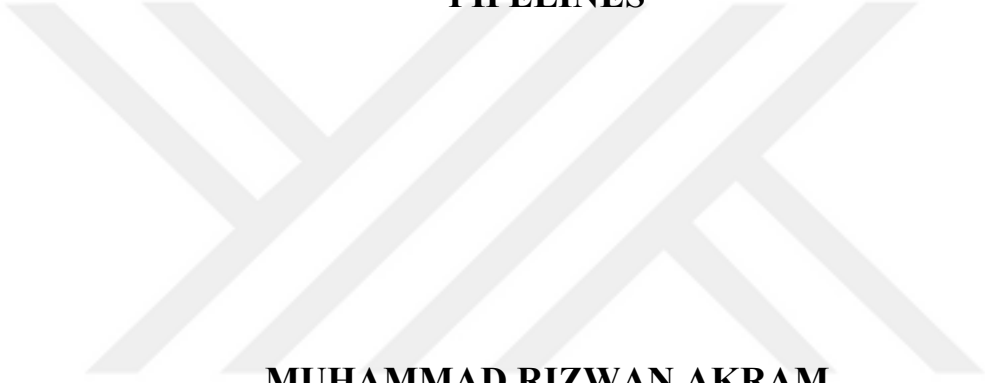


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**DANIŞMANI**  
**DOÇ. DR. ABDULLAH CAN ZÜLFİKAR**

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## DOKTORA JÜRİ ONAY FORMU

GTÜ Fen Bilimleri Enstitüsü Yönetim Kurulu'nun 09/12/2021 tarih ve 2021/58 sayılı kararıyla oluşturulan jüri tarafından 23/12/2021 tarihinde tez savunma sınavı yapılan MUHAMMAD RIZWAN AKRAM'ın tez çalışması İnşaat Mühendisliği Anabilim Dalında DOKTORA tezi olarak kabul edilmiştir.

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### ONAY

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## ÖZET

Sismik eğilimli alanlarda gömülü sürekli gaz boru hatları (BCGP'ler), özellikle BCGP'ler jeoteknik arızalar nedeniyle büyük eksenel gerilmeye maruz kalırsa, deprem nedeniyle segmentinde veya tüm şebekede sızıntılara veya kopmalara maruz kalabilmektedir. Mevcut tez, bir deprem sonrasında kentsel ölçekte BCGP'lerin riskini değerlendirmek için bir metodoloji sunmaktadır. Söz konusu bölge için sismik şiddet ölçümleri (IM'ler) tahmin edilmektedir. Bu IM'ler daha sonra, geçici dalga yayılımı (TWP) ve kalıcı zemin deformasyon tehlikeleri (PGD) nedeniyle BCGP'lerin hasarlarının davranışını analiz etmek için kullanılmaktadır. Risk değerlendirmesi, heyelan ve sıvılaşma üç fazlı modeller kullanılarak tamamlanmaktadır. Onarım ve bakım maliyetleri açısından kayıp rakamları, paydaşlar ve ağ yöneticileri için bir karar destekleyici araç olarak türetilmiştir. Ayrıca, 0,1g ila 1,5g arasında 0,5g aralıklarla ölçeklenen 10 kuvvetli yer hareketi (SGM) kaydı dikkate alınarak 3B sonlu elemanlar analizi (FEA) gerçekleştirilmektedir. Maksimum eksenel gerilme, her bir sismik yüke karşı BCGP'lerde hesaplanmaktadır. Sonuçlar, gömülü boruların ilk önce gevşek toprakta, pimli-pimli sınır koşullarında (BC'ler) ve sığ derinliklerde gömülü borularda hasara daha yatkın olduğunu ortaya koymaktadır. Mevcut tezde benimsenen prosedür, profesyonellere ve ilgili makamlara hızlı araştırma için basit ve ön tasarım yolu sağlamaktadır. Kentsel ölçekte yapısal bir sağlık izleme ve karar alma mekanizması geliştirilmiştir. Geliştirilen mekanizma senaryo ve/veya gerçek zamanlı olarak uygulanabilmektedir.

**Anahtar Kelimeler: Boru hatları, Deprem, Kalıcı Zemin Deformasyonları.**

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## LIST of ABBREVIATIONS and ACRONYMS

| <b><u>Abbreviations</u></b> | <b><u>Explanations</u></b>                         |
|-----------------------------|----------------------------------------------------|
| <b><u>and Acronyms</u></b>  |                                                    |
| $C_s$                       | : Shear wave velocity of ground surface layer      |
| $C_{r-ph}$                  | : Phase velocity of R-wave                         |
| $\alpha_\varepsilon$        | : Ground strain coefficient                        |
| $T_u$                       | : Axial force due to soil friction                 |
| $\lambda$                   | : Apparent wavelength of earthquake wave           |
| $A$                         | : Cross sectional area of buried pipe              |
| $E$                         | : Modulus of elasticity                            |
| $D$                         | : Outer diameter of pipe                           |
| $c$                         | : Co-efficient of cohesion of backfill soil        |
| $H$                         | : Burial depth of buried pipe                      |
| $\ddot{Y}$                  | : Effective soil unit weight                       |
| $\delta'$                   | : Interface frictional Angle between pipe and soil |
| $\phi$                      | : Soil internal friction angle                     |
| $f$                         | : Friction factor for various types of soil        |
| $K_0$                       | : Coefficient of soil pressure at rest             |
| $\alpha$                    | : Soil adhesion factor                             |
| $N_r, N_q, N_c$             | : Bearing capacity factors                         |
| $P_i$                       | : Internal gas pressure                            |
| $Z$                         | : Section modulus of pipe cross section            |
| SF                          | : Factor of safety for liquefaction                |
| $G$                         | : Shear modulus of soil                            |
| $K$                         | : Bulk modulus of soil                             |
| $\nu$                       | : Poisson's ratio                                  |
| $a_c$                       | : Critical acceleration                            |
| ALA                         | : American lifelines alliance                      |
| AWPV                        | : Apparent wave propagation                        |
| BCGPs                       | : Buried continuous gas pipelines                  |
| CIA                         | : Central intelligence agency                      |
| CRR                         | : Cyclic resistance ratio                          |

|        |   |                                                                            |
|--------|---|----------------------------------------------------------------------------|
| CSR    | : | Cyclic stress ratio                                                        |
| DLH    | : | Railway, ports and airports                                                |
| DS     | : | Damage states                                                              |
| DSHA   | : | Deterministic seismic hazard analysis                                      |
| EMME   | : | Earthquake model of middle east                                            |
| EWS    | : | Early warning system                                                       |
| GMPEs  | : | Ground motion prediction equations                                         |
| GSHAP  | : | Global seismic hazard project                                              |
| HDPE   | : | High density polyethylene                                                  |
| IMs    | : | Intensity measures                                                         |
| JSCE   | : | Japanese society of civil engineers                                        |
| LPI    | : | Liquidity potential index                                                  |
| MC     | : | Monte carlos                                                               |
| Mmax   | : | Maximum magnitude                                                          |
| NEHRP  | : | National earthquake hazard reduction program                               |
| NFM    | : | Nature of fault movement                                                   |
| PGA    | : | Peak ground acceleration                                                   |
| PGD    | : | Permanent ground deformation                                               |
| PGV    | : | Peak ground velocity                                                       |
| PSHA   | : | Probabilistic seismic hazard analysis                                      |
| RR     | : | Repair rate                                                                |
| Sa     | : | Spectral acceleration                                                      |
| SESAME | : | Seismotectonic and seismic hazard assessment of the<br>mediterranean basin |
| SGM    | : | Strong ground motion                                                       |
| SGS    | : | Strong ground shaking                                                      |
| SHARE  | : | Seismic hazard harmonization in europe                                     |
| TCIP   | : | Turkish catastrophe insurance pool                                         |
| TEFER  | : | Turkish emergency flood and earthquake recovery project                    |
| T-SHM  | : | Turkish seismic hazard map                                                 |
| TWP    | : | Transient wave propagation                                                 |

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# 1. INTRODUCTION

The progress of a nation depends on the reserves of their energy such as coal, petroleum, and gas. Among these, natural gas is the lowest hydrocarbon, odourless, colourless, and non-toxic form of energy. It offers heat for cooking and warmth environment for human livings, and it drives energy stations that supply electricity to homes and industries. Pipelines are usually used to carry gas from reserve points to the user's location as shown in Figure 1.1.

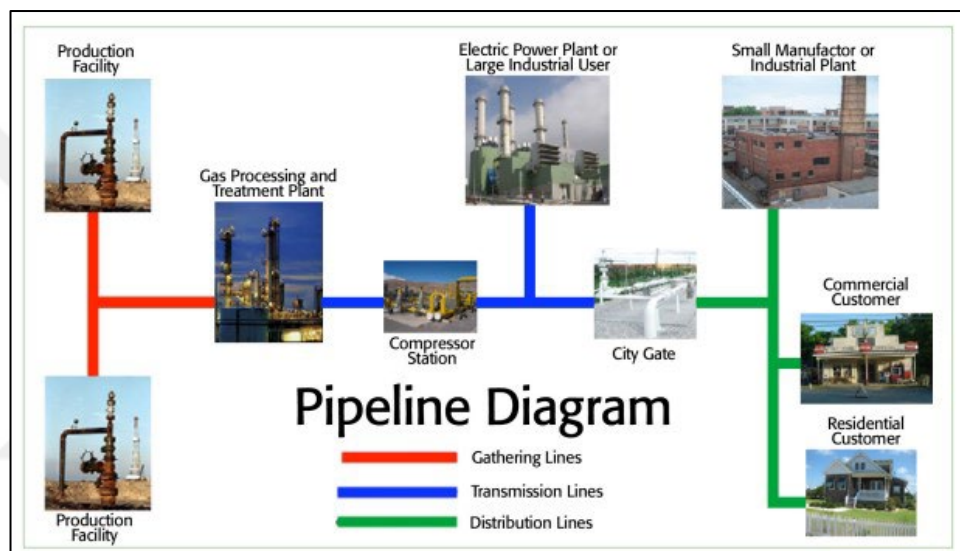


Figure 1.1: The gas pipeline transportation system from production to consumption.

Pipelines are of various kinds based on their geometry, physical properties, and chemical compositions. However, depending on their application, they are divided into two types i.e., segmented, and continuous pipelines as shown in Figure 1.2. Segmented pipes are used for transportation of water, sewage, oil, fluids etc. to short distances; and are commonly built of asbestos, cement, or concrete. On the other hand, continuous pipelines are used for long distances and are usually made up of steel or its composite alloys [O'Rourke and Liu, 1999]. These continuous pipelines are further classified into different kinds depending on the type of welds used for their connection. Buried steel pipeline with butt welded joints, bolted flanges, and High-Density Polyethylene (HDPE) pipelines with fused connection are some examples of buried continuous gas pipelines (BCGPs).

Around the world, many countries have dense network of BCGPs. For instance, in USA alone, total length of pipelines is 2,225,032 kms, following Russia and Canada as 259,913 kms and 100,000 kms, respectively. However, most of these countries such as California State in USA, European countries (Greece, Turkey, Italy, and the Balkans), Australia, New Zealand, Japan, all are considered as the highest seismic alert areas. In Turkey alone, there are total of 15,641 kms length of pipelines, of which 12,603 kms are gas pipelines, 3,038 kms are oil pipelines as per CIA recoded data accessed in June 2020 [Web1, 2021].



Figure 1.2: Segmented and continuous pipes.

In current research BCGPs are taken into consideration due to its extensive applications in gas industry. Pipes are usually buried due to aesthetic, security, and to avoid space problems. So, these pipelines are called as buried continuous pipelines.

## 1.1. Brief Literature Review

In the past, BCGPs were severely damaged due to seismic hazards such as transient wave propagation (TWP) and permanent ground deformation (PGD). Different kind of failure modes are observed in pipelines during past earthquakes. Many researchers [Ariman and Muleski, 1981], [O'Rourke et al., 1988], [Sun and Liang, 2000], [Chenna et al., 2014], [Sextos and Anastasios, 2017], [Psyrras et al., 2020] have made early attempts to review the performance of these pipelines. In addition, different seismic design codes such as Eurocode-8 [Eurocode-8, 2006], American Lifelines Alliance [ALA, 2001-2005], seismic design recommendation by Japan Society of Civil Engineers [JSCE, 2000], B-31 Code for pipes of high pressure

[ASME, 2012], and the Indian Seismic Design Code [I.I.T Kanpur, 2007] etc. have been prepared. Any interruption in the supply of gas pipelines can affect the usual life of society by stopping the energy demand in homes. The interruptions due to the earthquake can not only cause fire but also the explosions. Therefore, an urban earthquake risk assessment system should be prepared for the assessment of BCGPs that can generate risk and damage levels of buried infrastructure soon after the event. Such knowledge-based assessment tool then allows the decision makers to prioritize the inspections and interventions in more quick ways [Elsawah et al., 2016]. In civil engineering substructures, buried gas continuous pipelines (BCGPs) in seismic-prone regions may suffer leaks or breaks of its segment or as the whole network due to an earthquake, especially if the buried pipe is subjected to large axial strain due to geotechnical failures (e.g., landslide, fault crossing and/or liquefaction of soil).

The current thesis presents a methodology/ algorithm to evaluate the risk of BCGPs at an urban scale in the aftermath of an earthquake. The developed algorithm is then applied on a case study of BCGPs crossing through Marmara region. For this, seismic hazard analysis is carried in the East Marmara to center Izmit / Kocaeli regions. Seismic hazard methods such as, classical probabilistic seismic hazard analysis (PSHA), and PSHA with the Monte Carlos (MC) method are performed. Seismic intensity measures (IMs), such as peak ground acceleration (PGA) and spectral acceleration (Sa) are estimated for the subject region. These IMs are then used to analyze the behavior of BCGPs damages due to transient wave propagation (TWP) and permanent ground deformation hazards (PGD). Risk assessment is completed using three phase (susceptibility, triggering and PGD) models. Loss figures in terms of repair and maintenance costs are derived as a decision supporting tool for stakeholders and network managers.

Additionally, 3-D finite element analysis (FEA) is carried out using ABAQUS/CAE package considering rich strong ground motion (SGM) records, that are scaled at 0.1g to 1.5g with an interval of 0.5g. Maximum axial strain is computed in the selected pipe against each seismic load. The results reveal that buried pipes are more prone to damage first in loose soil than high plastic clayey, medium sand and sandstone respectively. It is also noticed that pinned-pinned boundary conditions (BCs) for pipe ends gain higher axial strain than the pinned-fixed, and fixed-fixed BCs. It is also observed that pipes buried at shallow depth are at higher risks to damage first than the deep ones.

The procedure adopted in current thesis provide a simple and preliminary design path to the professionals and the concerned authorities for quick investigation. A structural health monitoring and decision-making mechanism is developed at urban scale. The developed mechanism can be implemented in the scenario and / or real-time. The development of this study is expected to increase the reliability status of this country to a higher level.

## **1.2. Motivation**

In this thesis, the main objective is to develop an algorithm to evaluate the risk of BCGPs at an urban level after an earthquake, to review, analyze and classify the literature of this domain, to identify the gaps, derive an optional supporting tool for network authorities and stakeholders, and contribute for advances in future seismic risk assessment.

## **1.3. Outline of the Thesis**

The current thesis consists of total nine chapters and short explanation of each chapter is given as below.

Chapter 1 is the introductory part that includes the general view and past earthquake effects on BCGPs, gaps of the previous research and importance of the current thesis. It also includes aim and scope of the current study.

Chapter 2 describes the research methodology of the thesis. Research methodology is presented in the form of flow charts to clearly understand the main theme of the workflow.

Chapter 3 presents seismic hazard analysis for the region of the case study. In this chapter, evolution of all historical seismic zoning maps of Turkey are explained in detail. Further, the major projects (GSHAP, SESAME, TEFER, DLH, SHARE, EMME and T-SHM) are discussed with their workings in development of seismic zoning maps and its surrounding regions. Tectonics history and seismic activities of the (Kocaeli and Marmara) regions are described. PSHA is performed by adding the inputs as earthquake catalogue (EMME and SHARE models), de-clustering analysis and completeness analysis. The output in the form of peak ground acceleration (PGA),

spectral accelerations ( $S_a$ ) are determined. The seismic hazard analysis is completed using both classical PSHA of the SHARE and EMME models, PSHA by using Mont Carlos (MC) method of SHARE and EMME models. At the end, analysis is also performed with local TBDY 2018 catalog to check the correlations of national model with the international models.

In Chapter 4, the developed algorithm is applied for transient seismic wave propagation (TWP) analysis on the selected study.

In Chapter 5, PGD hazards on buried pipelines in earlier foremost earthquakes are explained in detail. Further, failure aspects of buried pipes in recent major earthquakes are enlisted. In the last section of Chapter-5, seismic safety of BCGPs due to PGD hazards (lateral spreading, liquefaction and fault crossings are studied based on maximum tensile strain values.

In Chapter 6 of the thesis study, a novel three-step geotechnical methodology is explained for the region, representing (1) first step of the susceptibility, (2) then secondly, triggering, and (3) the final phase of PGD estimation. The 1999 Kocaeli earthquake and the current gas network of Sakarya – Istanbul regions are used as a test-bed example for the risk methodology aimed at checking the pros and cons. In the last section of Chapter-6, vulnerability analysis in terms of repair rate per kms. and loss curves based on maintenance and replacement costs for BCGPs are discussed in detail.

Chapter 7 includes numerical analysis of BCGPs. Numerical simulations with addition of richer earthquake records data and various pipe inventory using ABAQUS finite element analysis is completed, and the output results are then compared with the ZeusNL that aid for analysis of BCGPs failure modules.

Chapter 8 of the thesis explains the development of structural health monitoring techniques in BCGPs. The procedure adopted and explained in the thesis is converted to a monitoring and early warning system. It has four phases. First phase is observation point. In this stage, strain accumulated in BCGPs due to seismic loading, internal gas pressure and temperature variations are observed. Further, nature of fault movement (NFM) and strength of earthquake based on strong motion (SGM) records are examined. The second phase is the data acquisition and processing unit that transfers the data to wireless communication system. Beidou satellite and 4G technology can be preferred to transfer the unreadable data to remote control receiving unit for display. The third phase is the analysis/display and management system. Display screen are

used for exhibition of earthquake early warning system (EWS), BGPs strain induced, leakage and location required for inspection. The output data can be shared by text messages, email, or smart watches. The fourth and last phase is the alarm system. If any parameter exceeds the threshold limit value (in case of tensile strain limiting value is taken as 0.003), alarm and light color will demonstrate the emergency.

Chapter 9 is the last chapter of the thesis that includes the conclusions of the research workings and references.



