac and Brady (1975) $\log A = 0.014+0.3I$ (3.4)

All the above relationships are log-linear in format. Using the above relationships, different PGA values calculated for different MM intensity and show in Table 3.2

MM intensity and PGA values from Table 3.2 have also been plotted in Figure 3.4 for comparison. It can be seen from the plot of Figure 3.4 that for a particular intensity, Ambraseys relation shows higher value and Gutenberg & Richter curve shows lower value. All most all of the remaining curves lie in between these two. The curve from Trifunac & Brady is close to the median value. Henceforth intensity-PGA relationships given by Gutenberg-Richter and Trifunac-Brady are considered for our analysis; values in higher side are ignored.

**Table 3.2: PGA values based on different existing empirical relationships for different intensity**

| Intensity (MMI) | Gutenberg and Richter (1942) | Hershberger (1956) | Ambraseys (1974) | Trifunac and Brady (1975) |
|---|---|---|---|---|
| 5 | 0.014 | 0.018 | 0.045 | 0.033 |
| 6 | 0.031 | 0.049 | 0.102 | 0.066 |
| 7 | 0.066 | 0.131 | 0.234 | 0.133 |
| 8 | 0.141 | 0.354 | 0.535 | 0.265 |
| 9 | 0.301 | 0.952 | 1.226 | 0.528 |
| 10 | 0.643 | 2.561 | 2.809 | 1.053 |
| 11 | 1.376 | 6.894 | 6.434 | 2.101 |

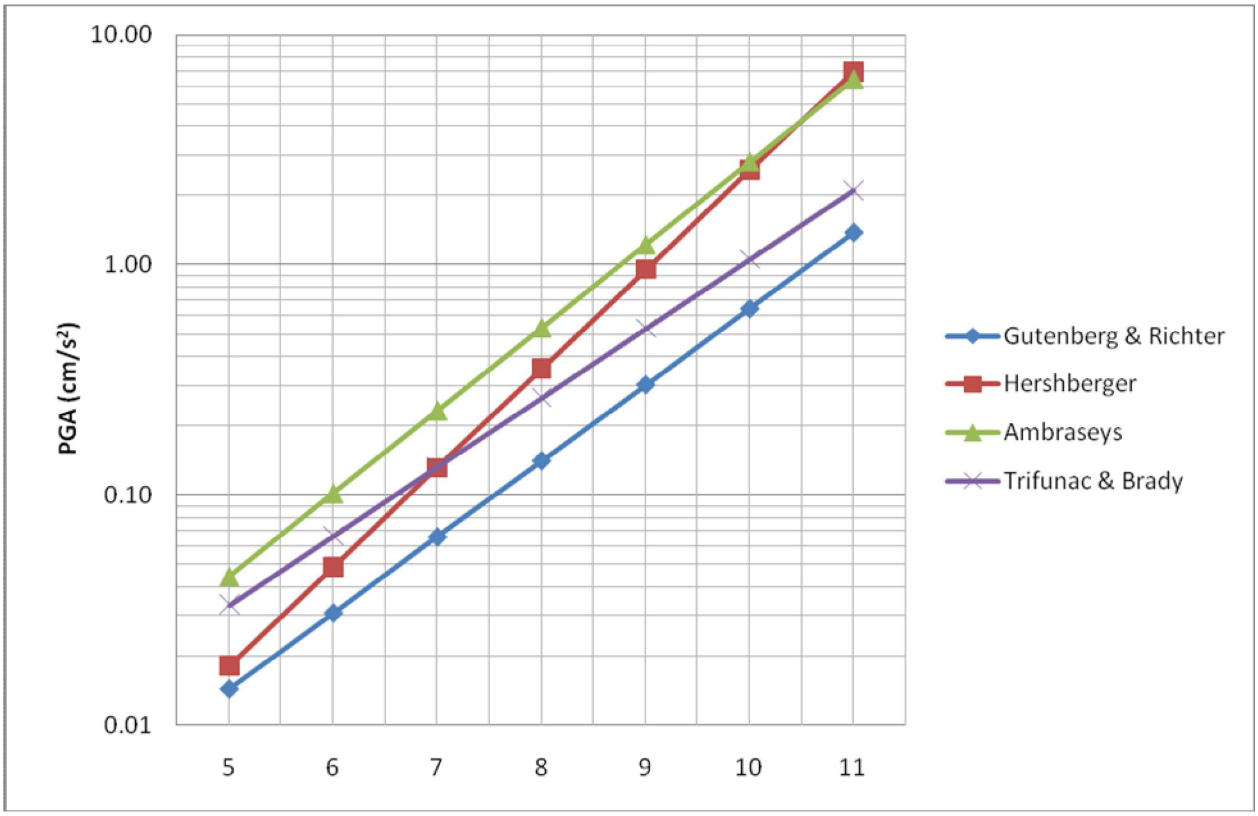



**Figure 3.5 Comparison of PGA values derived from different modified Mercalli intensity (PGA-MMI) empirical relationships.**

### 3.3 SELECTION OF PEAK GROUND VELOCITY (PGV) VALUES FROM INTENSITY

A number of researchers have also developed PGV-MMI relationships. In each of the relationships given below, I is Modified Mercalli intensity and V is peak ground velocity in cm/sec. Trifunac and Brady (1975), considered horizontal and vertical components separately.

Trifunac and Brady (1975) $\log V_v = -1.10+0.28\ I$ (3.5)

Trifunac and Brady (1975) $\log V_H = -0.63+0.25\ I$ (3.6)

Wald et. al. (1999) $I = 3.47 \log (PGV) + 2.35$ (3.7)

Using the above relationships (vertical component discarded), different PGV values calculated for different MM intensity and show in Table 3.3

MM intensity and PGV values from Table 3.3 recommended by Trifunac and Brady (1975) and Wald (1999) have also been plotted in Figure 3.5 for comparison. It can be seen from the two plot that Trifunac and Brady gives higher value than Wald et. al.