## **2. RESEARCH METHODOLOGY**

The research methodology plays a backbone role of the whole study. It presents the idea and flow of the research in the form of identification, selection, process, and examination of the topic under consideration. In a nutshell, the importance of this chapter is to represent the inclusive validity and reliability of the thesis study in the form of flow charts.

### **2.1. Seismic Performance Evaluation of BCGPs**

In recent studies, it is recommended that the performance of BCGPs should be checked against both static and dynamic loads before and after the design and operations of the substructures. In dynamic loads, the safe operation of gas pipelines should be carefully examined by using operating loads such as internal pressure, temperature variations, developed stresses and strain values. The main aim behind this thesis is to highlight the response and design guidelines for quick survey of buried pipes against seismic loads. The analysis and design benchmarks stated in this thesis involve the following research methodology.

Firstly, seismic performance evaluation of BCGPs is checked against two hazard types as transient wave propagation (TWP) and permanent ground deformation (PGD) hazard. The procedure adopted for TWP analysis and its application on the case study is presented in chapter-4 of this thesis. In PGD hazard analysis, the damage is further classified into two categories as abrupt failure or spatially distributed with the event. Spatially distributed failure can be due to liquefaction phenomena or seismic settlement of the soils. In current research, liquefaction effect on BCGPs is considered due to its impact observed in recent earthquakes. Seismic settlement is not considered as this phenomenon is very less common to be observed in the past earthquake reconnaissance. As liquefaction is the strength loss of soil particles with the addition of water in their pore holes, therefore, effect of both buoyancy and lateral spreading are examined. For abrupt failure analysis, response of BCGPs against fault crossing is studied. As landslide is a general term and it can trigger due to the seismic event, therefore, lateral spreading in the form of landslide is evaluated as longitudinal and transverse directions. The PGD hazard analysis is presented in chapter-5 of this thesis

to examine the seismic performance of BCGPs of the selected case study. The whole procedure in the form of flow chart is presented in Figure 2.1.

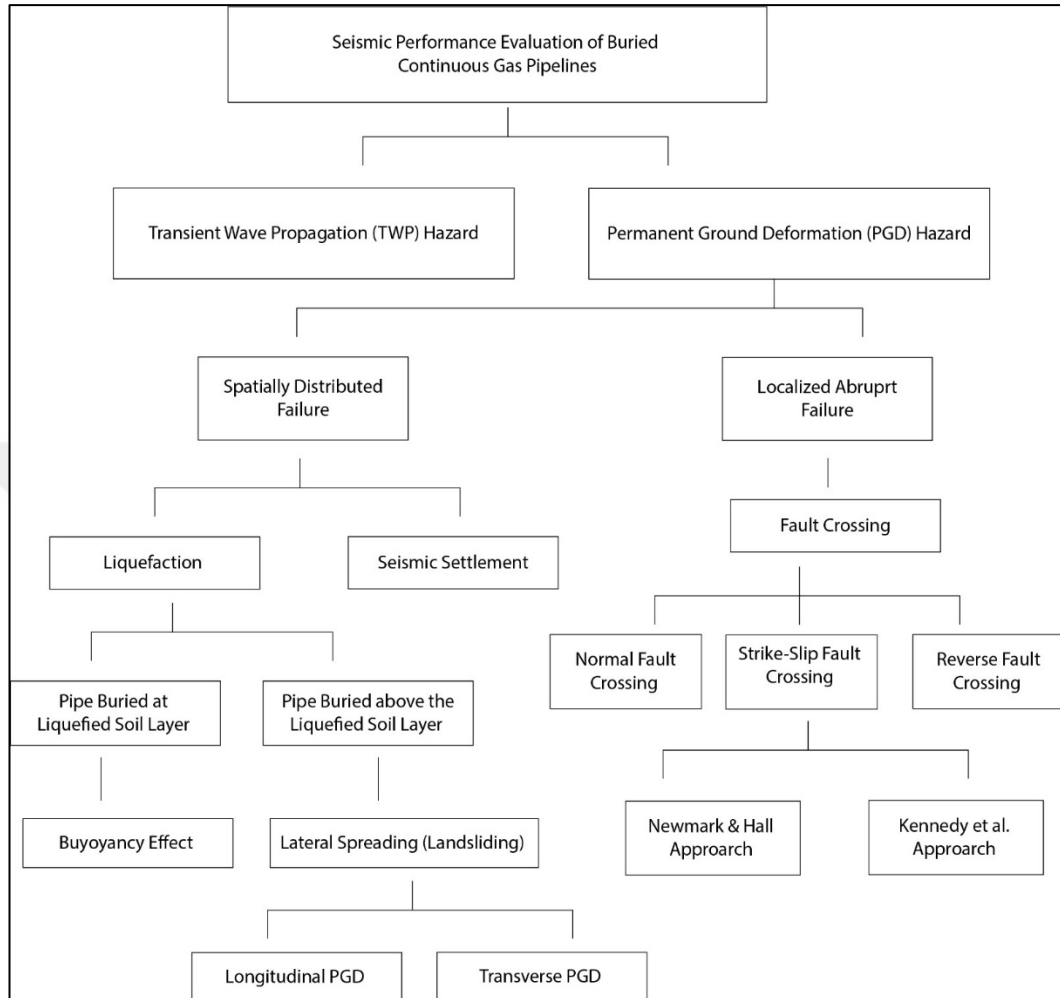


Figure 2.1: Seismic performance evaluation of BCGPs.

The input data required for seismic performance evaluation includes the below details.

### 2.1.1. Pipelines Data

Pipeline data includes buried pipelines geometry, connection types, material stress-strain relationship, pipeline operation and post seismic performances, exterior coating requirement, operating internal pressure, operating, and installing temperature

values, pipeline configuration features (layout, contour, site of installation, etc.), and lower strain threshold for present pipe inventory.

### **2.1.2. Site Details**

Site details include pipes buried depth, also termed as H value, geotechnical parameters (unit weight, void ratio, cohesion value, internal friction, and unit density), backfill soil properties and water table location.

### **2.1.3. Seismic Hazard Background**

Seismic hazard background can be anticipated seismic ground motion intensity, expected PGD extent, size, and length of BCGPs subjected to PGD, active fault data: expected fault dislocation, and alignment of pipeline with respect to path of fault dislocation.

## **2.2. Urban Earthquake Risk Assessment of Buried Gas Pipelines: Current Case Study**

The graphical representation of the case study selected for seismic analysis is shown in Figure 2.2. First part is the Turkish regional map with the buried gas pipelines network of the whole region. The second part presents the gas distribution network of Düzce – Gebze regions as a test-bed example. The material composition of pipes are BCGPs with outer diameters varying from 18 inches (0.4572 meters) to 36 inches (0.9144 metres). The yellow circles show the pumping station represent as pipe joint (meeting point of two pipe segments). The pipe elements (PE) are shown in blue colour and marked as PE-1 to PE-8 as of total 8 parts considered in this study.

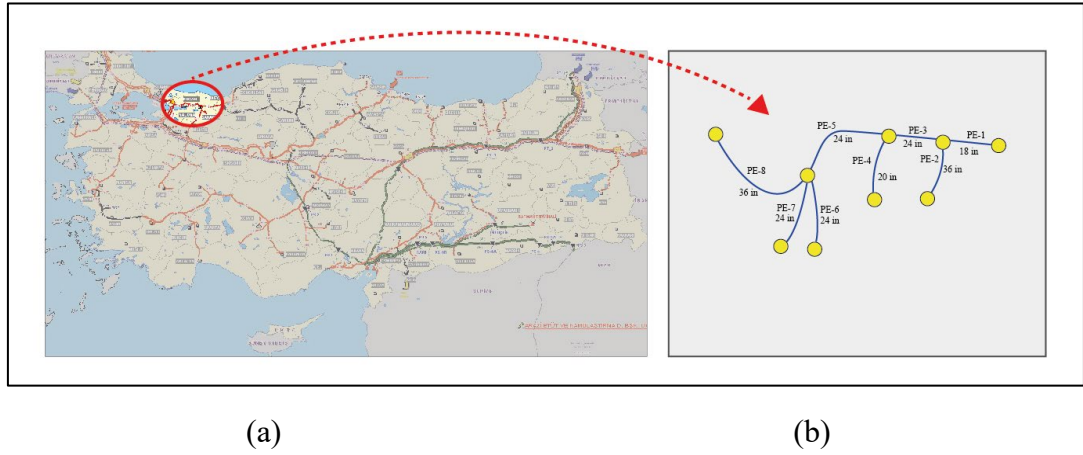


Figure 2.2: a) Turkish regional map and buried gas pipelines network b) High-pressure gas network of Düzce – Gebze regions as a test-bed scenario.

After selection of test bed scenario, the next procedure is to gather all physical pipelines data related to inventory (geometry, diameter, pressure, material, joint typology, burial depths etc.). In addition, the geotechnical site information of the buried pipelines in the form of elevation maps, slope, Vs30 profiles, geo-lithology, effective unit weight, density of each layer, ground water table depth etc. are considered in detail. Although it is difficult to receive the exact soil-pipe data from the piping company due to security reasons, however maximum data is collected from published reports, journal papers and news feeds of internet sources.

The next step is to examine the tectonic structure of the Marmara region, seismicity, and all historical seismic records. Based on this complete information, probabilistic seismic hazard analysis (PSHA) is performed considering the inputs factors such as earthquake catalogue, their de-clustering analysis, completeness analysis, source typologies for modeling seismicity distributions, magnitudes distribution and recurrence patterns, activity ratio, recurrence relations, maximum magnitude (Mmax), GMPEs, uncertainties, and logic tree application. PSHA of the region is completed using Monte Carlos (MC) method by using Open quake analysis program. Methodology for seismic hazard analysis is presented in Figure 2.3.

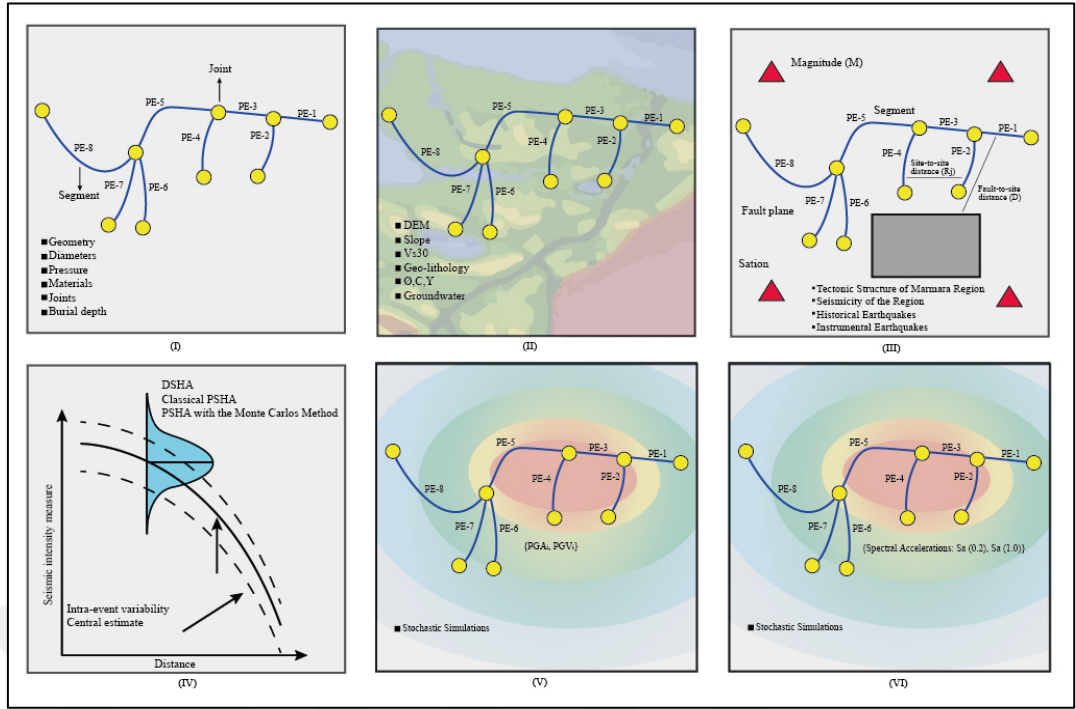


Figure 2.3: Methodology for seismic hazard evaluation: (I) pipe inventory, (II) available maps of the region, (III) hazard factors, (IV) GMPEs, (V) PGA, PGV, (VI) Sa (0.1), Sa (0.2).

After evaluating the seismic hazard parameters in the form of peak ground acceleration (PGA) and spectral acceleration (Sa) values of the concerned region, the next step is to analyze the BCGPs performance against both TWP and PGD hazards. For TWP hazard, the complete procedure in the form of flow chart is presented in Figure 2.4. The conversion table from PGA to PGV values according to the soil properties is applied as per ALA guidelines. By this way, estimated PGV values are used as hazard parameter for TWP analysis. The other input required for TWP analysis is the selection of seismic wave propagation and its related velocity values. In body waves, seismic design codes recommend considering shear waves rather p-waves due to their capacity of carrying higher energy and induction of large axial strain values. Similarly, for surfaces waves, recommendation is to select Rayleigh waves than the Love waves due to their ability of generating large axial strain values in BCGPs. The detail of whole procedure is presented in Chapter-4 of the thesis. Maximum friction and axial strain in the BCGPs are calculated to check the safety of the gas inflow.

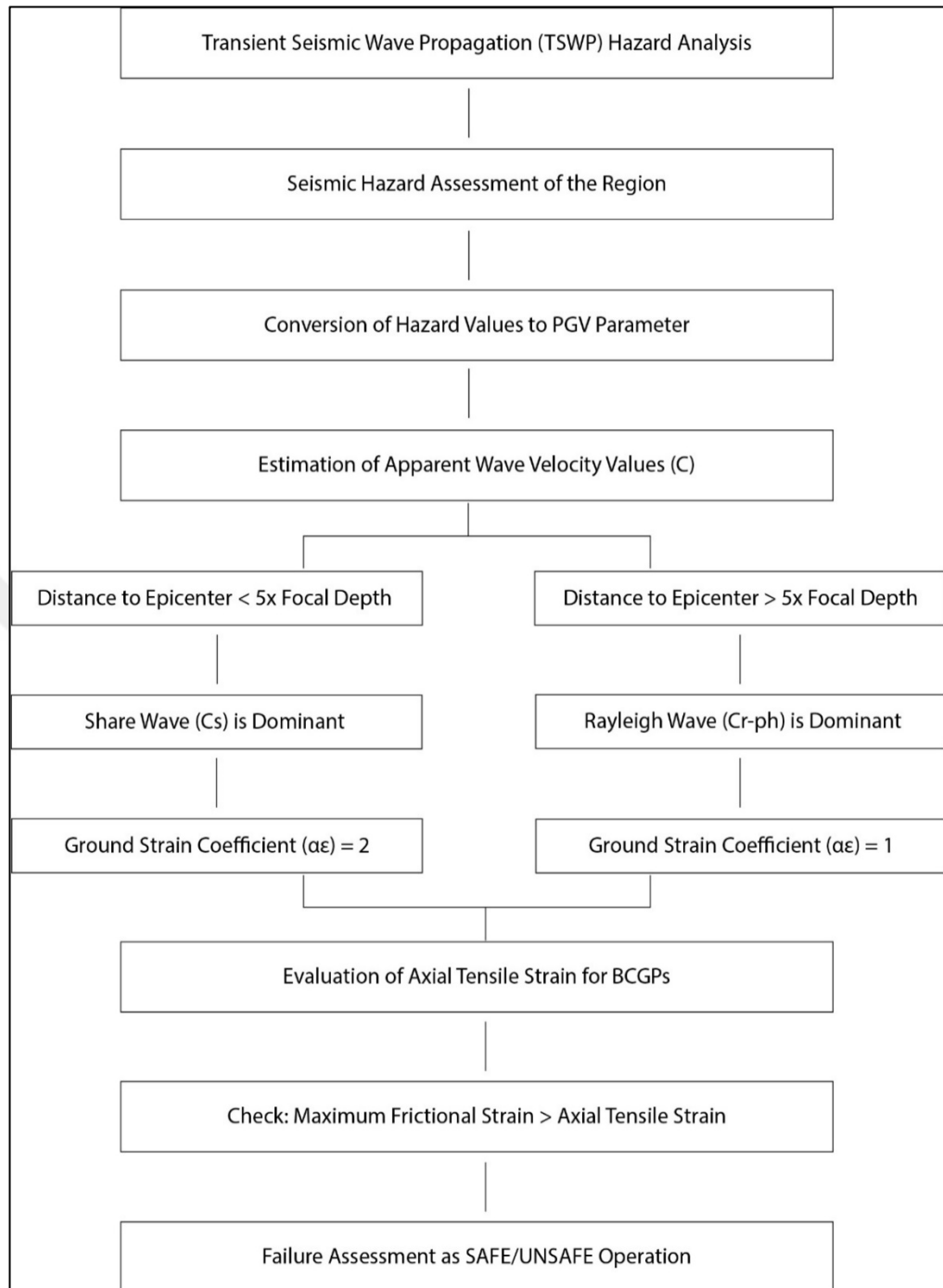


Figure 2.4: Flow chart for evaluation of axial strain in BCGPs due to TWP hazard.

TWP hazard as mentioned earlier create transient tensile strain and curvature in the ground due to the propagation of transient wave, however PGD hazard is characterized by the extent of its zone to which it is spatially distributed. Therefore, PGD hazard is more crucial to the BCGPs due to its higher potential risk. As per guidelines of ALA (2001) and ASCE (1984), PGD hazard can be assessed more precisely by FE analysis programs, however various researchers have performed

analytical based approaches for PGD analysis of BCGPs. In this thesis, an analytical based approach is presented for PGD analysis of the current study and reliability of the results are checked with the available approaches. In PGD hazard analysis, when the buried pipe is placed above the liquefied soil layer, then lateral spreading is the main subject to consider during design procedure. The lateral spreading can be either as longitudinal direction or transverse direction. Both conditions are considered in this thesis. The algorithm adopted for longitudinal PGD analysis is shown in Figure 2.5.

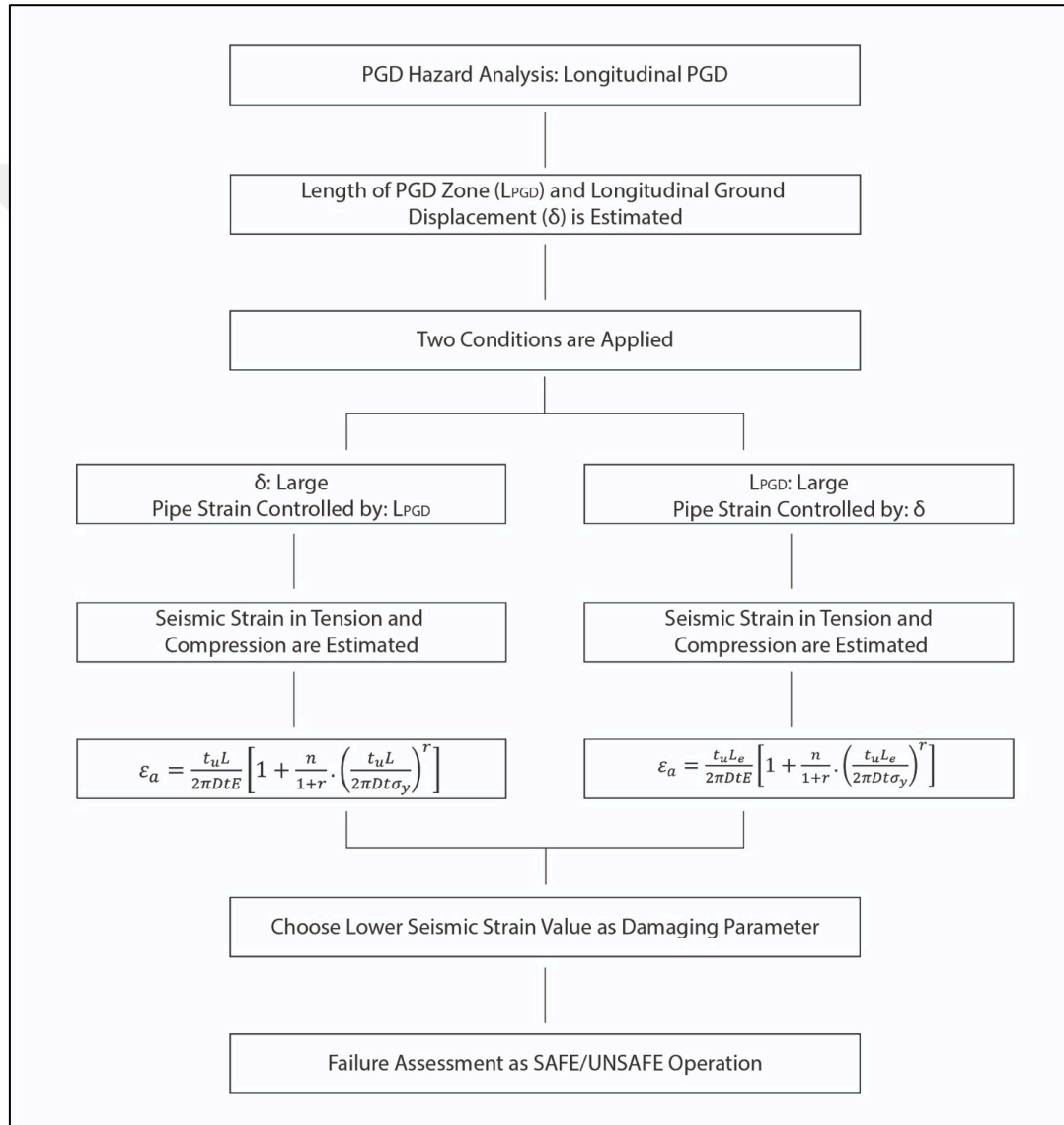


Figure 2.5: Flow chart for evaluation of axial strain in BCGPs due to longitudinal PGD.

The seismic effect of transverse PGD is slightly different than the longitudinal PGD. As longitudinal PGD concern to the length of PGD zone, transverse is related to

the width of PGD zone. The recommended length and width of PGD zones related to various seismic intensity levels are available in standard and guidelines. The procedure developed for transverse PGD analysis is presented in the form of flow chart in Figure 2.6.

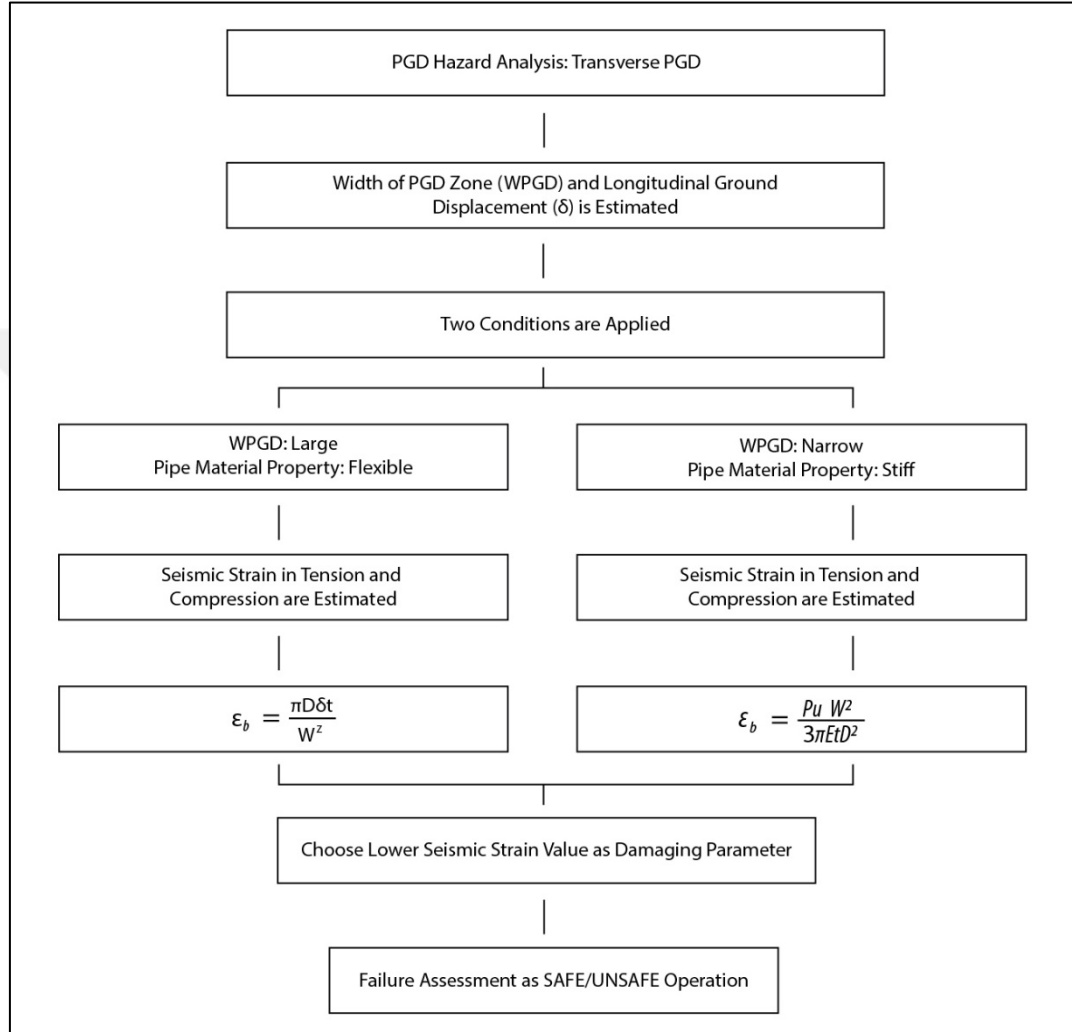


Figure 2.6: Flow chart for evaluation of axial strain in BCGPs due to transverse PGD.

The phenomena of buoyancy effect happens when the BCGPs are placed in the liquefied layer of the soil. The failure of BCGPs is observed in Tohoku earthquake Japan, where gas pipes were buried deeper and failed due to the seismic displacement caused by buoyancy. Due to buoyancy, BCGPs comes on the surface of ground and started to float with the soil. The PGD estimation procedure for liquefaction induced buoyancy is somewhat like the transverse PGD of lateral spreading. The algorithm

used for buoyancy effect is explained in the form of flow chart in Figure 2.7. The completed analysis details are explained in chapter-5 of this thesis.

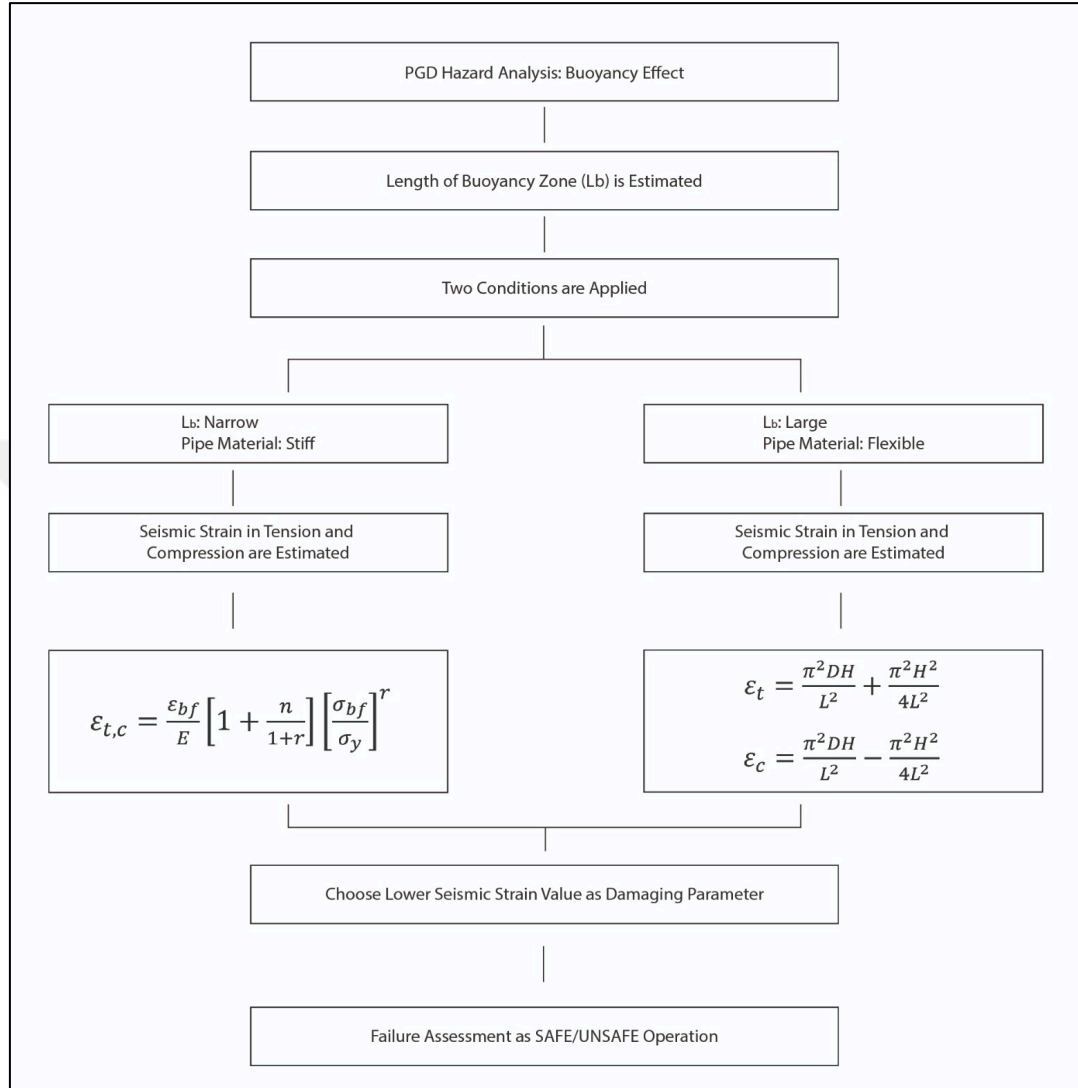


Figure 2.7: Algorithm for seismic analysis of BCGPs against buoyancy effect.

The PGD hazard due to fault crossing is considered as one the crucial factor to be considered for seismic design of BCGPs. As seen from past earthquakes, the major damages were observed due to the pipelines crossing the seismic faults. For instance, in 1971 San Fernando earthquake, the shaking was of small duration, however gas pipes crossing the seismic faults were severely damages and almost 1400 breaks were reported at various locations. There are different kind of seismic faults based on their direction of movement. In usual terms, 3 main kinds of faults including normal, reverse and strike-slip faults are considered by experts of this field. For geometrical

explanation of these faults, normal and reverse faults are of vertical plane movement whereas strike-slip is of horizontal direction. Normal and reverse fault create tensile and compression strains in BCGPs simultaneously. Whereas for strike slip fault crossing, the nature of induced strain is based on the angle of fault crossing and the position of buried pipes.

For the current case study, the subjected region is prone to the strike-slip fault, therefore the research on strike-slip fault effect on the nearby gas pipes are evaluated. For this study, two well-known approaches Newmark-Hall (1975) and Kennedy et al. (1977) methods are considered. Newmark-Hall (1975) method is one of the earliest developed analytical procedures that is still in use by many experts. Their approach provides satisfactory results for BCGPs crossing lateral strike-slip faults having angle of intersection between fault plane and buried pipe less than  $90^\circ$ . The only drawback of this method is the negligence of pipe bending stiffness and horizontal pipe-soil interaction. The whole procedure adopted by Newmark-Hall (1975) is presented in the form of flow chart in Figure 2.8 below.

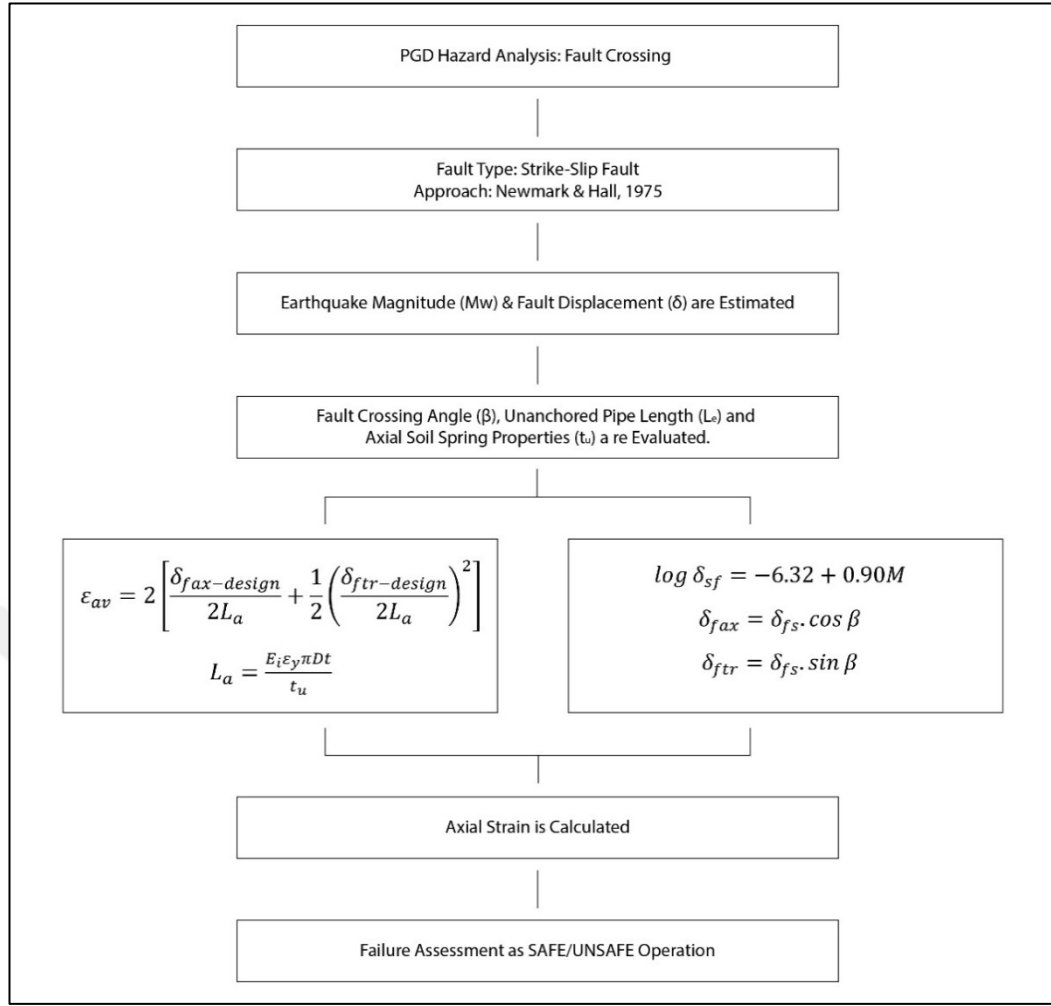


Figure 2.8: Flow chart for evaluation of axial strain in BCGPs due to fault crossing-I.

After the analytical findings of Newmark-Hall (1975), Kennedy et al. (1977) upgraded this approach by considering lateral soil-spring properties. Not only this, but authors also update their approach by adding factors such as soil curvature and bending strain values induced in the buried pipes. The methods yield tensile strain values due to the pipes crossing strike-slip fault. The procedure of Kennedy et al. (1977) method is given in Figure 2.9 below.

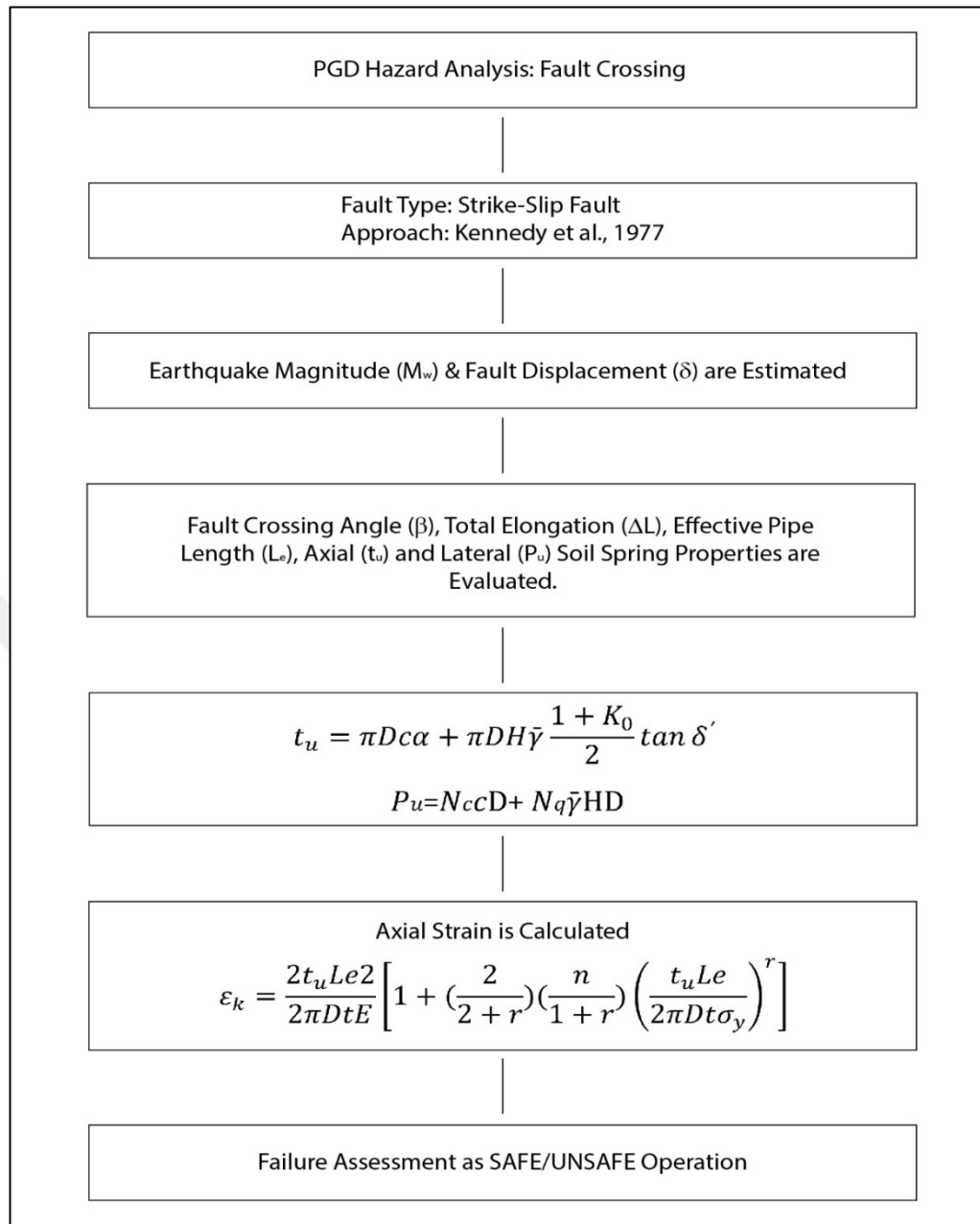


Figure 2.9: Flow chart for evaluation of axial strain in BCGPs due to fault crossing-II.