**Table 3.3: PGV values based on different existing empirical relationships for different intensity**

| Intensity (MMI) | Trifunac and Brady (1975) | Wald et al (1999) |
|---|---|---|
| 5 | 4.169 | 5.803 |
| 6 | 7.413 | 11.269 |
| 7 | 13.183 | 21.881 |
| 8 | 23.442 | 42.486 |
| 9 | 41.687 | 82.495 |
| 10 | 74.131 | 160.181 |
| 11 | 131.826 | 311.025 |

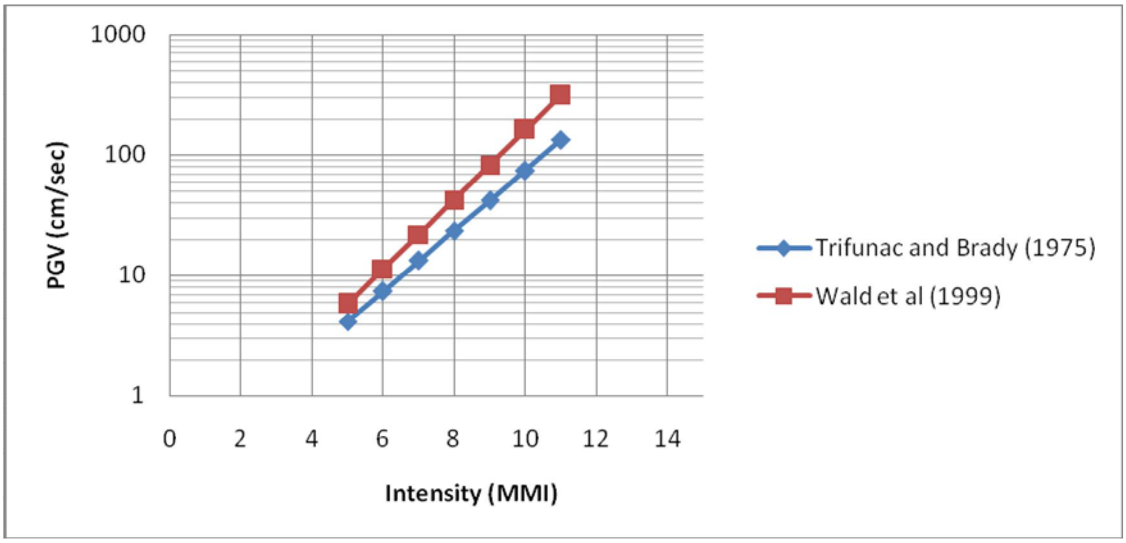



**Figure 3.6 Comparison of PGV values derived from different PGV-MMI empirical relationships.**

## 3.4 SELECTION OF DAMAGE ANALYSIS MFTHODS BASED ON PGA

Different pipeline fragility relation give very different predictions of pipeline damage rate for the same PGA value. The values presented in Table 3.4 are from katayama et. (1975), Isoyama and Katayama (1982), O'Rourke et al. (1998) and Isoyama et al. (2000) fragility relations (Equations 2.9, 2.10, 2.11 and respectively). All these relations assume CI pipe irrespective of diameters.

**Table 3.4: Pipeline repair rate from different fragility relations for PGA values based on Trifunac and Brady and Gutenber and Richter relations**

| Equation Used | Intensity (MMI) | Repair Rate (RR) | | | | |
|---|---|---|---|---|---|---|
| | | PGA ($cm/S^2$) | Katayama et al. (1975) | Isoyama and Katayama (1982) | O'Rourke et. al. (1998) | Isoyama et al. (2000) |
| Trifunac and Brady | 7 | 130.017 | 0.011 | 0.001 | 0.019 | 0.002 |
| | 8 | 259.418 | 0.911 | 0.072 | 0.044 | 0.063 |
| | 9 | 517.607 | 75.276 | 4.751 | 0.105 | 0.419 |
| | 10 | 1032.761 | 6218.090 | 312.448 | 0.250 | 2.041 |
| Gutenberg and Richter | 7 | 64.565 | 0.000 | 0.000 | 0.008 | 0.000 |
| | 8 | 138.038 | 0.016 | 0.002 | 0.020 | 0.004 |
| | 9 | 295.121 | 2.077 | 0.158 | 0.052 | 0.094 |
| | 10 | 630.957 | 266.807 | 15.774 | 0.135 | 0.673 |

From the selected peak ground acceleration (PGA) values and fragility relations, following four methods are used for damage analysis.

Method 1: In this method PGA and repair rate are based on Trifunac – Brady MMI-PGA relations and O'Rourke damage prediction relation.

Method 2: This method is based on Gutenberg-Richter MMI-PGA relation and O'Rourke damage prediction relation.

Method 3: This method involves Trifunac-Brad MMI-PGA relation and Isoyama damage prediction relation.

Method 4: Where damage analysis is based on Gutenberg-Richter MMI-PGA relation and Isoyama damage prediction relation.

### 3.5 GAS PIPELINE DAMAGE ANALYSIS BASED ON PGA

Pipeline damage estimation is related to damage prediction relationship. Relative results of different damage prediction relationships are studied in the previous articles. Using the methods outlined in the preceding articles and pipeline lengths calculated from the digitized maps which are shown in Table 3.1. Repair rates and number of repairs are worked out and presented in Table 3.5 to 3.8 and finally a different table, table 3.9 is prepared for comparison of these repair rates which are also presented in graph of Figure 3.7.

The result presented in Table 3.5 shows pipeline length, repair rate und number of repairs based on O'Rourke damage prediction relation for different peak ground acceleration (PGA) derived from Trifunac and Brady PGA-MMI relation. The result presented in Table 3.6 shows pipeline length, repair rate and number of repairs based on O'Rourke damage prediction relation for different peak ground acceleration (PGA) derived from Gutenberg and Richter PGA-MMI relation. The result presented in table 3.7 shows pipeline length, repair rate and number of repairs based on Isoyama damage prediction relation for different peak ground acceleration (PGA) derived from Trifunac & Brady PGA-MMI relation. The result presented in Table 3.8 shows pipeline length, repair rate and number of repairs based on Isoyama damage prediction relation for different peak ground acceleration (PGA) derived from Gutenberg and Richter PGA-MMI relation.

**Table 3.5: Intensity and number of repairs based on O'Rourke (1998) and Trifunac and Brady(1975) relation**

| Intensity (MMI) | PGA ($cm/s^2$) | Pipe length (km) | | | | | | | | | | Total Length (km) | Repair rate | Repair number |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 20 mm | 25 mm | 50 mm | 75 mm | 100 mm | 150 mm | 200 mm | 250 mm | 300 mm | N/A | | | |
| 8 | 259.42 | 10.03 | 92.33 | 167.02 | 33.72 | 24.3 | 23.8 | 15.8 | 0.00 | 0.00 | 9.04 | 376.03 | 0.044 | 17 |
| 9 | 517.61 | 1.27 | 18.58 | 8.14 | 5.53 | 4.06 | 3.35 | 19.1 | 1.50 | 1.55 | 4.24 | 67.31 | 0.105 | 7 |
| 10 | 1032.7 | 0.00 | 10.37 | 0.00 | 1.43 | 0.00 | 1.39 | 7.09 | 0.00 | 1.94 | 0.00 | 22.23 | 0.250 | 6 |

**Table 3.6: Intensity and number of repairs based on O'Rourke (1998) and Gutenberg-Richter (1942)**

| Intensity (MMI) | PGA ($cm/s^2$) | Pipe length (km) | | | | | | | | | | Total Length (km) | Repair rate | Repair number |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 20 mm | 25 mm | 50 mm | 75 mm | 100 mm | 150 mm | 200 mm | 250 mm | 300 mm | N/A | | | |
| 8 | 138.04 | 10.03 | 92.33 | 167.02 | 33.72 | 24.3 | 23.8 | 15.8 | 0.00 | 0.00 | 9.04 | 376.03 | 0.02 | 8 |
| 9 | 295.12 | 1.27 | 18.58 | 8.14 | 5.53 | 4.06 | 3.35 | 19.1 | 1.50 | 1.55 | 4.24 | 67.31 | 0.052 | 4 |
| 10 | 630.96 | 0.00 | 10.37 | 0.00 | 1.43 | 0.00 | 1.39 | 7.09 | 0.00 | 1.94 | 0.00 | 22.23 | 0.135 | 3 |

**Table 3.7: Intensity and number of repairs based on Isoyama (2000) and Trifunac and Brady (1975) relation**

| Intensity (MMI) | PGA (cm/s$^2$) | Pipe length (km) | | | | | | | | | | Total Length (km) | Repair rate | Repair number |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 20 mm | 25 mm | 50 mm | 75 mm | 100 mm | 150 mm | 200 mm | 250 mm | 300 mm | N/A | | | |
| 8 | 259.42 | 10.03 | 92.33 | 167.02 | 33.72 | 24.3 | 23.8 | 15.8 | 0.00 | 0.00 | 9.04 | 376.03 | 0.063 | 24 |
| 9 | 517.61 | 1.27 | 18.58 | 8.14 | 5.53 | 4.06 | 3.35 | 19.1 | 1.50 | 1.55 | 4.24 | 67.31 | 0.419 | 28 |
| 10 | 1032.7 | 0.00 | 10.37 | 0.00 | 1.43 | 0.00 | 1.39 | 7.09 | 0.00 | 1.94 | 0.00 | 22.23 | 2.041 | 45 |

**Table 3.8: Intensity and number of repairs based on Isoyama (2000) and Gutenberg-Richter (1942)**

| Intensity (MMI) | PGA (cm/s$^2$) | Pipe length (km) | | | | | | | | | | Total Length (km) | Repair rate | Repair number |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 20 mm | 25 mm | 50 mm | 75 mm | 100 mm | 150 mm | 200 mm | 250 mm | 300 mm | N/A | | | |
| 8 | 138.04 | 10.03 | 92.33 | 167.02 | 33.72 | 24.3 | 23.8 | 15.8 | 0.00 | 0.00 | 9.04 | 376.03 | 0.004 | 1 |
| 9 | 295.12 | 1.27 | 18.58 | 8.14 | 5.53 | 4.06 | 3.35 | 19.1 | 1.50 | 1.55 | 4.24 | 67.31 | 0.094 | 6 |
| 10 | 630.96 | 0.00 | 10.37 | 0.00 | 1.43 | 0.00 | 1.39 | 7.09 | 0.00 | 1.94 | 0.00 | 22.23 | 0.673 | 15 |

**Table 3.9: Intensity and number of repairs based on various PGA based relation.**

| Intensity (MMI) | Pipeline Repair Rate (RR) based on relations: | | | |
|---|---|---|---|---|
| | O'Rourke (1998) and Trifunac and Brady (1975) (OTB) | O'Rourke (1998) and Gutenberg-Richter (1942) (OGR) | Isoyama (2000) and Trifunac and Brady (1975) (ITB) | Isoyama (2000) and Gutenberg-Richter (1942) (IGR) |
| 8 | 0.044 | 0.020 | 0.063 | 0.004 |
| 9 | 0.105 | 0.052 | 0.419 | 0.094 |
| 10 | 0.250 | 0.135 | 2.041 | 0.673 |