A unique three-stages geotechnical methodology is offered for PGD estimation of landslide (lateral spreading) and liquefaction, symbolizing (i) the susceptibility of the hazard, (ii) then triggering control, and (c) finally the PGD assessment levels. The 1999 Kocaeli earthquake data and the gas network of Sakarya – Istanbul regions are used as a test-bed example and the procedure is graphically presented in Figure 2.10.

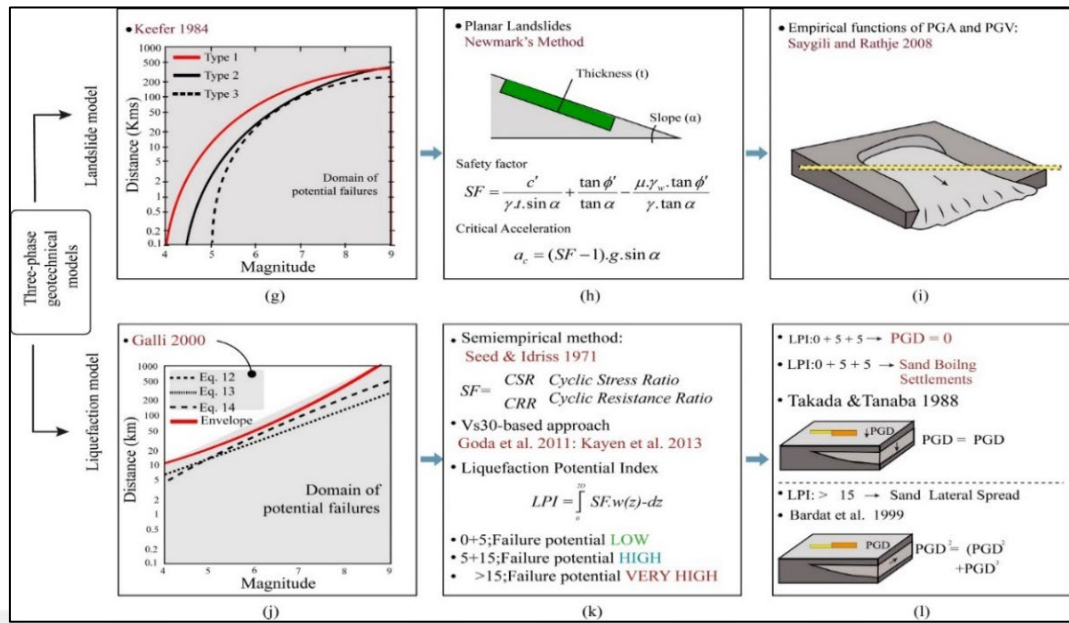


Figure 2.10: Three phase geotechnical models, (g) the susceptibility chances, (h) triggering control, (i) and PGD due to landslide, (j) the susceptibility chances, (k) triggering control, and (l) PGD due to liquefaction.

Urban earthquake risk assessment is completed in chapter-6 of the thesis. The hazard contribution of both PGD and TWP are considered to analyze the failure in terms of number of leaks and breaks. The analytical procedure available in ALA 2001 guidelines is used, where number of leaks incorporate 80% failure due to strong ground shaking (SGS) or TWP hazard and 20% for PGD and for breaks vice versa as shown in Figure 2.11 below. After estimation of number of leaks and breaks from the concerned seismic hazards, one can calculate the pipe replacement and maintenance costs as per local rate of labor costs, pipe material and addition of other factors.

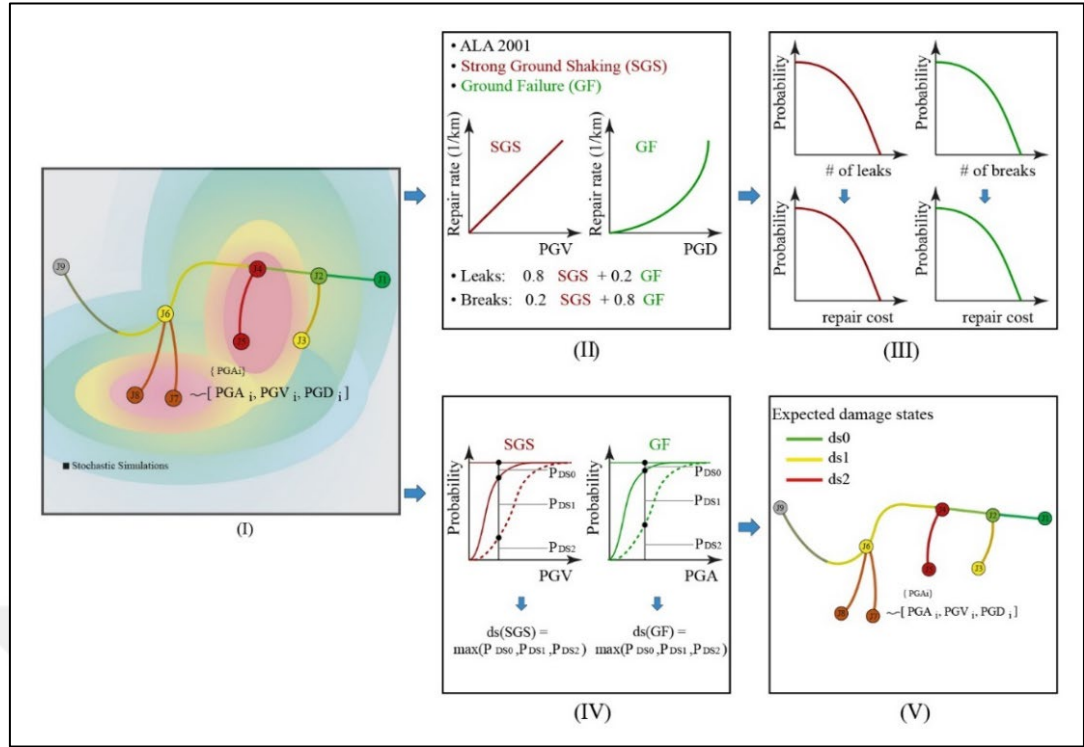


Figure 2.11: Seismic risk evaluation: (a) hazard parameters (b) vulnerability analysis, (c) loss figures, (d) fragility functions (methodology), and (e) hazard maps.

In chapter-7, seismic analysis is performed for API Grade B 5L steel pipe, a kind of continuous gas pipeline commonly used in Turkey. Three main parameters such as soil type, burial depth and pipe boundary end conditions are studied in detail. Total 6 strong ground motion records are used as input earthquake data that are further scaled from 0.1g to 1.5g to get behavior of selected pipe at various intensity levels. Results from numerical analysis are then compared with the nonlinear fiber element analysis to check the compatibility of the model and its response. The damage to the buried pipe in the form of maximum axial strain are evaluated. The results of current research give a useful insight for seismic performance evaluation of API Grade B 5L steel pipe to the researchers of this field.

Further, the procedure adopted and explained in the thesis is converted to a monitoring and early warning system as explained in Figure 2.12. It has four phases. First phase is the observation point. In this stage, strain accumulated in BGPs due to seismic loading, internal gas pressure and temperature variations are observed. Further, nature of fault movement (NFM) and strength of earthquake based on strong motion (SGM) records are examined. The second phase is the data acquisition and

processing unit that transfers the data to wireless communication system. Beidou satellite and 4G technology can be preferred to transfer the unreadable data to remote control receiving unit for display. The third phase is the analysis/display and management system. Display screens are used for exhibition of earthquake early warning system (EWS), BGPs strain induced, leakage and location required for inspection. The output data can be shared by text messages, email, or smart watches. The fourth and last phase is the alarm system. If any parameter exceeds the threshold limit value (in case of tensile strain limiting value is taken as 0.003), alarm and light color will demonstrate the emergency.

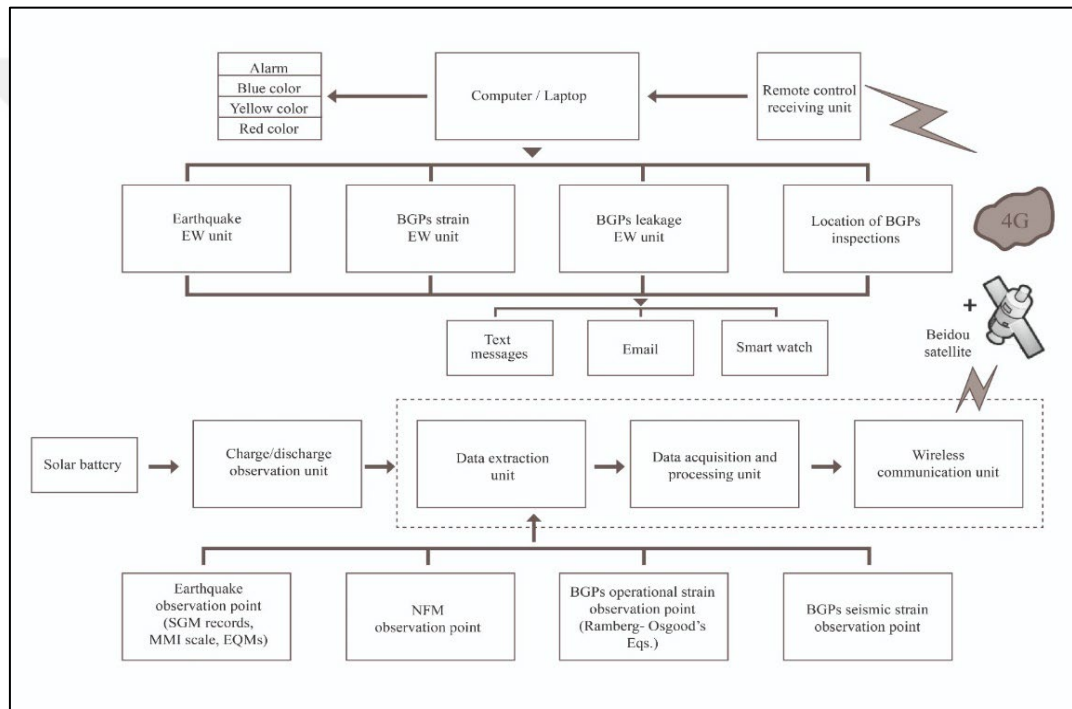


Figure 2.12: Development of EWS for urban earthquake risk assessment of buried gas pipelines.

### **3. SEISMIC HAZARD ANALYSIS AT URBAN SCALE**

Hazard is described as the essential physical characteristics that cause possible threats to citizens, land, or the atmosphere. In terms of earthquake engineering, hazard is termed as the frequency of shaking level in the form of liquefaction, landslides, or fault rupture.

#### **3.1. Seismic Hazard Analysis**

Seismic hazard analysis includes the quantitative assessment of subsurface shaking hazards at a specific area. Seismic hazards can be examined deterministically when an earthquake scenario is anticipated, or probabilistically, in which uncertainty levels in earthquake magnitude, site, and time of incidence are explicitly considered [Kramer, 1996]. The most important part of seismic hazard analysis is the evaluation of peak ground acceleration (PGA), peak ground velocity (PGV) and spectral accelerations ( $S_a$ ) based on the recorded ground motion data histories. This data is essential to develop design response spectrums for civil engineering infrastructures. In seismic hazard analysis, the following two main steps are taken into consideration:

Firstly, it is necessary to identify the whole earthquake sources along with the fault geometry of the site under consideration. This procedure is possible by examining geology, tectonic behavior, seismic histories, and seismic records at the specific locations. This information could be helpful for estimation of expected future earthquakes [Kramer, 1996].

Secondly, it is essential to determine the ground motions parameters. This can be achieved by using GMPEs that further dependent on two basic seismic hazard analysis methods. One is a single earthquake scenario phenomenon and is known as deterministic seismic hazard analysis (DSHA). In DSHA, a predefined earthquake magnitude with single source-site distance is accounted. The other one is probabilistic seismic hazard analysis (PSHA) in which various earthquake scenarios are considered. In PSHA, all uncertainties related to earthquake magnitude, size location and time of occurrence are considered [Lin et al., 2013].

### 3.1.1. Deterministic Seismic Hazard Analysis (DSHA)

This methodology considers earthquake scenario as assumed or the realistic one. DSHA uses identified seismic sources which are close to the site with historic seismic and geographical information to create isolated, single-estimated events or forms of ground motion at the location. In DSHA method, three main steps are involved. Firstly, all nearby earthquake sources are necessary to identify. Then, controlling earthquake at the specific source is determined. Final input is the use of attenuation relation to develop seismic hazard of a specific site. The uncertainties in the magnitude, location, size, and time of earthquake are not considered in DSHA. The four steps that are involved in evaluation of DSHA are present in Figure 3.1. As uncertainties are not considered, so DSHA approach gives the higher bound values for the ground motions. This method is adopted for seismic hazard analysis of some crucial structures as dams, bridges, and nuclear-powered plants etc.

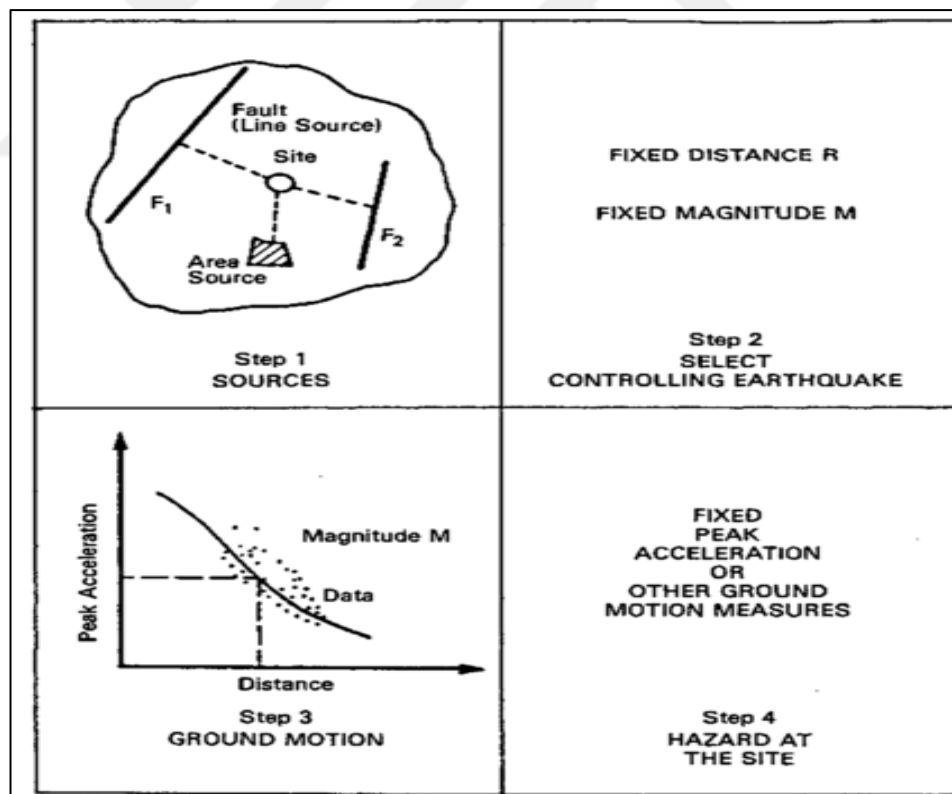


Figure 3.1: The primary stages of DSHA analysis.

### 3.1.2. Probabilistic Seismic Hazard Analysis (PSHA)

PSHA was firstly introduced by Cornell [Cornell, 1968]. In PSHA, all uncertainties in earthquake magnitude, site, and recurrence rate are considered. This analysis methodology provides us a framework for classifying, quantifying, and compounding these uncertainties in a balanced way. The main steps that are involved in PSHA includes: identifying and characterizing all seismic sources, earthquake event recurrence rate characterization, estimation of ground motions by using GMPEs, and evaluation of ground motion parameters for mean annual rate of exceedance. All these steps involved in PSHA are described in Figure 3.2.