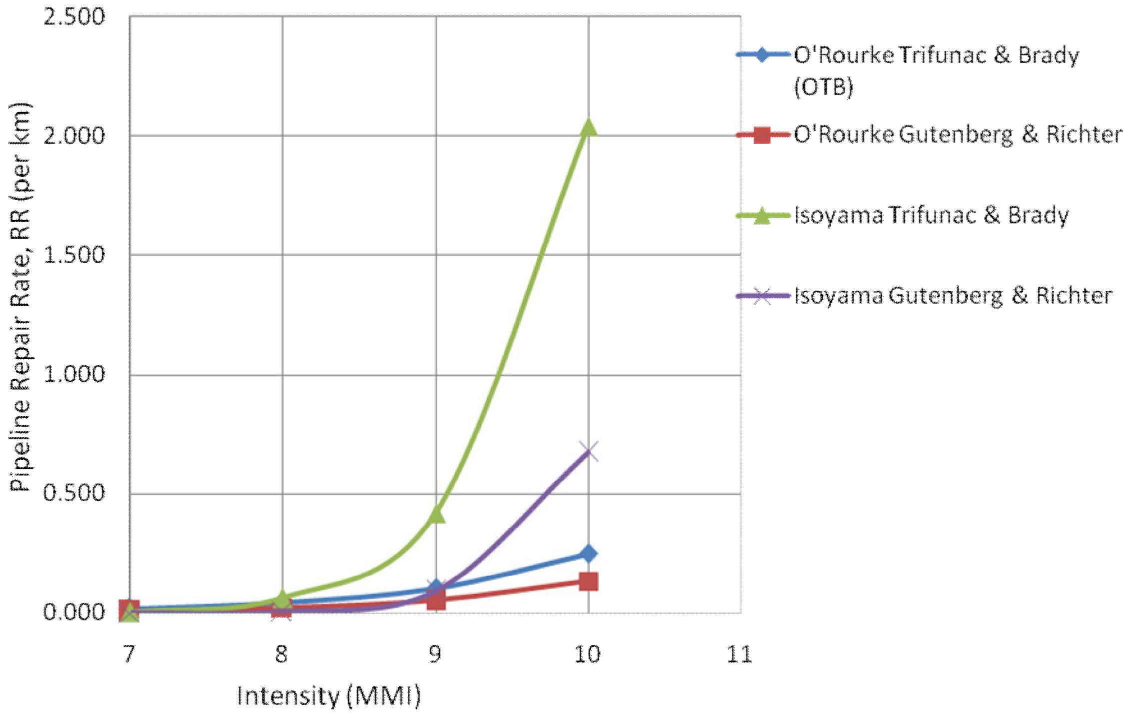



**Figure 3.7 Comparison of pipeline repair rate for different Intensity and fragility relation.**

From the table 3.9 and Figure 3.7 it is seen that repair rates of O'Rourke and Trifunac and Brady realtion are two times higher than those of O'Rourke and Gutenberg and Richter relation for all intensities. On the other hand Isoyama and Trifucan and Braddy relation shows 15 times higher repair rate than the repair rate obtained from Isoyama and Gutenberg and Richter relation for intensity 8 while it is 5 times and 3 times higher for intensity 9 and 10 respectively. On the whole it is seen from these relations that repair rates vary from 0.004 to 0.06 for intensity 8, 0.052 to 0.424 for intensity 9 and 0.134 to 2.029 for intensity 10.

However due to absence of real repair rate data in Bangladesh, it is advisable to use a range of repair rates instead of a single value in the event of damage analysis of buried water supply pipelines due to earthquake.

### 3.6 SELECTION OF DAMAGE ANALYSIS MFTHODS BASED ON PGV

Different pipeline fragility relation give very different predictions of pipeline damage rate for the same PGA value. The values calculated in Table 4.10 are from O'Rourke & Ayla (1993), Endinger et al. (1995, 1998), Isoyama et al. (2000) & O'Rourke et al. (2001).

**Table 3.10: Pipe line repair rate form different fragility relations for PGV values based on Trifunac and Brady and Wald et. al. relations.**

| Equation Used | Intensity (MMI) | Repair Rate (RR) | | | | |
|---|---|---|---|---|---|---|
| | | PGV (cm/sec) | O'Rourke & Ayala (1993) | Endinger et al. (1995, 1998) | Isoyama et al. (2000) | O'Rourke et al. (2001) |
| Trifunac and Brady (1975) | 7 | 13.183 | 0.033 | 0.027 | | |
| | 8 | 23.442 | 0.121 | 0.086 | 0.050 | 0.119 |
| | 9 | 41.687 | 0.442 | 0.267 | 0.222 | 0.287 |
| | 10 | 74.131 | 1.613 | 0.836 | 0.625 | 0.455 |
| Wald et al (1999) | 7 | 21.881 | 0.104 | 0.075 | 0.038 | 0.099 |
| | 8 | 42.486 | 0.461 | 0.278 | 0.231 | 0.293 |
| | 9 | 82.495 | 2.051 | 1.033 | 0.743 | 0.487 |
| | 10 | 160.181 | 9.128 | 3.843 | 2.010 | 0.681 |

From the selected peak ground acceleration (PGV) values and fragility relations, following four methods are used for damage analysis.

Method 1: In this method PGV and repair rate are based on Trifunac – Brady MMI-PGV relations and Isoyama damage prediction relation.

Method 2: This method is based on Wald et al MMI-PGV relation and O'Rourke damage prediction relation.

Method 3: This method involves Trifunac-Brad MMI-PGV relation and Isoyama damage prediction relation.

Method 4: Where damage analysis is based on Wald et al MMI-PGV relation and O'Rourke damage prediction relation.

### 3.7 GAS PIPELINE DAMAGE ANALYSIS BASED ON PGV

Using the methods outlined in the preceding articles and pipeline lengths calculated from the digitized maps which are shown in Table 3.1. Repair rates and number of repairs are worked out and presented in Table 3.11 to 3.14 and finally a different table, table 3.15 is prepared for comparison of these repair rates which are also presented in graph of Figure 3.8

**Table 3.11: Intensity and number of repairs based on O'Rourke (2001) and Trifunac and Brady (1975) relation**

| Intensity (MMI) | PGV (cm/s) | Pipe length (km) | | | | | | | | | | Total Length (km) | Repair rate | Repair number |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 20 mm | 25 mm | 50 mm | 75 mm | 100 mm | 150 mm | 200 mm | 250 mm | 300 mm | N/A | | | |
| 8 | 23.44 | 10.03 | 92.33 | 167.02 | 33.72 | 24.29 | 23.79 | 15.81 | 0.00 | 0.00 | 9.04 | 376.03 | 0.119 | 45 |
| 9 | 41.69 | 1.27 | 18.58 | 8.14 | 5.53 | 4.06 | 3.35 | 19.09 | 1.50 | 1.55 | 4.24 | 67.31 | 0.287 | 19 |
| 10 | 74.13 | 0.00 | 10.37 | 0.00 | 1.43 | 0.00 | 1.39 | 7.09 | 0.00 | 1.94 | 0.00 | 22.23 | 0.455 | 10 |

**Table 3.12: Intensity and number of repairs based on O'Rourke (2001) and Wald (1999)**

| Intensity (MMI) | PGV (cm/s) | Pipe length (km) | | | | | | | | | | Total Length (km) | Repair rate | Repair number |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 20 mm | 25 mm | 50 mm | 75 mm | 100 mm | 150 mm | 200 mm | 250 mm | 300 mm | N/A | | | |
| 8 | 42.49 | 10.03 | 92.33 | 167.02 | 33.72 | 24.29 | 23.79 | 15.81 | 0.00 | 0.00 | 9.04 | 376.03 | 0.293 | 110 |
| 9 | 82.49 | 1.27 | 18.58 | 8.14 | 5.53 | 4.06 | 3.35 | 19.09 | 1.50 | 1.55 | 4.24 | 67.31 | 0.487 | 33 |
| 10 | 160.18 | 0.00 | 10.37 | 0.00 | 1.43 | 0.00 | 1.39 | 7.09 | 0.00 | 1.94 | 0.00 | 22.23 | 0.681 | 15 |

**Table 3.13: Intensity and number of repairs based on Isoyama (2000) and Trifunac and Brady (1975) relation**

| Intensity (MMI) | PGV (cm/s) | Pipe length (km) | | | | | | | | | | Total Length (km) | Repair rate | Repair number |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 20 mm | 25 mm | 50 mm | 75 mm | 100 mm | 150 mm | 200 mm | 250 mm | 300 mm | N/A | | | |
| 8 | 23.44 | 10.03 | 92.33 | 167.02 | 33.72 | 24.29 | 23.79 | 15.81 | 0.00 | 0.00 | 9.04 | 376.03 | 0.050 | 19 |
| 9 | 41.69 | 1.27 | 18.58 | 8.14 | 5.53 | 4.06 | 3.35 | 19.09 | 1.50 | 1.55 | 4.24 | 67.31 | 0.222 | 15 |
| 10 | 74.13 | 0.00 | 10.37 | 0.00 | 1.43 | 0.00 | 1.39 | 7.09 | 0.00 | 1.94 | 0.00 | 22.23 | 0.625 | 14 |