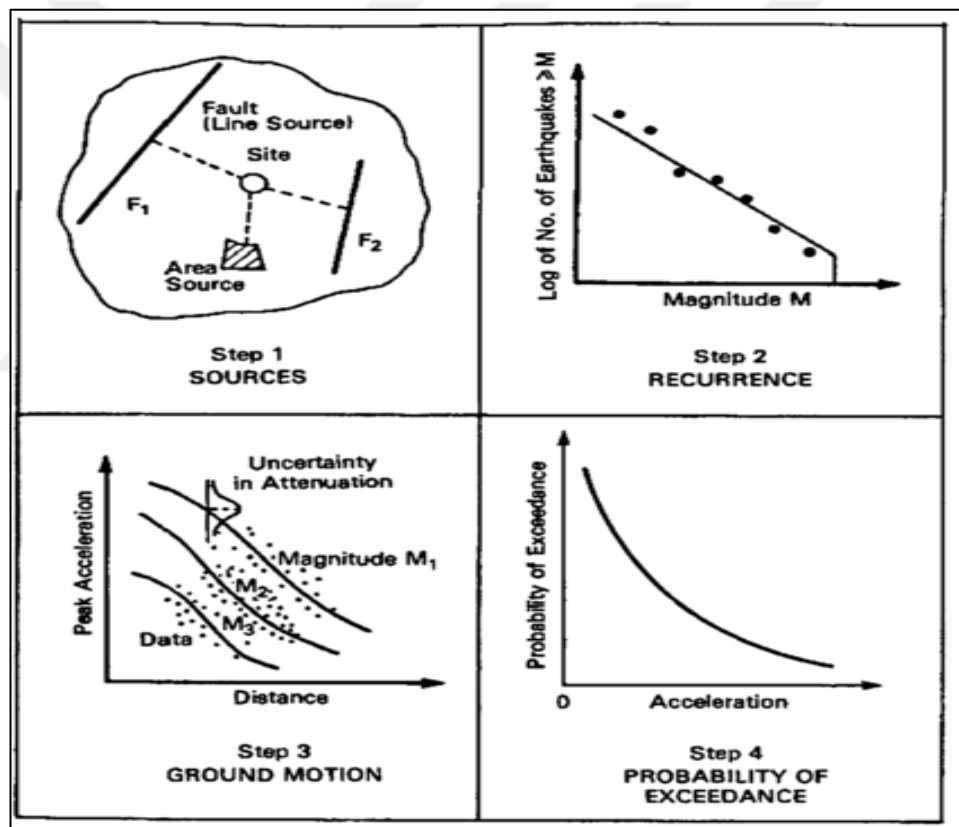


Figure 3.2: The primary stages of PSHA analysis.

### 3.2. Seismic Zoning Maps (SZM) of Turkey

Firstly, un-official SZP of Turkey was developed by Pamir in 1948. The map was divided into 3 regional parts as 1st degree (seismic alert zone), 2nd degree (Moderate zone), and 3rd degree as Safe zone, respectively.

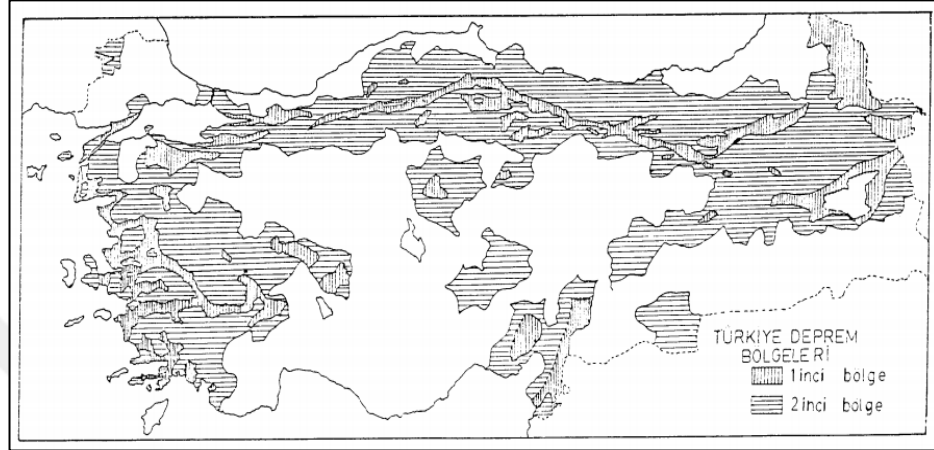


Figure 3.3: SZM of Turkey presented in 1948.

Firstly, official SZM of Turkey was developed in 1963 by Ministry of Public Works. The map was divided into four zones as: First degree (High seismic alert zone), Second degree (Moderate zone), Third degree (Low hazard zone) and Forth zone as safe zone, respectively.

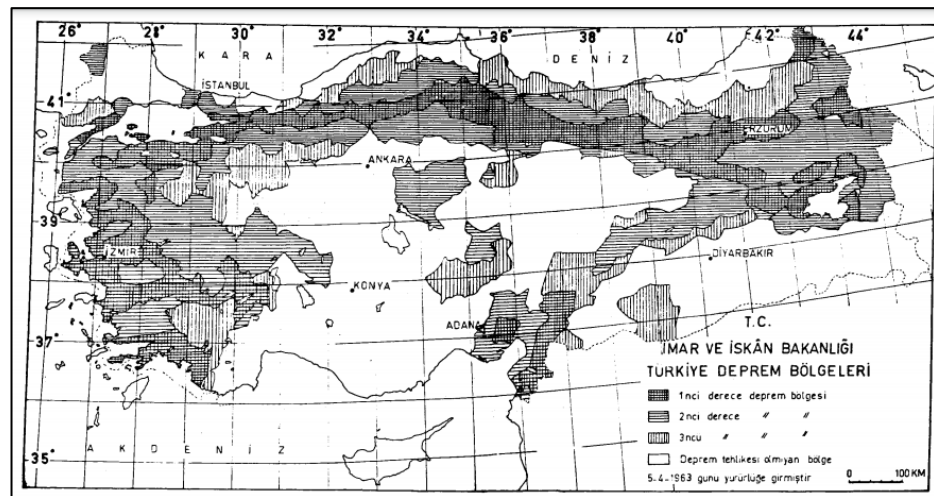


Figure 3.4: SZM of Turkey published in 1963.

In 1965, the SZM of Turkey was recommended by Institute of Seismology, Istanbul Technical University. The map was divided into four zones as before.  $I = I_0 + 3.58 - 3.33 \log_{10} R$  Macro-seismic attenuation relationships were used to develop this map.

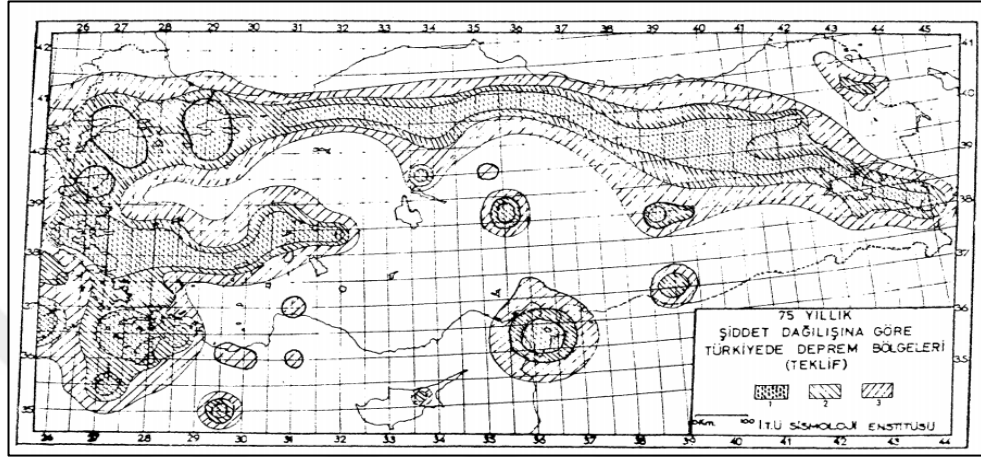


Figure 3.5: SZM of Turkey recommended in 1965.

In 1971, SZM with five-degree zones was presented to Ministry of works in Turkey by Ergin and Güçlü and is shown in Figure below,

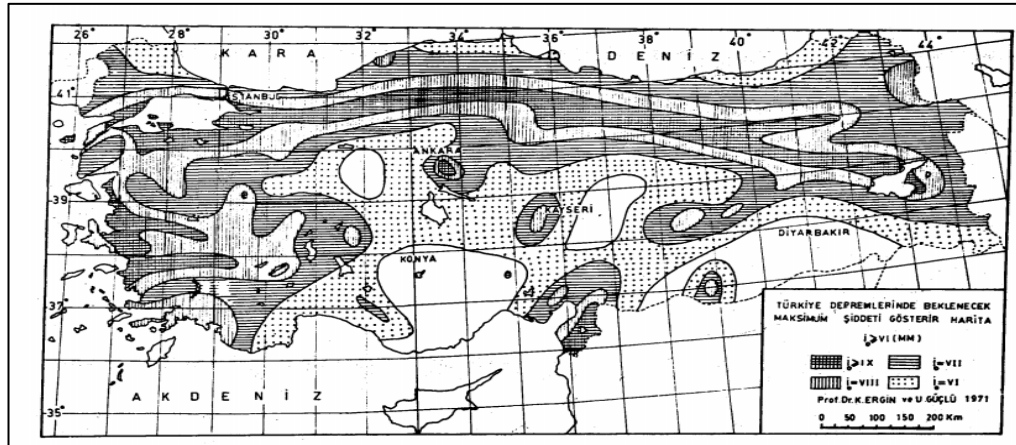


Figure 3.6: SZM of Turkey proposed in 1971.

In 1972, based on Modified Mercalli Scale (MMS) hazard classes, the Ministry of Works in Turkey published the below seismic zoning map of 5 grades. The first

zone was the MMS level higher or equal to IX, second zone for VIII, third zone for VII, four equals to VI and in the 5th, zone was for V or no hazards.

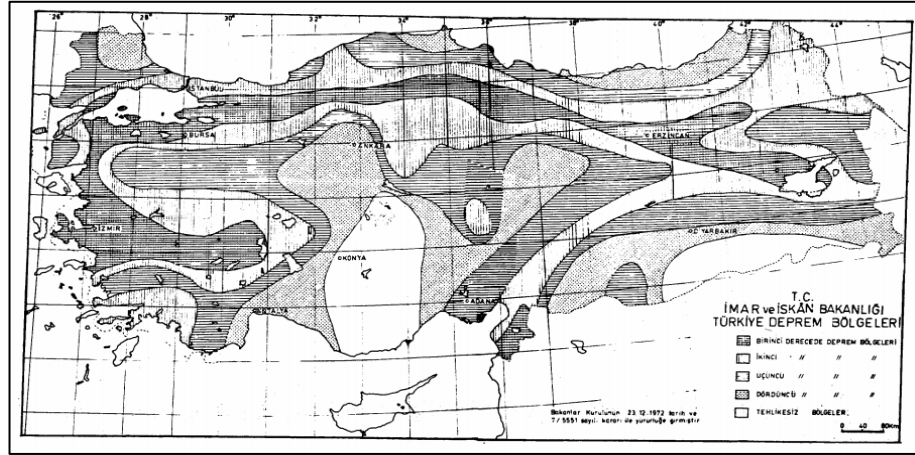


Figure 3.7: SZM of Turkey published in 1972.

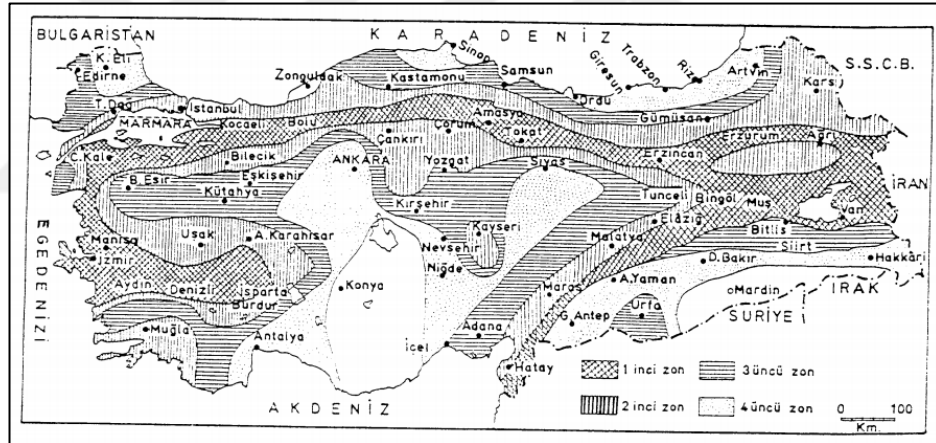


Figure 3.8: SZM of Turkey published in 1972 (District wise).

In 1996, the AFAD published a seismic zoning map on their website based on acceleration contours as presented in Figure 3.9.

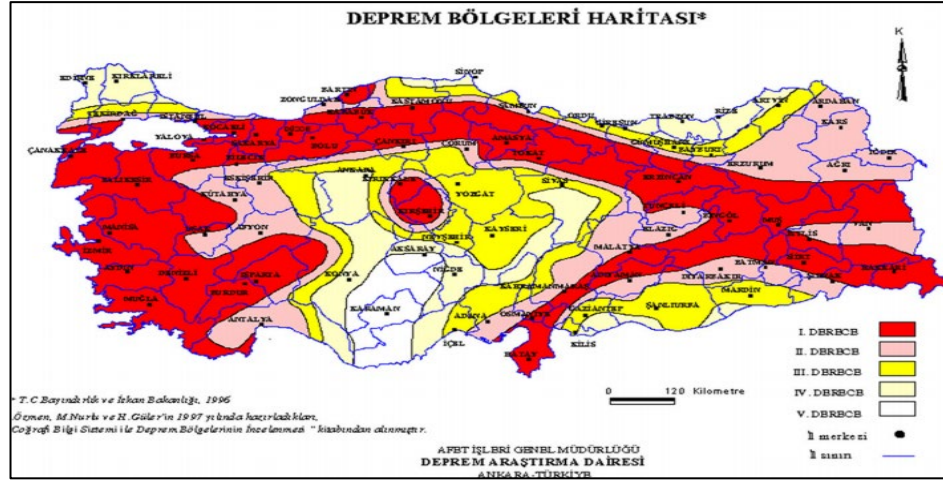


Figure 3.9: SZM of Turkey published in 1996.

Global Seismic Hazard Project (GSHAP) is carried out by [Erdik et al., 1999] and [Giardini, 1999] for Turkey and its surrounding regions. In their study as a source model, they used field (area) source. GPMs used in their project includes the equations developed by [Campell, 1997], [Boore et al., 1997] and [Sadigh et al., 1997]. Calculations were made for PGA ground motion at 100, 225, 475 and 1000-year return periods. Logic tree application was not used, and the hazard values are calculated considering stiff soil. GSHAP project containing Turkish hazard map is shown in Figure 3.10.

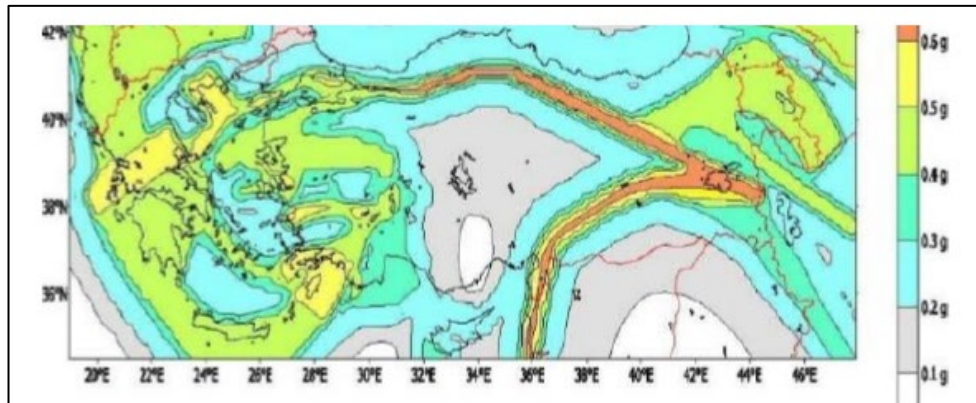


Figure 3.10: SZM of Turkey published in 1999 by GSHAP.

In the year of 2000, Turkey Emergency Flood and Earthquake Recovery Project (TEFER) was carried out and under this project, seismic zoning map for Turkey is

prepared as shown in Figure 3.11. For PGA evaluation, GMPEs of [Campell, 1997], [Boore et al., 1997] and [Sadigh et al., 1997] and for PSA, GMPEs of [Boore et al., 1997] and [Sadigh et al., 1997] were used. The logic tree application was made by giving equal weight to the GMPE's and the calculation were made on the ground rock.

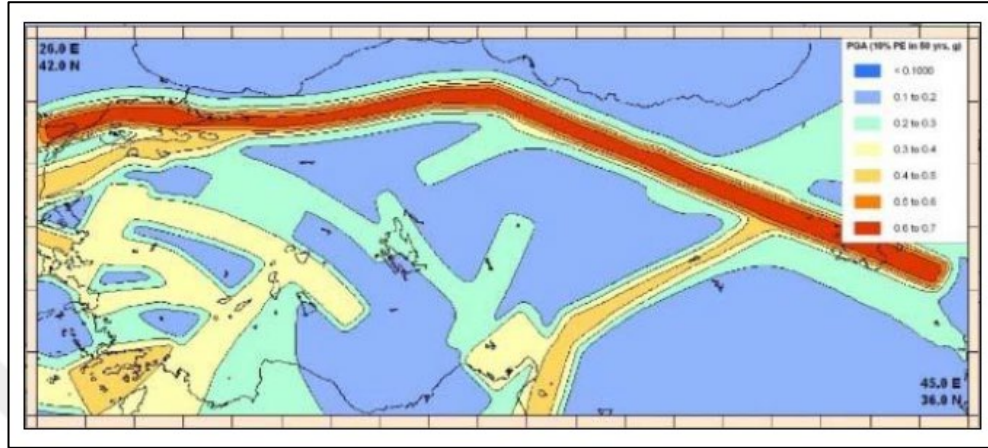


Figure 3.11: SZM of Turkey published in 2000 by TEFER.

Later, DLH [Demircioğlu et al., 2007] project has been carried out for Turkey, and source models such as area sources and line sources are used. For shallow crust tectonics, the GMPEs used for PGA are [Campell, 1997], [Boore et al., 1997] and [Sadigh et al., 1997] and for PSA, [Boore et al., 1997] and [Sadigh et al., 1997]. In this project, equal weights have been used for attenuation relations and the ground features considered are rock soil properties and  $V_{s30} = 760$  m/s.

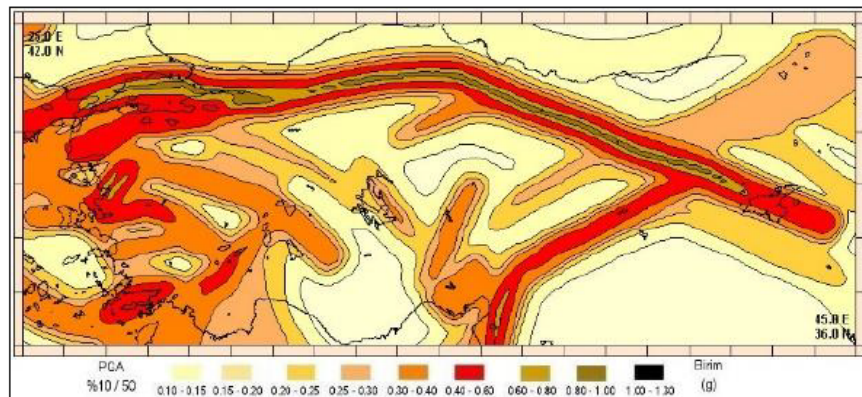


Figure 3.12: SZM of Turkey published in 2008 under DLH project.

Seismic Hazard Harmonization in Europe (SHARE) developed by [Woessner et al. 2015], is a study that starts from the ridge of the middle Atlantic and spread in a wide area up to the European continent. The objective of the study is to update the earthquake hazard model of the Euro-Mediterranean regions. In this project, with the leadership and close collaboration of seismologists, engineers, geologists, they aimed to establish new standards in probabilistic earthquake hazard analysis. Three different earthquake source models were used in the study. The first one is the area source model (AS), the second is the Kernel smoothed seismicity (SEIFA), the third is the fault (line source) and background (FSBG). The form of tectonic zones are stagnant regions, ocean crust, active shallow crust such as compression, elongation, strike-slip, mid-ocean ridge, subduction zones, deep seismicity zones and volcanoes.

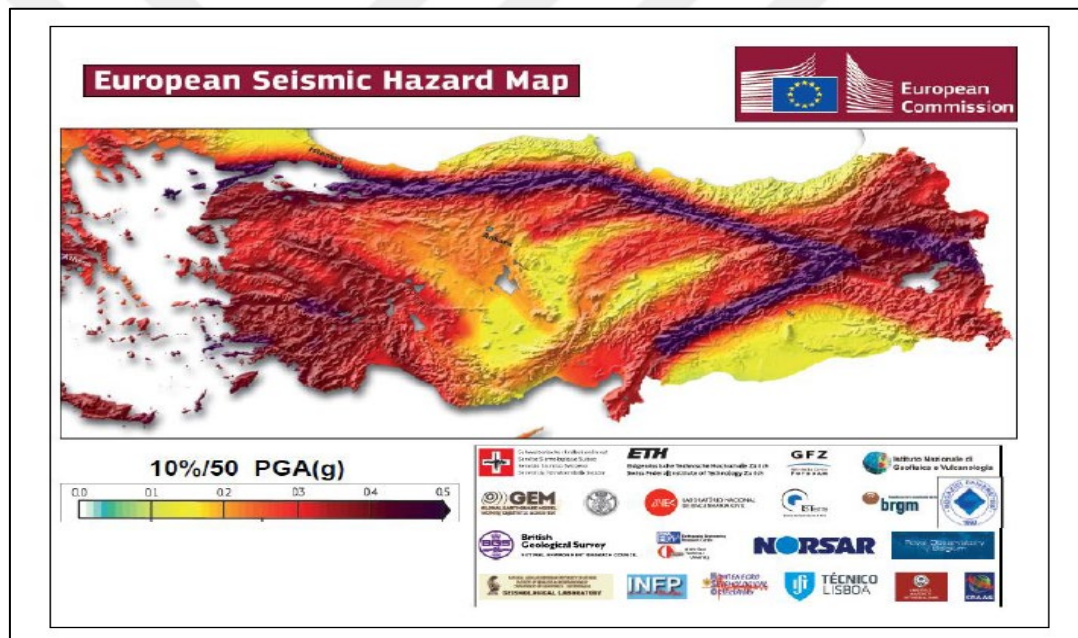


Figure 3.13: SZM of Turkey published in 2013 under SHARE project.

Another regional project is EMME ‘Earthquake Model of Middle East’, which aims at the evaluation of earthquake hazard, the concerned risk in terms of structural failures, mortalities, and economical losses and at the assessment of the effects of related moderation measures in the Middle East region in relation with the aims and apparatuses of GEM (Global Earthquake Model). For PGA and  $T = 0.10, 0.15, 0.20, 0.25, 0.30, 0.50, 0.75, 1.0$  and  $2.0$  seconds, the damping ratio for 5% PSA, hazard calculations were made in 72, 475, 975, 2575 and 4975 return periods.

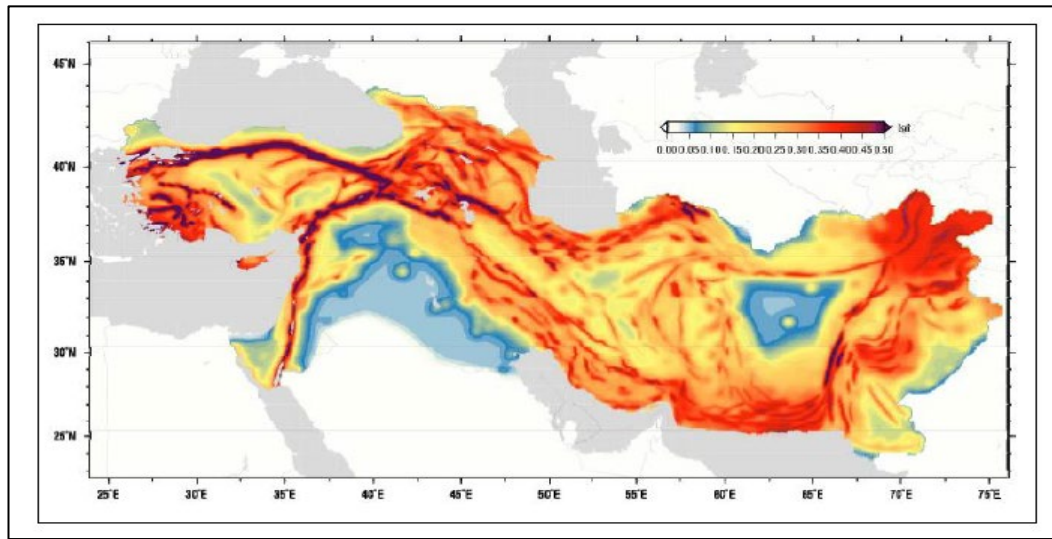


Figure 3.14: Hazard map showing the PGA value corresponding to the 475-year return period published in 2018 under EMME project.

Turkish Seismic Hazard Map (T-SHM) was carried out under the project UDAP-O-13-06. A little after the end of EMME, AFAD (public authority responsible of national earthquake hazard maps and building design standards in Turkey) issued the “Revision of National Earthquake Hazard Map of Turkey termed as T-SHM” project initiated by the requisite of developed definition of design ground motion by update in the local seismic design standard. The project is also supported by the Turkish Catastrophe Insurance Pool (TCIP) for more interesting in financial loss figures to establish well-constrained seismic insurance values. This two-year research project gathered the national earth scientists and earthquake experts from different Turkish schools and institutions who were experienced in different subjects of PSHA. Some of these experienced personnel contributed to SHARE and EMME research findings, that leads the creation of this local project an example cooperation to display the interaction between local and country-wise seismic hazard assessment.

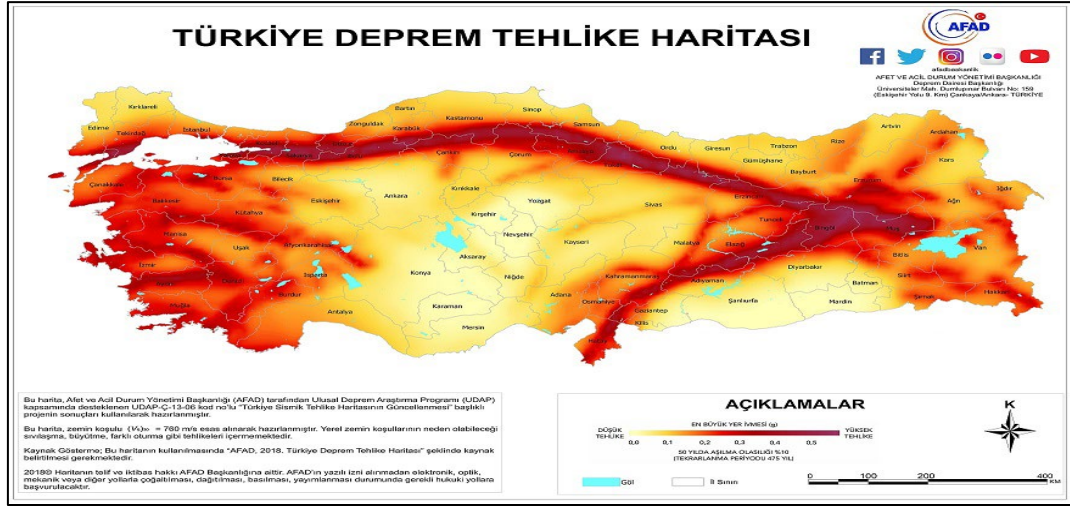


Figure 3.15: Turkish seismic hazard map corresponding to the 475-year return period published in 2018 under UDAP-O-13-06.

### 3.3. Seismic Hazards to BCGPs

Pipelines cross through diverse environments and face various soil conditions and seismic hazards in their routes of laying. Sometimes, it is not possible to change the path due to unavailability of land or environmental conditions. Seismic hazards are classified into two categories. One is seismic transient wave propagation (TWP) hazard and other one is permanent ground deformation (PGD) hazard. TWP hazard is due to the itinerant of seismic waves as shown in Figure 3.16a. PGD hazard comprises seismic faults, liquefaction and lateral spreading as shown in Figure 3.16b-d. In earlier seismic events, TWP influenced the large network of buried pipes, but no catastrophic damage has been reported. However, PGD hazard has contributed to small, isolated areas, but damage rate is noticed as more than TWP hazard.

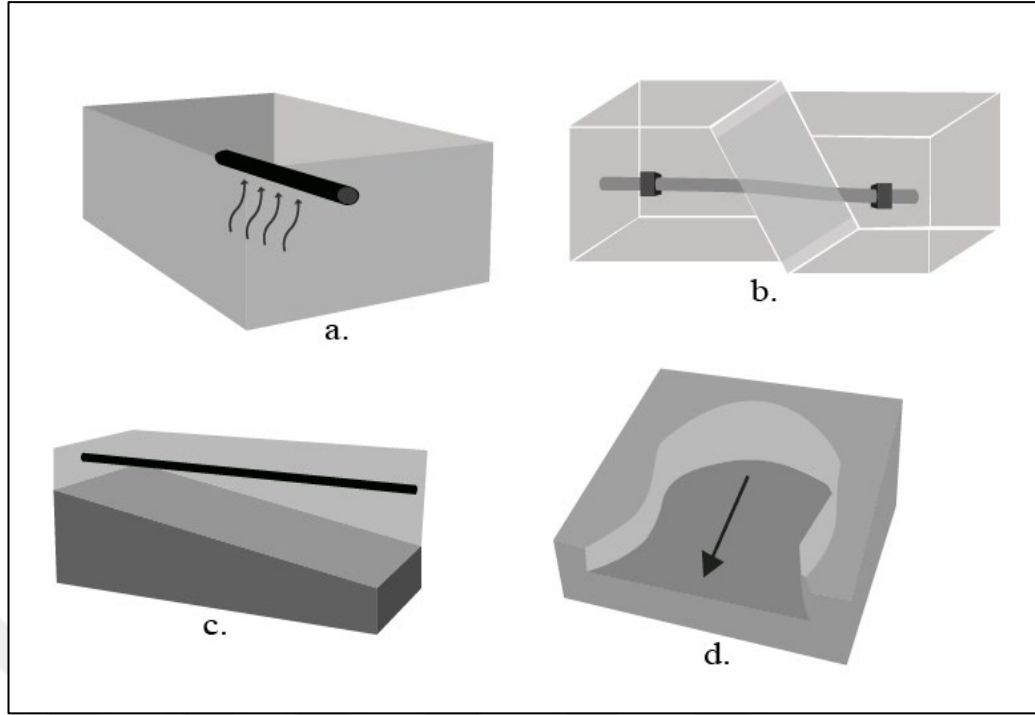


Figure 3.16: Seismic hazards for BCGPs a). wave propagation b). fault crossing c). liquefaction d). lateral spreading.

### 3.4. Seismic Hazard Analysis: Current Study

Within the scope of this thesis, seismic hazard analysis is carried in the East Marmara region to center Izmit / Kocaeli as shown in Figure 3.17. In this study, classical PSHA, and PSHA with the Monte Carlos method are performed. The source models used in this study are SHARE and EMME source models of the specified case study region. The PSHA is also performed on TBDY 2018 source model to check the consistency of EMME and SHARE model results with TBDY 2018. The earthquake catalogue used in EMME include  $M_{min}$  as 5.5,  $M_{max}$  6.4-7.6 and for SHARE;  $M_{min}$ : 6.4,  $M_{max}$ : 6.8-7.8.

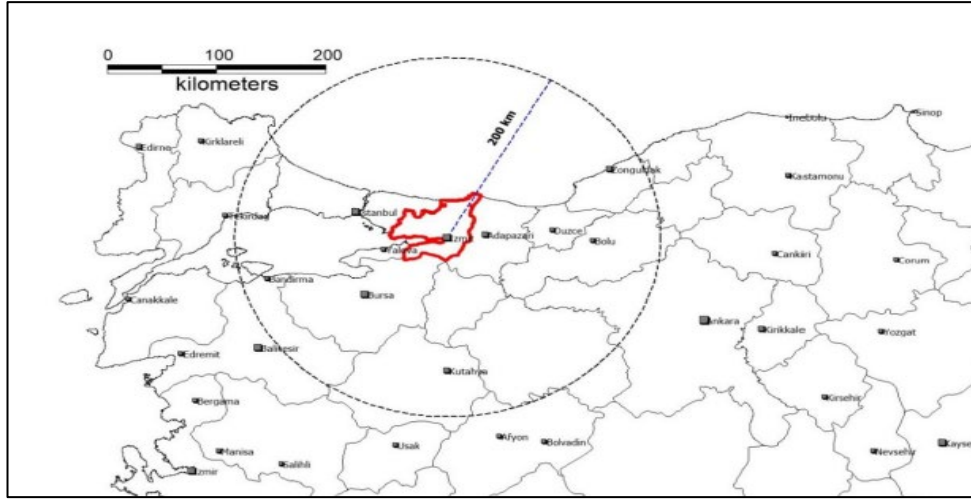


Figure 3.17: Selected area.

The GMPEs used for EMME project include Akkar et al. [Akkar et al., 2014], Akkar and Cagnan [Akkar and Cagnan, 2010], Chiou and Youngs [Chiou and Youngs, 2008], Zhao et al. [Zhao et al., 2006]. Similarly, the GMPEs used in SHARE project are Cauzzi and Faccioli [Cauzzi and Faccioli, 2008], Akkar and Bommer [Akkar and Bommer, 2010], Chiou and Youngs [Chiou and Youngs, 2008] and Zhao et al. [Zhao et al., 2006]. Same GMPEs are used for the classical probabilistic method and the probabilistic method with the Monte Carlo method.

Seismic hazard output in the form of hazard curves as PGA,  $S_a(0.2)$  and  $S_a(1.0)$  considering rock surface ( $V_{s30} = 760$  m/sec) at five selected locations (Gebze, İzmit, Sapanca, Sakarya and Düzce) are calculated. The evaluated seismic hazard values aid for the risk assessment of BCGPs of the region.

### 3.5. Tectonics and Seismicity of the Region

In this section, tectonics structure and seismicity history of the selected region is explained in detail.

### 3.5.1. Tectonic Structure of the Region

The Anatolian region is in the Alpine-Himalayan fold belt belonging to the last mountain formation from the Alpine Orogeny period. This tectonic region, which started with the Pyrenees Mountains forming the Spain-France line in the west, extends for 15000 km along a wide east-west direction to the Southeast Asia Islands. It can be understood that the Anatolian region is geologically in a place where the earth's crust has been active for a long time. A significant part of the cities in Anatolia carries a high risk in terms of natural disasters. Anatolia and its surroundings are in the most active part of the Alp-Himalayan Belt, which is the most seismic active region known in the world. If one look at the history of the cities in such a risky region, it is seen that these regions have been destroyed several times due to the earthquakes.

The Anatolian landmass is available on a tectonic region compressed by the confrontation of the African plate in the south and the Eurasian plates in the north. On this region, there are places such as Anatolia, Caspian, Black Sea and Arabian Peninsula, which are relatively moveable with respect to each other. This relative movement has brought along important fault formations. While the relative movements of the Anatolian Plate and the Arabian Plate with respect to each other form the Eastern Anatolian Fault Zone, the relative movement of the Anatolian and Black Sea Plates forms the North Anatolian Fault Zone (NAFZ). The plate tectonic model of Anatolia and its surroundings are shown in Figure 3.18.

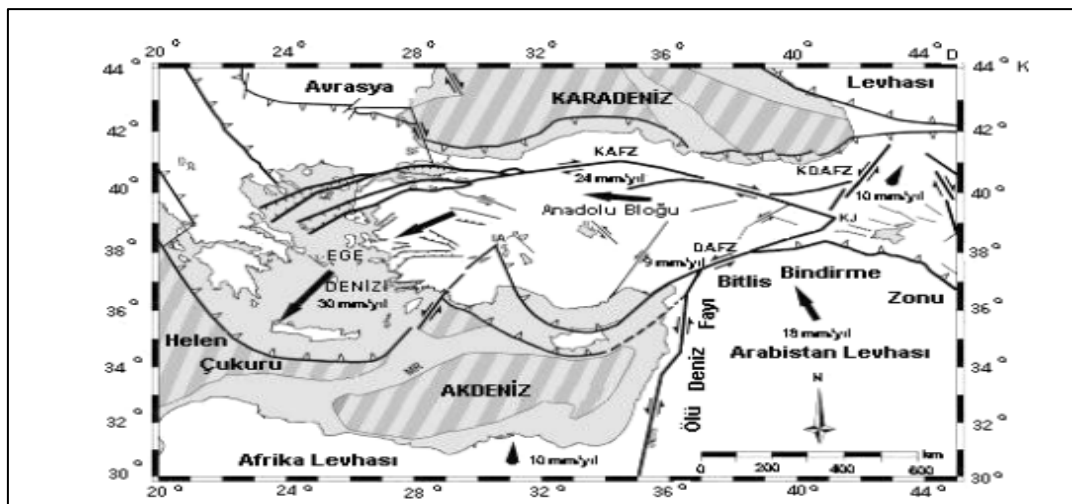


Figure 3.18: Plate tectonic model of Anatolia and its surroundings – I.

The right-lateral fault also known as NAFZ, forms the border region between the Eurasian and Anatolian plates, and moves at an average speed of 25 mm per year, causing serious earthquakes since the last century. This fault extends approximately 1500 km along the southern coast of the Black Sea. The fault has seriously affected the morphological and tectonic processes of the Marmara Sea and its surroundings in this geological period as shown in Figure 3.19. At the same time, 17 August 1999 Izmit / Kocaeli ( $M_s = 7.8$ ) and then 12 November 1999 Düzce ( $M_s = 7.5$ ) earthquakes are the major and last earthquakes that occurred in the Eastern Marmara Region because of the northern branch of this fault.