**Table 3.14: Intensity and number of repairs based on Isoyama (2000) and Wald (1999)**

| Intensity (MMI) | PGV (cm/s) | Pipe length (km) | | | | | | | | | | Total Length (km) | Repair rate | Repair number |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 20 mm | 25 mm | 50 mm | 75 mm | 100 mm | 150 mm | 200 mm | 250 mm | 300 mm | N/A | | | |
| 8 | 42.49 | 10.03 | 92.33 | 167.02 | 33.72 | 24.29 | 23.79 | 15.81 | 0.00 | 0.00 | 9.04 | 376.03 | 0.231 | 87 |
| 9 | 82.49 | 1.27 | 18.58 | 8.14 | 5.53 | 4.06 | 3.35 | 19.09 | 1.50 | 1.55 | 4.24 | 67.31 | 0.743 | 50 |
| 10 | 160.18 | 0.00 | 10.37 | 0.00 | 1.43 | 0.00 | 1.39 | 7.09 | 0.00 | 1.94 | 0.00 | 22.23 | 2.010 | 45 |

**Table 3.15: Intensity and number of repairs based on various PGV based relation.**

| Intensity (MMI) | Pipeline Repair Rate (RR) based on relations: | | | |
|---|---|---|---|---|
| | Isoyama et al. (2000) & Trifunac Brady (1975) | O'Rourke et al. (2001) & Trifunac Brady (1975) | Isoyama et al. (2000) & Wald (1999) | O'Rourke et al. (2001) & Wald (1999) |
| 7 | 0.000 | 0.000 | 0.038 | 0.099 |
| 8 | 0.050 | 0.119 | 0.231 | 0.293 |
| 9 | 0.222 | 0.287 | 0.743 | 0.487 |
| 10 | 0.625 | 0.455 | 2.010 | 0.681 |

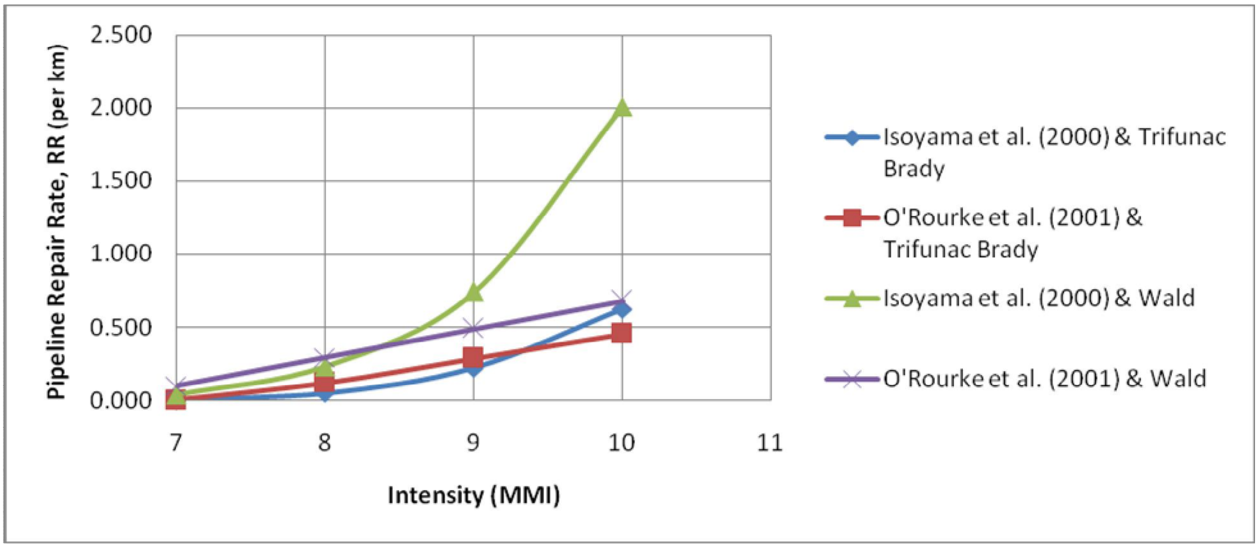



**Figure 3.8 Comparison of pipeline repair rate for different Intensity and PGV based fragility relation.**

### 3.8 Comparison of Repair rate

From above analysis a comparison of pipeline repair rate based on PGV and PGA has done and presented in figure 3.9 figure 3.10 & figure 3.11. Although pipe line repair rate of various fragility relations are available but fragility relations based on PGA is more acceptable in context of Bangladesh since all data available are based on PGA. But PGV based fragility relation is more reliable than PGA as discussed in chapter 2.