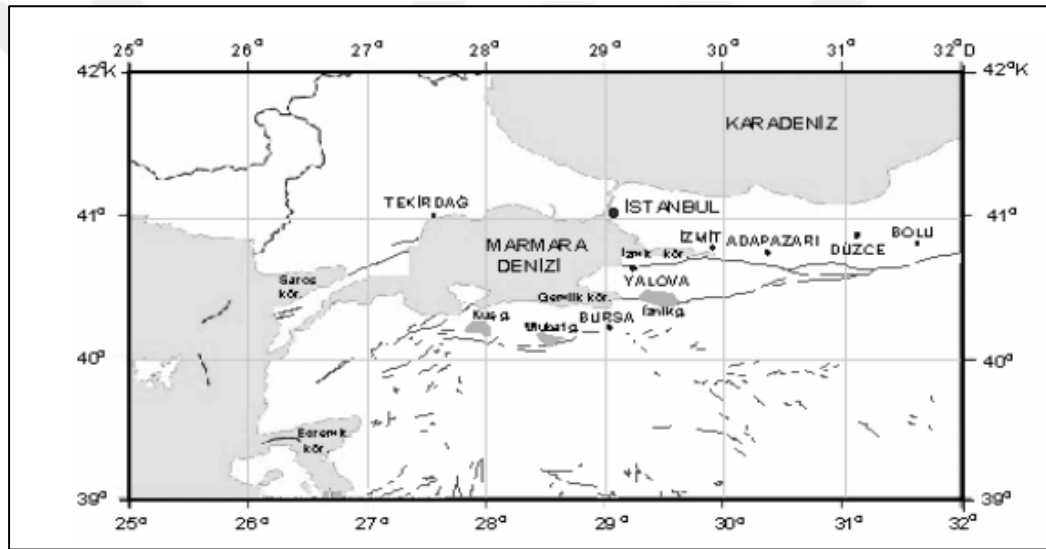


Figure 3.19: Plate tectonic model of Anatolia and its surroundings-II.

The NAFZ is divided into three main branches from the west of Bolu province. The first branch is the southern branch. This branch extends between Gemlik Bay and Almacık Mountain. The second branch is the middle branch. This extends from Adapazarı in the direction of Northeast-Southwest to the direction of Geyve. The third and last branch is the north branch. This branch starts from Bolu-Mudurnu and moves to Izmit Gulf, Sapanca and Marmara Sea. As per geological reconnaissance, the most active branch is the northern branch. The earthquake characteristics, behavior of the fault and fault geometry of NAFZ in the Marmara Sea are different from the features of the fault that can be observed on land [Kalafat 2000, 2003; Kalafat et al., 2001].

### 3.5.2. Seismicity of the Region

Seismological and seismotectonic features of NAFZ shows that it is one of the world's well-known strike-slip fault and one of the most important tectonic structures of the region. The NAFZ has been scientifically defined for the first time after the 1939 Erzincan earthquake and has been described as a right-sided strike-slip fault (Ketin, 1948; 1966; 1968). After the 1939 Erzincan earthquake, a series of large earthquakes that migrated westward occurred over the main segments of the NAFZ for certain periods. Table 3.1 shows the list of major earthquakes in the NAFZ segment after the 1939 Erzincan earthquake.

Table 3.1: Major earthquakes in the NAFZ after the 1939 Erzincan Earthquake (1939-2010).

| No. | Date |    |      | Time |      |     | Coordinates |       | Depth | Mag. | Location                                            |
|-----|------|----|------|------|------|-----|-------------|-------|-------|------|-----------------------------------------------------|
|     | D    | M  | Y    | Hr.  | Min. | Sn. | Lat.        | Long. | kms   | Ms   |                                                     |
| 1   | 12   | 12 | 1939 | 23   | 57   | 21  | 39.8        | 39.51 | 20    | 7.9  | Erzincan                                            |
| 2   | 20   | 12 | 1942 | 14   | 3    | 41  | 40.87       | 36.47 | 10    | 7.0  | Erba. Niksr.<br>Tokat.                              |
| 3   | 26   | 11 | 1943 | 22   | 20   | 41  | 41.05       | 33.72 | 10    | 7.2  | Ladik. Tosya.<br>Samsun                             |
| 4   | 1    | 2  | 1944 | 3    | 22   | 41  | 41.41       | 32.69 | 10    | 7.2  | Gerede. Çerkeş.<br>Bolu                             |
| 5   | 26   | 5  | 1957 | 6    | 33   | 41  | 40.67       | 31.00 | 10    | 7.1  | Abant. Bolu                                         |
| 6   | 22   | 7  | 1967 | 16   | 56   | 41  | 40.67       | 30.69 | 33    | 6.8  | Adapazari.<br>Mudurnu Vadisi                        |
| 7   | 17   | 8  | 1999 | 0    | 1    | 41  | 40.76       | 29.95 | 17    | 7.7  | Kocaeli. Golcuk.<br>Yalova. Sakarya.<br>Marmara Sea |
| 8   | 12   | 11 | 1999 | 16   | 57   | 41  | 40.81       | 31.19 | 10    | 7.4  | Duzce. Kaynaşlı                                     |

The seismic pattern of Turkey earthquakes is presented in Figure 3.20.

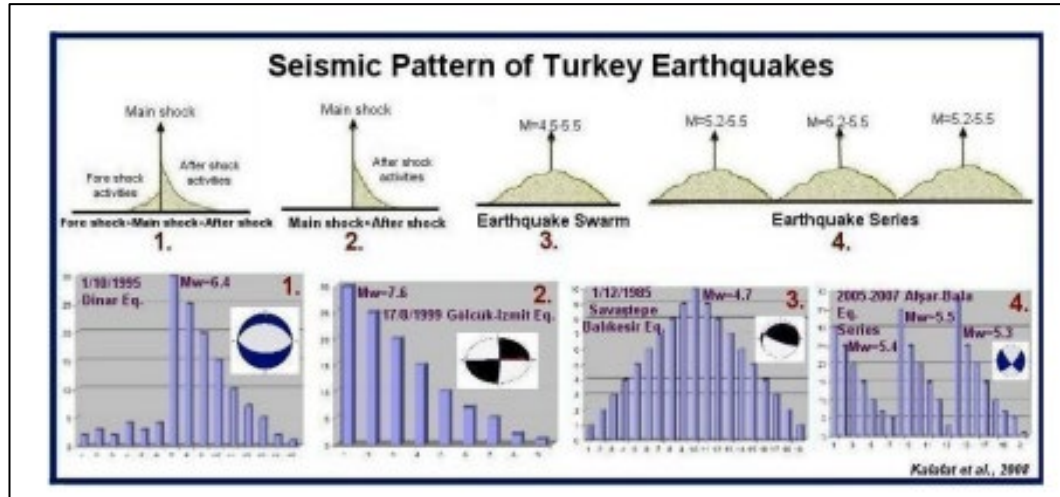


Figure 3.20: Examples illustrating the layout of the earthquake occurred in Turkey.

### 3.5.3. Historical Earthquakes

Historical earthquake records are the records available before the invention of instruments for earthquakes data recordings. These records are written as 3000 years in China; It only dates to around 2000 years in the Middle East and Japan, while in the USA around a few centuries [Kramer, 1996]. With the historical record of ground motion, it can be used to confirm earthquakes that occurred in the past and determine the intensity distribution. Based on the sufficient data availability, the maximum intensity can be estimated, and epicenter can be determined [Kramer, 1996].

Since the time of occurrence of historical earthquakes is recorded, it is used to evaluate the seismicity and recurrence rate of the earthquake in a certain region. The data of AFAD and KOERI (Kandilli Observatory Earthquake Research Institute) is compared for the historical earthquakes that occurred in the study area. As a result of the comparison, it is observed that the intensities of the earthquakes matching each other (except the 22.05.1766 earthquake; while it was said to be VII intensity in AFAD, the same earthquake was said to be IX in KOERI). Figure 3.21 (AHEAD; European Archives of Historical Earthquake Data) shows the seismicity distribution of the historical earthquakes that occurred in the study area determined between 1000 and 1899. This is also the part of historical earthquake study that was used in the SHARE project.

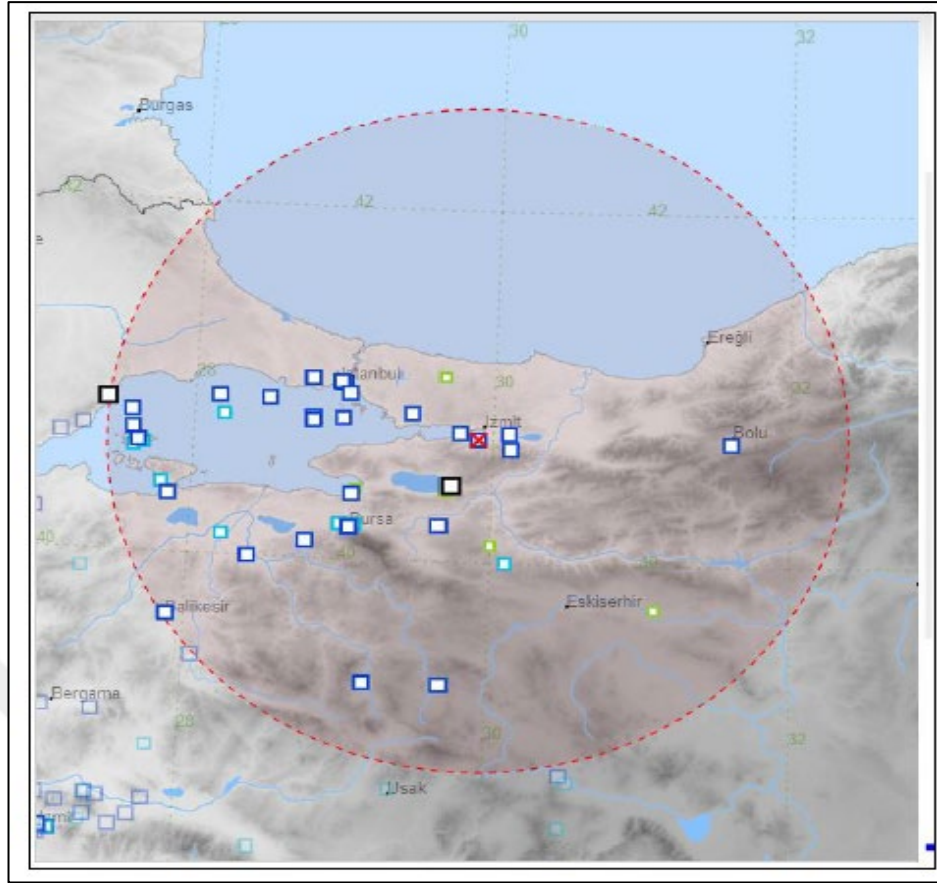


Figure 3.21: Seismicity distribution of the historical earthquakes between 1000-1899 in the study area.

The magnitude distribution of historical earthquakes that occurred between years of 1000-1899 in the study area are presented in Figure 3.22.

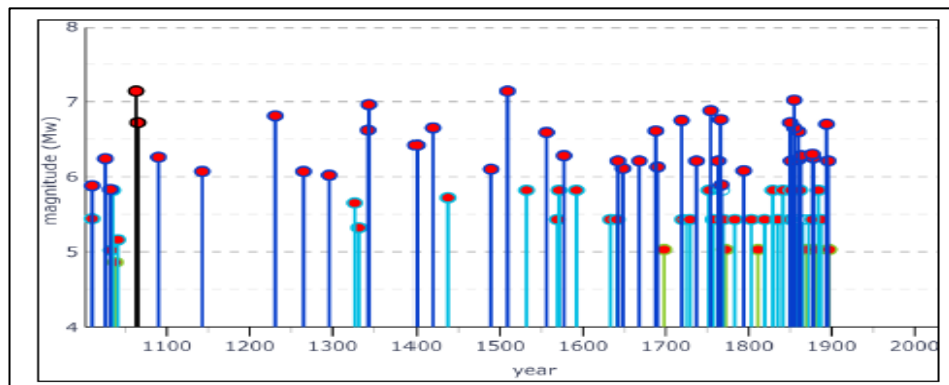


Figure 3.22: The magnitude distribution of historical earthquakes that occurred between 1000-1899 in the study area.

### **3.5.4. Instrumental Earthquakes**

Approximately 10 earthquakes with surface magnitude ( $M_S > 7$ ) occur each year for the last 90 or 80 years in different parts of the world [Kramer, 1996]. Instrumental recordings of major earthquakes data began in 1900. However, many earthquakes before 1960 have a quality problem or its data of recordings are incomplete. When instrumental earthquakes are arranged on a line, their hypocenter or epicenter indicates the source of the earthquake. Analysis of aftershocks also helps in determining the sources of earthquakes [Kramer, 1996].

## **3.6. Earthquake Hazard Analysis Inputs**

The inputs or in other words procedure for seismic hazard analysis includes the following below mentioned steps to follow.

### **3.6.1. Earthquake Catalog**

Earthquake catalogs are the important component of PSHA that can be prepared from national and international sources considering both historical and instrumental records. However, as the magnitudes are given in different scales in earthquake catalogs, they may not be suitable for direct use. For this reason, it is recommended to convert these magnitudes into a single scale before to use it [Yüçemen, 2011]. In catalogs, the moment magnitude ( $M_w$ ) is considered as suitable scale for converting various magnitude scales such as duration magnitude ( $M_d$ ), body wave magnitude ( $M_b$ ), surface wave magnitude ( $M_s$ ), local magnitude ( $M_L$ ) into this scale. For this purpose, different empirical relationships have been developed to ensure precise conversion. In the following sections, empirical formulas used in the catalogs created for EMME and SHARE projects are given.

#### **3.6.1.1. EMME Homogeneous Earthquake Catalog**

To create a good catalog, all data recording information should be collected from national and international centers. For instrumental recordings, three centers

contributed to this domain namely International Seismological Center (ISC), National Earthquake Information Center (NEIC) and EHB Bulletin. For the historical part of the catalog, the National Oceanic and Atmospheric Administration (NOAA) records are considered [Erdik et al., 2012]. There are 28244 data available with depth, magnitude, latitude, longitude, and time data of earthquakes in the EMME catalog. The important ones of these data are given in table 3.2 [Zare et al., 2014].

Table 3.2: List of collected data regarding the catalog used in the EMME project.

| Original source of data | The number of records | Time period    | Included magnitude values |
|-------------------------|-----------------------|----------------|---------------------------|
| Armenia's list of data  | 3,280                 | 995–2006       | <i>Ms, mb</i>             |
| Ambrasys                | 342                   | 400 B.C.–1999  | <i>Ms, mb</i>             |
| BHRC                    | 431                   | 1503–2005      | <i>Ms, mb</i>             |
| EDIG                    | 1,400                 | 1250 B.C.–2003 | <i>Ms, mb</i>             |
| EHB                     | 2,686                 | 1960–2006      | <i>Ms, mb</i>             |
| IIIES                   | 1,013                 | 2000–2006      | <i>Ms, mb, ML</i>         |
| IRSC                    | 37                    | 2006           | <i>MN</i>                 |
| ISC                     | 13,758                | 1901–2006      | <i>Ms, mb, ML</i>         |
| ISK                     | 1,521                 | 1901–2006      | <i>mb</i>                 |
| KOERI                   | 1,817                 | 1901–2006      | <i>mb</i>                 |
| Jordan                  | 52                    | 1990–2006      | <i>Ms, mb</i>             |
| Mak                     | 5,459                 | 1901–2006      | <i>mb</i>                 |
| NC                      | 265                   | 50–1969        | <i>Ms, mb</i>             |
| NCUSSR                  | 189                   | 1900–1976      | <i>mb</i>                 |
| NEIC                    | 95                    | 1963–2002      | <i>Ms, mb</i>             |
| NOAA                    | 61                    | 31 B.C.–1970   | <i>mb</i>                 |
| PDE                     | 77                    | 1973–2006      | <i>mb, Ms</i>             |
| PLN                     | 987                   | 1976–2006      | <i>Ms, mb</i>             |
| NOW                     | 54                    | 1924–1964      | <i>Ms, mb</i>             |
| PAPA003                 | 336                   | 1010–1899      | <i>mb</i>                 |
| SHE                     | 231                   | 1900–1999      | <i>mb</i>                 |
| SMCG                    | 535                   | 1991–2006      | <i>Ms, mb</i>             |
| SOYAL981                | 239                   | 1003–1998      | <i>mb</i>                 |
| Sul                     | 269                   | 1905–1999      | <i>mb</i>                 |
| TARMAN                  | 221                   | 1970–2002      | <i>Ms, mb</i>             |
| USGS                    | 199                   | 1903–2006      | <i>Ms, mb</i>             |

Ambraseys and Jackson [Ambraseys and Jackson, 1998] investigated the earthquake data that recorded in the Eastern Mediterranean from 464 BC. to 1995. From 150 earthquake records, they considered 62 as well defined *Ms* values and added to the considerable records. The recorded data showed that contribution of earthquakes

was 55% due to strike-slip fault, 30% normal and 15% reverse faults [Zare et al., 2014]. The conversion formulas used for the homogeneous catalog were prepared by [Scordilis., 2006]. The formulas created for the study considering the suitability of the region are shown in Table 3.3.

Table 3.3: The formulas used to convert the magnitudes  $M_b$ ,  $M_s$ ,  $M_L$  to  $M_w$  to obtain a homogeneous catalog.

| Scale      | Conversion Formula                                      | Limit Values                  | Explanation              |
|------------|---------------------------------------------------------|-------------------------------|--------------------------|
| $M_{bod.}$ | $M_w = 0.870 M_{bod.} + 0.830$                          | $3.5 \leq M_{bod.} \leq 6.0$  | Developed for this study |
| $M_w$      | $M_w = 0.850 (\pm 0.040) M_{bod.} + 1.030 (\pm 0.230)$  | $3.5 \leq M_{bod.} \leq 6.2$  | Scordili work            |
| $M_{sur.}$ | $M_w = 0.660 M_{surf.} + 2.110$                         | $2.8 \leq M_{surf.} \leq 6.1$ | Developed for this study |
|            | $M_w = 0.930 M_{surf.} + 0.450$                         | $6.2 \leq M_{surf.} \leq 8.2$ |                          |
|            | $M_w = 0.670 (\pm 0.005) M_{surf.} + 2.070 (\pm 0.030)$ | $3.0 \leq M_{surf.} \leq 6.1$ | Scordili work            |
|            | $M_w = 0.990 (\pm 0.020) M_{surf.} + 0.080 (\pm 0.130)$ | $6.2 \leq M_{surf.} \leq 8.2$ |                          |
| $M_{Loc.}$ | $M_w = 1.01 M_{Loc.} - 0.05$                            | $4.0 \leq M_{Loc.} \leq 8.3$  | Developed for this study |

After removing all earthquakes with magnitudes lower than 0 and of greater error depths, a total number of 28244 earthquake records were considered for EMME project catalog and the distribution of the selected records are shown in Figure 3.23.