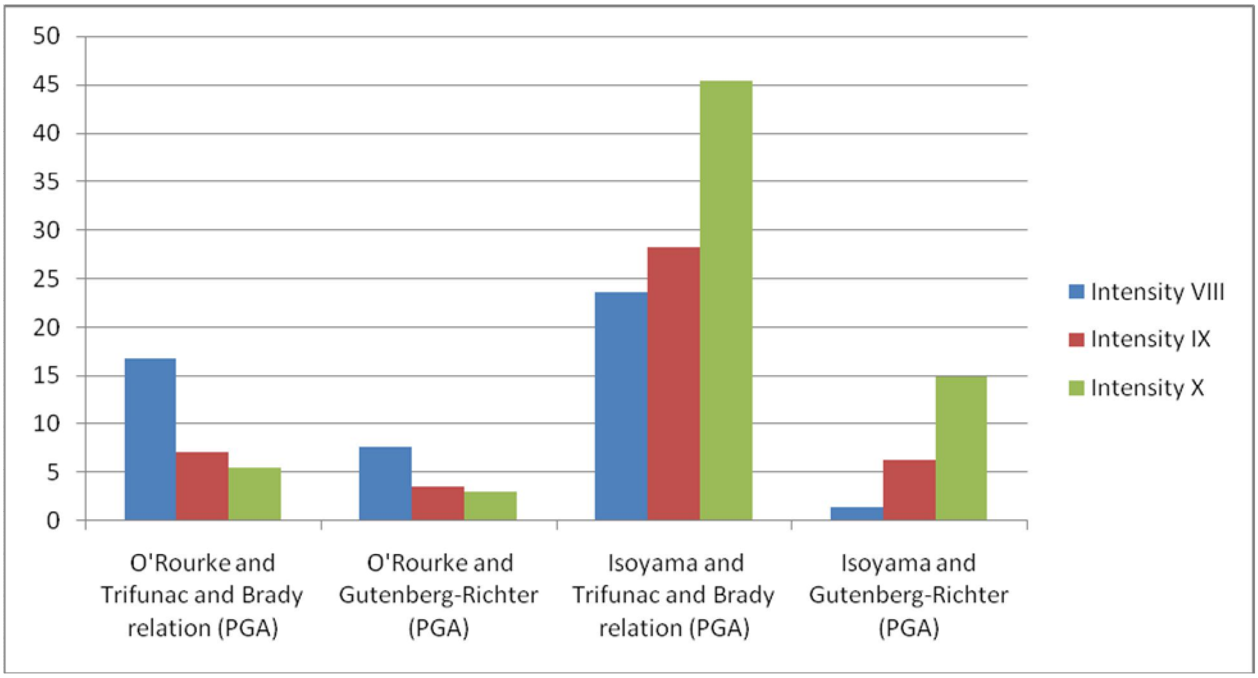



**Figure 3.9 Comparison of pipeline repair rate for different Intensity and PGA based fragility relation.**

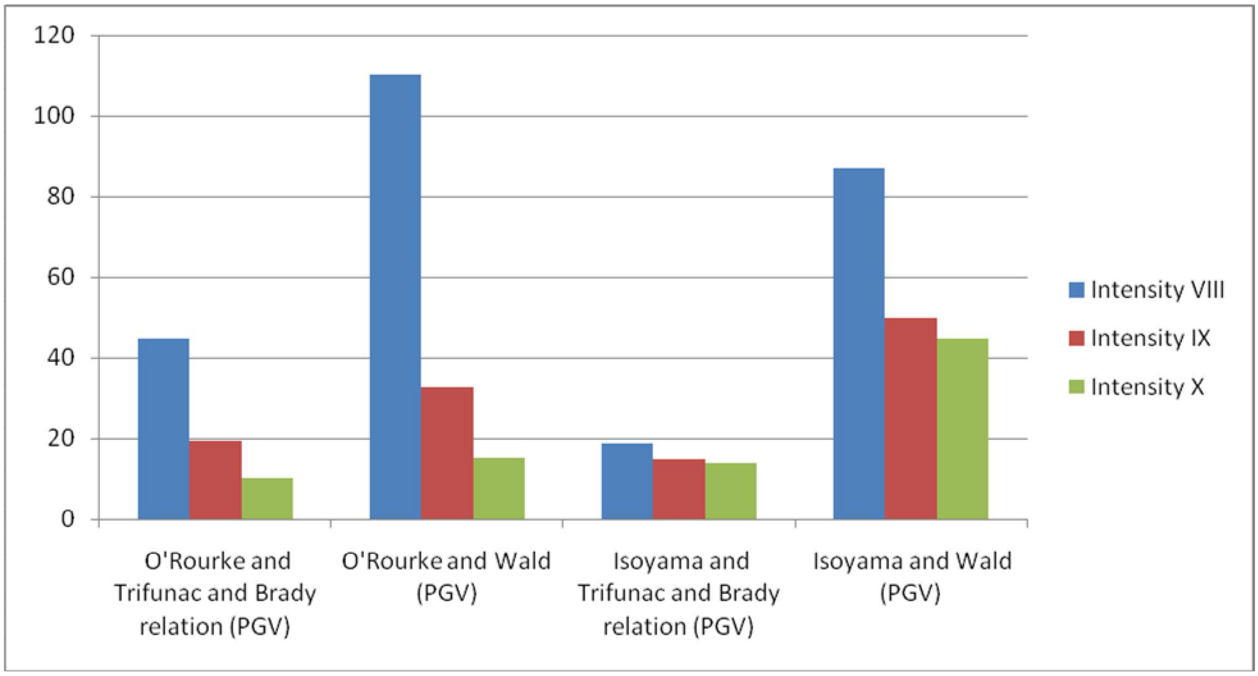
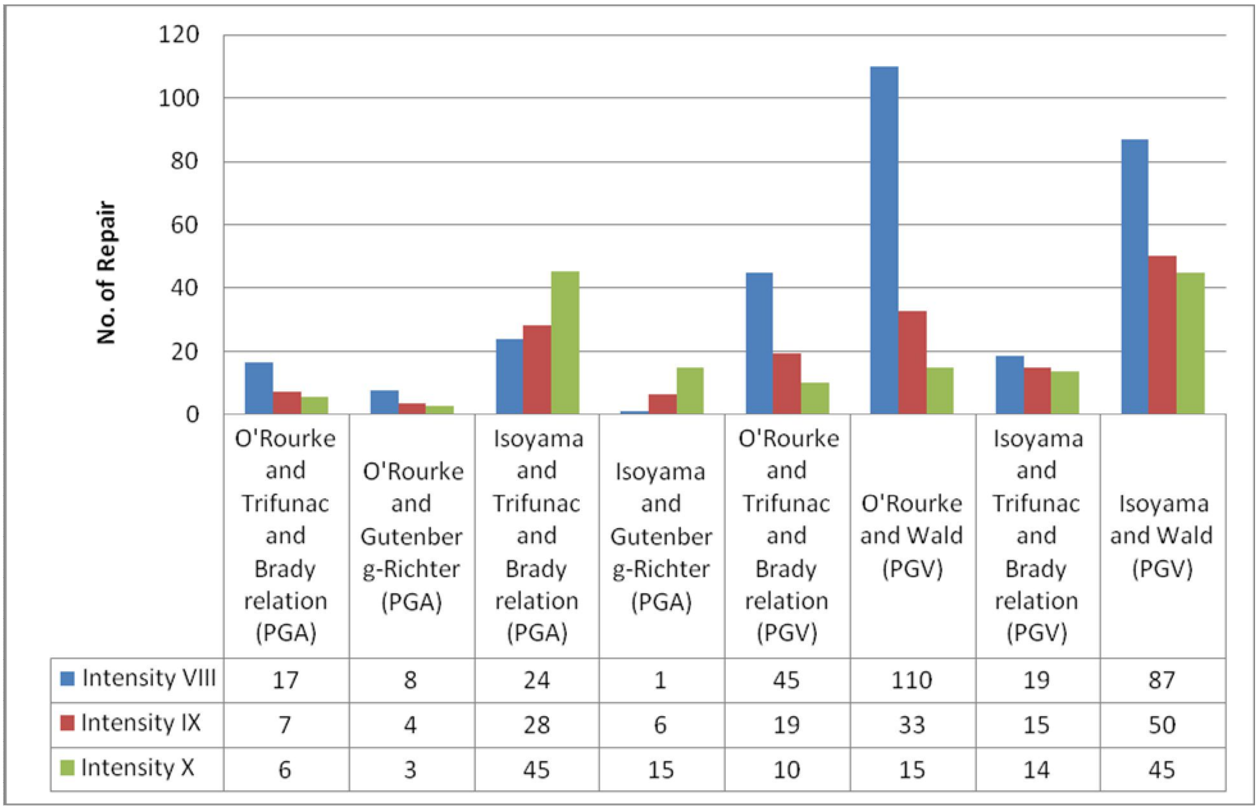



**Figure 3.10 Comparison of pipeline repair rate for different Intensity and PGV based fragility relation.**




| | O'Rourke and Trifunac and Brady relation (PGA) | O'Rourke and Gutenberg-Richter (PGA) | Isoyama and Trifunac and Brady relation (PGA) | Isoyama and Gutenberg-Richter (PGA) | O'Rourke and Trifunac and Brady relation (PGV) | O'Rourke and Wald (PGV) | Isoyama and Trifunac and Brady relation (PGV) | Isoyama and Wald (PGV) |
|---|---|---|---|---|---|---|---|---|
| Intensity VIII | 17 | 8 | 24 | 1 | 45 | 110 | 19 | 87 |
| Intensity IX | 7 | 4 | 28 | 6 | 19 | 33 | 15 | 50 |
| Intensity X | 6 | 3 | 45 | 15 | 10 | 15 | 14 | 45 |

**Figure 3.11 Comparison of pipeline repair rate both for PGV & PGA based fragility relation for different Intensity at Dhaka City.**

### 3.9 ESTIMATION OF THE EXPECTED MONETARY LOSSES

In this study only the direct loss is taken into account. The direct loss for the gas pipeline consists of two major parts: (1) The vented gas cost and (2) The repair cost.

#### 3.9.1 THE VENTED GAS COST

Usually repairing the damaged gas pipeline includes welding procedure and grinding for polishing the surface. As the natural gas is explosive, it is necessary to completely vent the pipeline section gas before welding procedure. The isolation and then ventilation tasks are applicable by using Line Break Valve (LBV) among the pipeline in each 20 km (M. Hesari, M. Mousavi and A. Azarbakht, 2012**)**. This type of valve has the ability to sense pipeline breaks at the upstream or downstream and shuts off the line immediately in the associated section. In the case of line break or line leak, the closest two LBVs to the damaged joint isolate the pipe section and only the containing gas of this section is vented for the purpose of repairing. Therefore this amount of vented gas is the wasted gas. By assuming the average working pressure of pipeline equal to 55 bars and natural gas as an ideal gas, the wasted gas volume can be calculated as written in Eqn. 3.8

$$V = \frac{\pi D^2}{4} * L * P_{ave} \tag{3.8}$$

Where V is the volume of gas with standard pressure (1 bar), D is pipe diameter, L is length of pipe and $P_{ave}$ is average working pressure. Usually pipe having diameter 20 mm to 100 mm is subjected to low pressure as 15 bar (200 psi) and 150 to 200 mm is subjected to relatively high pressure like 55 bars. Considering this the amount of vented gas has been calculated and presented in table 3.16.

**Table 3.16: Vented gas cost within pipe**

| Pipe dia (mm) | Length in KM intensity 8 | Length in KM intensity 9 | Length in KM intensity 10 | Volume of gas usinig equation 4.6 (in cum) |
|---|---|---|---|---|
| 20 | 10.03 | 1.27 | - | 2.34 |
| 25 | 92.33 | 18.58 | 10.37 | 39.23 |
| 50 | 167.02 | 8.14 | - | 226.60 |
| 75 | 33.72 | 5.53 | 1.43 | 118.44 |
| 100 | 24.29 | 4.06 | - | 146.70 |
| 150 | 23.79 | 3.35 | 1.39 | 1,217.61 |
| 200 | 15.81 | 19.09 | 7.09 | 3,187.39 |
| 250 | - | 1.50 | - | 177.30 |
| 300 | - | 1.55 | 1.94 | 596.00 |
| | | | Total | 5,711.59 |

Cost considering Tk 50/cum is Tk. 2.86 million

### 3.9.2 THE PIPELINE REPAIR COST

Generally when a pipe is damaged due to ground failure, the type of damage is likely to be a break while when a pipe is damaged due to seismic wave propagation, the type of damage is likely to be leak. In the loss of methodology, it is assumed that damage due to seismic waves will consist of 80% leaks and 20% breaks, while damage due to ground failure will consist of 20% leaks and 80% breaks. As damage due to ground failure is beyond the scope of this work it is assumed that 80% leak and 20% breaks will occur.

Pipeline repair cost is approximately 5 to 7 times more than the construction procedure as a consequence of the mobilization costs, machinery transfer for each repair and lack of time for pressurizing the line after repair to make the line alive. Usually the cost of construction of 300 mm dia pipe is Tk. 12 million/km. This has been found from information of "News watch of National Geographic" and by article published in "Daily Star" analyzing the proposed India Myanmar natural gas pipeline project through Bangladesh (Daily Star, June 17, 2013). From above rate analysis it is assumed that repair cost for 20mm to 150mm diameter pipe is Tk. 45 million/km and for 200mm to 300 mm diameter pipe Tk.60 million/km. Based on this data pipe line repair cost for both case of PGA and PGV based analysis has been furnished in table 3.17 & 3.18

**Table 3.17: Pipe line repair cost from PGA base fragility relations.**

| Pipe dia (mm) | No of repair | Repair cost / point | Total repair cost (in million of Tk.) |
|---|---|---|---|
| 20 | 2 | 45,000.00 | 0.09 |
| 25 | 36 | 45,000.00 | 1.62 |
| 50 | 15 | 45,000.00 | 0.68 |
| 75 | 9 | 45,000.00 | 0.41 |
| 100 | 4 | 45,000.00 | 0.18 |
| 150 | 7 | 45,000.00 | 0.32 |
| 200 | 24 | 60,000.00 | 1.44 |
| 250 | 1 | 60,000.00 | 0.06 |
| 300 | 5 | 60,000.00 | 0.30 |
| | | Total | 5.09 |

Total loss is Tk.2.86 million (vented gas) + Tk. 5.09 million (Lline repair) = Tk. 8.00 million for PGA based relation.

**Table 3.18: Pipe line repair cost from PGV base fragility relations.**

| Pipe dia (mm) | No of repair | Repair cost / point | Total repair cost (in million of Tk.) |
|---|---|---|---|
| 20 | 4 | 45,000.00 | 0.18 |
| 25 | 57 | 45,000.00 | 2.57 |
| 50 | 46 | 45,000.00 | 2.07 |
| 75 | 16 | 45,000.00 | 0.72 |
| 100 | 10 | 45,000.00 | 0.45 |
| 150 | 12 | 45,000.00 | 0.54 |
| 200 | 34 | 60,000.00 | 2.04 |
| 250 | 2 | 60,000.00 | 0.12 |
| 300 | 6 | 60,000.00 | 0.36 |
| | | Total | 9.05 |

Total loss is Tk.2.86 million (vented gas) + Tk. 9.05 million (Lline repair) = Tk. 12.00 million for PGV based relation.

### 3.10 SUMMARY

The pipeline network is very important for daily life in Dhaka city like elsewhere. It must be kept well maintained, especially to the city. It offers basic need, but it can be greatly damaged by earthquake. In Order to predict the damage of gas pipeline network after earthquake, the fragility curves are very useful means to do so. Different available pipeline fragility relations such as Katayama (1975), O'Rourke (1982), Isoyama and Katayama (1998) and Isoyama (2000) are compared. Finally using two relations namely O'Rourke (1982) and Isoyama (2000) damage rate of pipe lines is determined where PGA/PGV values obtained from Trifunac and Brady, Guutenberg and Richter & Wald MMI-PGA/PGV relations are used.

Pipeline damage rate is expressed in number of repairs per unit length of pipe. No of repairs for both PGA and PGV based fragility relations has been furnished and compared. It has been found that PGV based fragility relation results higher number of repair than PGA.

Any hazard especially earthquake hazard invokes financial involvement. So monetary loss estimation directly related to pipeline damage due to earthquake is a first step to mitigate the risk. This study also presented a picture of monetary loss due to earthquake damage of gas pipeline network for different intensity. Total cost estimated for damage of pipeline is Tk-8.0 million. Though this figure of amount apparently is not large, secondary or indirect loss due to damage of buried gas supply pipelines may be greater, such as interrupted gas supply to business and industrial sectors cause reduction of output. The fire hazard following gas leakage may be tremendous in many cases.

# CHAPTER FOUR
# CONCLUSIONS AND RECOMMENDATIONS

## 4.1 CONCLUSIONS

The main purpose of this project was to study the damage analysis of buried gas supply pipelines of Dhaka city subject to earthquake and make an attempt for quantification of the problem in term of monetary loss.

Researchers have developed relationships between pipeline damage and various seismic and geotechnical parameters using empirical data. The seismic performance of buried gas supply pipelines has been investigated by means of detailed review of these existing empirical pipeline fragility relations such as Katayama (1975, O'Rourke (1982), Isoyama and Katayama (1998) and Isoyama (2000). In the process geograpchic information system (GIS) was used and several maps of pipeline networks were prepared to develop database on pipeline networks. These maps and database are shown at various stages of this study.

The major findings and conclusions drawn from various aspects of the study are summarized below:

- On the basis of MM intensity the whole Dhaka city has been divided into three different zones. Out of total area of 135 sq.km, 88 sq.km is (65%) of intensity VIII, 39 sq.km is (29%) of intensity IX and remaining 9 sq.km is (6%) of intensity X.

- The available empirical relationships between modified Mercalli Intensity (MMI) and peak ground acceleration (PGA) have been studied. Also modified Mercalli Intensity (MMI) and peak ground velocity (PGV) have been studied. Different relation shows different PGV/PGA for same intensity.

- Different pipeline fragility relations give different prediction of pipe line damage rate per km for same PGA/PGV value. For PGA based relations it is from 0.004 to 0.063 for intensity 8, 0.094 to 0.419 for intensity 9 and 0.135 to 2.01 for intensity 10. For PGV based relations it is from 0.05 to 0.293 for intensity 8, 0.222 to 0.743 for intensity 9 and 0.455 to 2.01 for intensity 10.

- From the digitized pipeline network, the length of 20mm, 25mm, 50mm, 75mm, 100mm, 150mm, 200mm, 250mm, 300mm diameter is found 11.3km, 121.28km, 175,16km, 40.69km, 28.35km, 28.52km, 42km, 1.5km, 3.49km. Again with respect to intensity 376 km is found in intensity 8, 67.31 km found in intensity 9 and 22.23 km found in intensity 10.

- No of repair for PGV based relations give higher value than PGA. In case of PGA based fragility relation highest number of repair is 24, 28 and 45 for intensity 8, 9 and 10 respectively. For PGV based fragility relations highest number of repair is 110, 33 and 15 for intensity 8, 9 and 10 respectively.

- For PGA base fragility relation direct loss due to damage of pipeline in term of money is Tk. 8.00 million. Among this amount Tk. 2.86 million is cost of vented gas and Tk. 5.09 million is pipeline repair cost. For PGV base fragility relation direct loss due to damage of pipeline in term of money is Tk. 12.00 million. Among this amount Tk. 2.86 million is cost of vented gas and Tk. 9.05 million is pipeline repair cost. Though this figure of amount apparently is not large, secondary or indirect loss due to damage of buried gas supply pipelines may be greater, such as interrupted gas supply to business and industrial sectors cause reduction of output. The fire hazard following gas leakage may be tremendous in many cases.

## 4.2 RECOMMENDATIOS FOR FURTHER STUDY

Lifeline earthquake engineering is relatively a new one especially in a country like Bangladesh. In this study only the damage of pipeline within Dhaka city has been done based PGA and PGV based fragility relations. The following recommendation can be made for further study:

1. Fragility relations based on ground permanent ground displacement has not been used. So there is wide scope to do the same base of PGD.
2. This study was only with the gas Pipelines of Dhaka city. Usually within city pipe diameter is small and less dangerous as gas pressure is not so high. But the main distribution line beyond city having larger diameter with high pressure gas is more vulnerable to earthquake. So vulnerability of these lines may be assessed due to earthquake.
3. The effect of size, material, joint type and age of pipe should be considered in further study.

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# APPENDIX