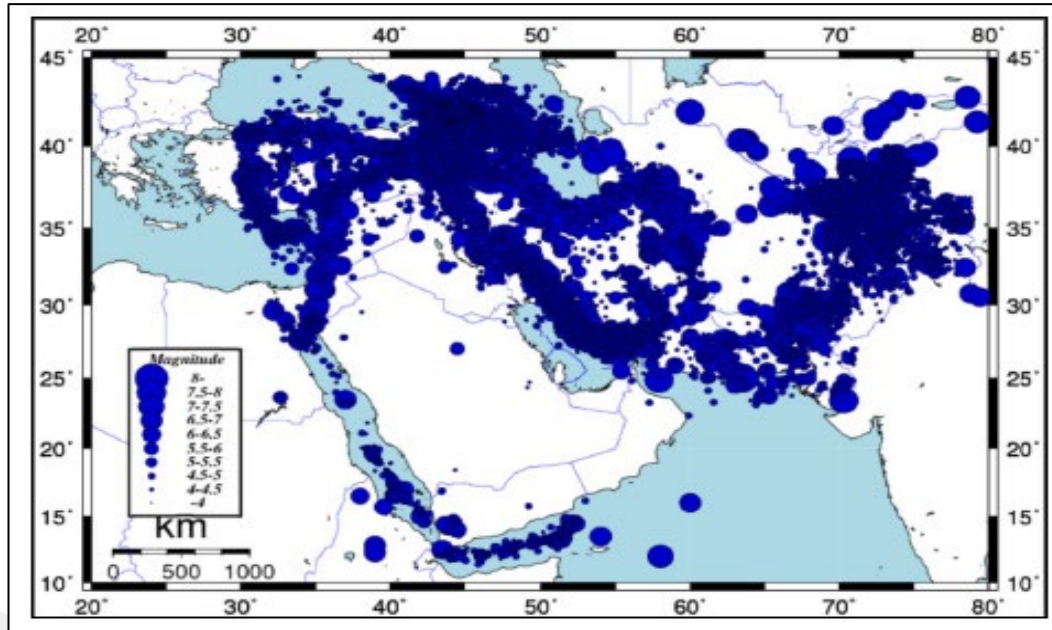


Figure 3.23: Homogeneous catalog containing 28244 earthquakes in EMME project.

### 3.6.1.2. SHARE Homogenous Project Catalogue

In SHARE project, historical earthquake records are taken from the SHARE European Earthquake Catalog (SHEEC) that are recorded between the years of 1000-1899 as shown in Figure 3.24. For preparation of catalog, the assistance is also gained from Archive of Historical Earthquake Data (AHEAD), that is considered as one of the precise data records of all the time for European region. The study includes 4,722 earthquake records with 306 records outside from regional catalogs [Stucchi et al., 2013].

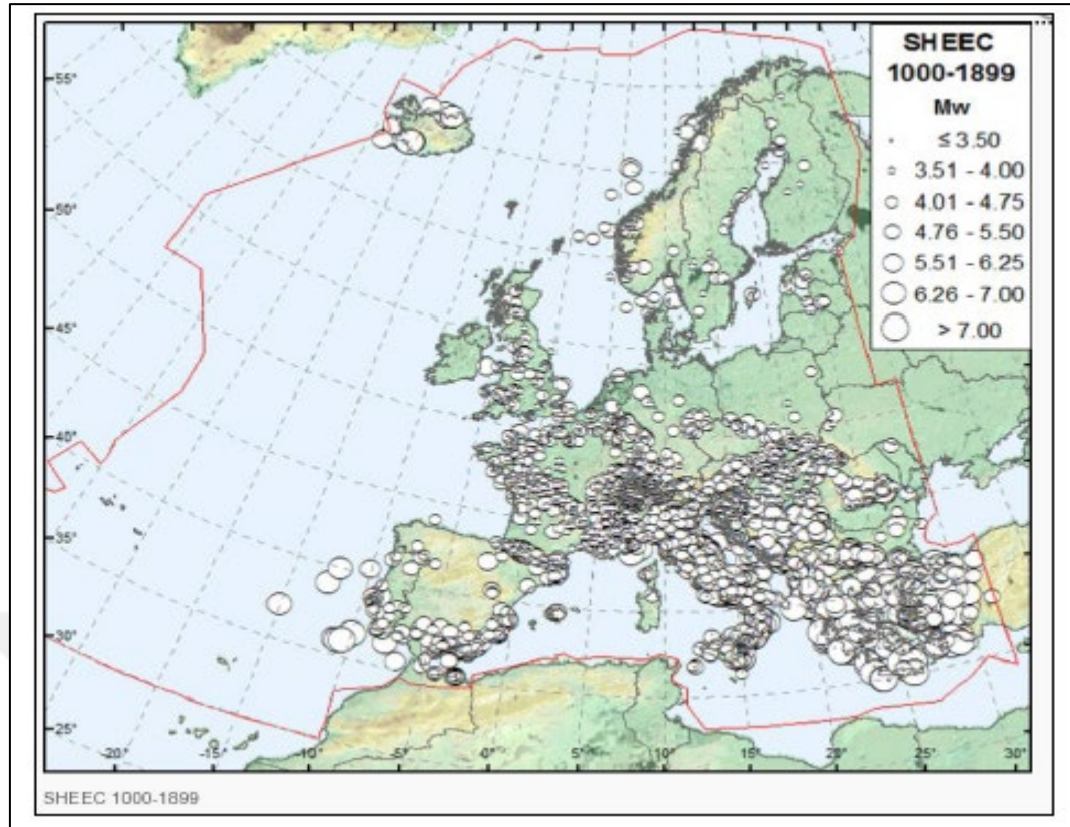


Figure 3.24: Historical earthquake records available in SHEEC.

In Table 3.4, the data are arranged/ converted to  $M_w$  using the eccentric density ( $I_o$ ) parameter from Stucchi et al. workings [Stucchi et al., 2012].

Table 3.4: Conversion formulas to moment magnitude considering regional characteristics within the scope of SHARE project.

| Region                                     | Number of Events | $I_o$ Range | $M_w$ Range | Formula                       | Standard deviation |
|--------------------------------------------|------------------|-------------|-------------|-------------------------------|--------------------|
| Betic                                      | 32               | 4 - 8       | 3.2 - 6.3   | $M_w = 1.48 + 0.52 \cdot I_o$ | 0.38               |
| Stable Continental                         | 26               | 4.5 - 8     | 3.5 - 5.7   | $M_w = 0.52 + 0.65 \cdot I_o$ | 0.25               |
| Western Alps                               | 17               | 5 - 8.5     | 3.4 - 5.7   | $M_w = 1.44 + 0.50 \cdot I_o$ | 0.31               |
| Apennines, Nort-Eastern Alps and Dinarides | 345              | 5.5 - 11    | 4.1 - 7.0   | $M_w = 2.18 + 0.42 \cdot I_o$ | 0.34               |
| Broad Aegean, Shallow                      | 62               | 5 - 10.0    | 4.6 - 7.2   | $M_w = 3.40 + 0.35 \cdot I_o$ | 0.25               |

Instrumental earthquakes that were recorded in the central and western Turkey between the years 1900-2006 under SHARE project are shown in Figure 3.25.

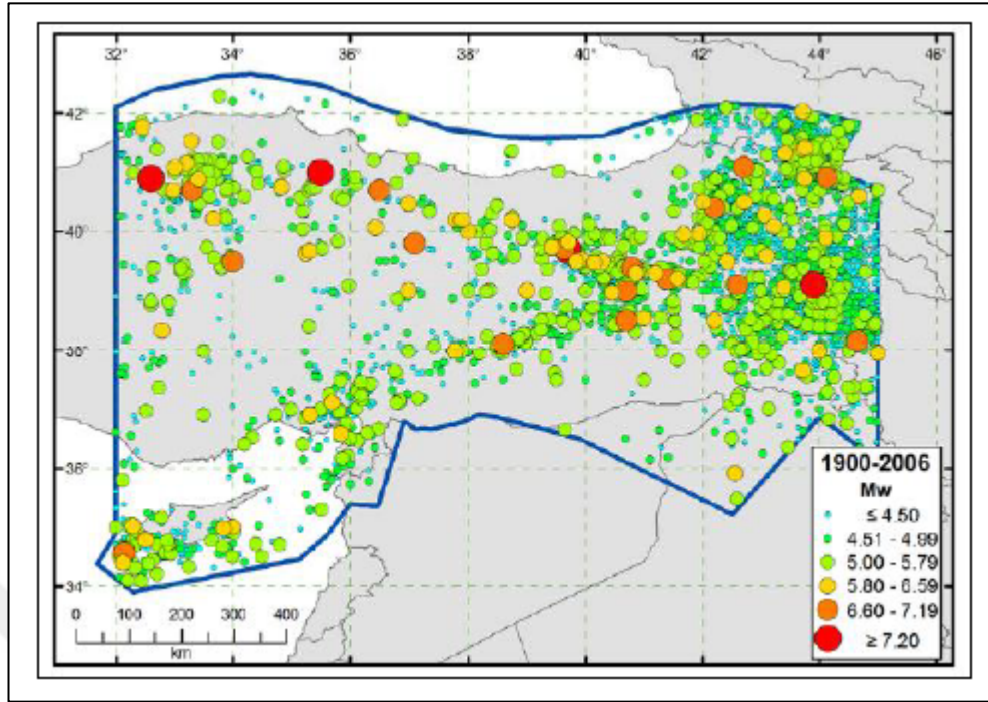


Figure 3.25: Instrumental earthquakes occurring in the central and western Turkey between the years 1900-2006 under SHARE project.

### 3.6.2. De-clustering Analysis

The Poisson model is a frequently used assumption in PSHA, which accepts that earthquakes occur independently of each other in terms of time and space. Therefore, to provide this independence condition in the Poisson model, aftershock and foreshocks data should be extracted from the earthquake data [Yüçemen, 2011].

Different studies have been carried out to determine aftershocks and foreshocks according to space and time (For example: [Omori, 1894], [Gardner and Knopoff, 1974], [Prozorov and Dziewonski, 1982], [Utsu et al., 1995], [Frans and Dyck, 1985], [Kagan, 2002] and [Savage and Rupp, 2000]. Aftershocks and foreshocks show similar distribution in the main shock region as spatially and temporally [Yüçemen, 2011]. Therefore, aftershocks and foreshocks are considered the same in determining secondary earthquakes. In all the mentioned studies, it is accepted that earthquakes of certain magnitudes cause secondary earthquake activity without considering the differences such as earthquake source, earthquake region, type and length of the fault [Yüçemen, 2011].

These aftershocks and foreshocks should be removed from the main shock to get the actual hazard output. Therefore, ZMAP software Wiemer (2001) was used for de-clustering in the EMME project. The methods adopted consider the approximation formulas given by Gardner and Knopoff (1974), Gruenthal and Uhrhammer (1986) in Table 3.5.

Table 3.5: Approximation formulas of distance and time windows by Gardner and Knopoff (1974), Gruenthal, and Uhrhammer (1986).

| Method                     | Distance (km)               | Time (days)                                                               |
|----------------------------|-----------------------------|---------------------------------------------------------------------------|
| Gardner and Knopoff (1974) | $10^{0.1238M+0.983}$        | $10^{0.032M+2.7389}$ , if $M \geq 6.5$<br>$10^{0.5409M-0.547}$ , else     |
| Gruenthal (pers.comm.)     | $10^{1.77+(0.037+1.02M)^2}$ | $e^{-3.95+(0.62+17.32M)^2}$ , if $M \geq 6.5$<br>$10^{2.8+0.024M}$ , else |
| Uhrhammer (1986)           | $e^{-1.024+0.804M}$         | $e^{-2.87+1.235M}$                                                        |

The change of magnitude with respect to time and distance for Gardner and Knopoff (1974), Gruenthal, and Uhrhammer (1986) are given in Figure 3.26.

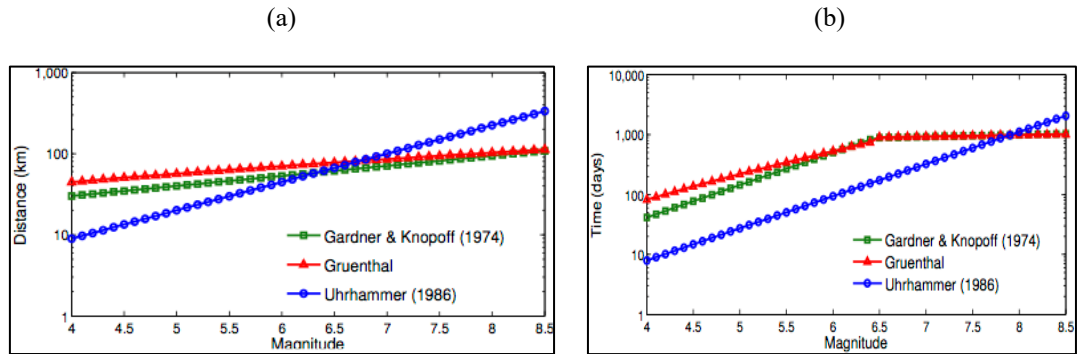


Figure 3.26: The change of magnitude with respect to time and distance (a) Distance-Magnitude relation, (b) Time-Magnitude relation.

In Table 3.6 and Figure 3.26, the de-clustering analysis made with Wiemer's ZMAP package shows the earthquake events that were extracted [Zare et al., 2014].

Table 3.6: Events extracted after the four algorithms used in the EMME catalog and events remaining in the last catalog.

| Algorithm                | No. of events | Number of events clustered | Number of Events in the Last Catalog | Number of Events Extracted from the Catalog |
|--------------------------|---------------|----------------------------|--------------------------------------|---------------------------------------------|
| Gardner and Knopoff 1974 | 28,244        | 3042                       | 10,131                               | 17,933                                      |
| Uhrhammer 1986           |               | 2398                       | 16,569                               | 11,495                                      |
| Reasenberg 1985          |               | 1041                       | 24,530                               | 3534                                        |
| Gruenthal                |               | 2960                       | 7272                                 | 20,792                                      |

Figure 3.27 shows the frequency-magnitude relation graphs in de-clustering analysis for each algorithm in the EMME catalog such as Gardner and Knopoff method, Grunthal method, Reseanberg method and Uhrhammer method.

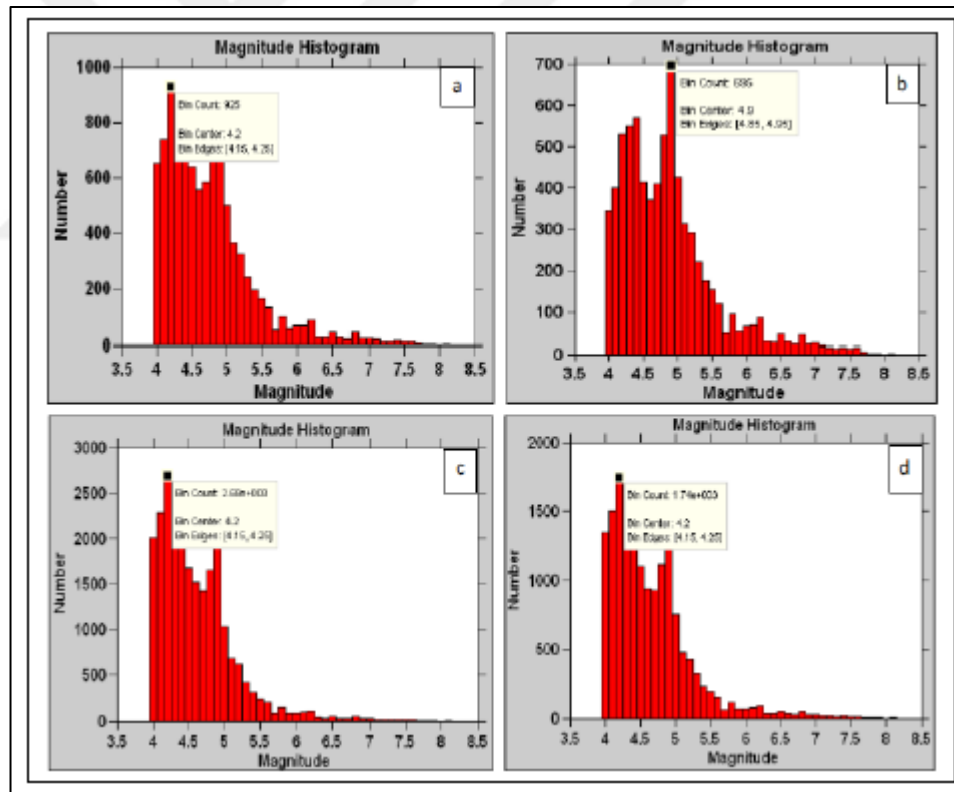


Figure 3.27: Frequency-magnitude relation graph in de-clustering analysis for each algorithm in the EMME catalog; a) Gardner and Knopoff method, b) Grunthal method, c) Reseanberg method and d) Uhrhammer method.

### 3.6.3. Completeness Analysis

Determining magnitude completeness ( $M_c$ ) in instrumental earthquake catalogs is a necessary and mandatory step for seismicity analysis. There are different methods to determine  $M_c$ . Completeness analysis in the EMME catalog is calculated using two main methods. The cumulative frequency-magnitude distribution of Gutenberg and Richter (1944) and Richter, (1958), which is the traditional and common method for catalog completeness (Zare et al., (2014)). (e.g.  $\log N = a - b * M$ , where  $N$  is the number of earthquakes with magnitude greater than or equal to magnitude  $M$ ; and  $a$  and  $b$  are seismicity and regional constants). With this method,  $M_c$  was determined for each region in the EMME project [Zare et al., 2014]. Table 3.7. presents the completeness analysis performed based on the start year with information of magnitudes and depths.

Table 3.7: Some of the seismicity information for each zone are magnitude (min., max. and  $M_c$ ), depth (in case of max. and 0 equality).

| Selection            | Mag.  |            | Depth. |     | Average Depth | Starting Years |
|----------------------|-------|------------|--------|-----|---------------|----------------|
|                      | $M_c$ | $M_{max.}$ | Max.   | =0  |               |                |
| Afghan, Pak.         | 4.90  | 7.92       | 302    | 185 | 9.0           | 817            |
| Kopeh , Dagh         | 4.90  | 7.65       | 45     | 84  | 15.0          | 11             |
| Iran                 | 4.90  | 7.73       | 108    | 93  | 15.0          | 735            |
| Makran region        | 4.80  | 8.12       | 166    | 17  | 9.0           | 1486           |
| Zagros region        | 4.90  | 7.45       | 184    | 145 | 13.0          | 858            |
| Alborz,<br>Azerbaij. | 4.30  | 7.83       | 94     | 415 | 11.0          | 551 B.C.       |
| Caucasus area        | 4.20  | 7.09       | 166    | 65  | 9.0           | 1252 B.C.      |
| Turkey region        | 4.40  | 7.76       | 168    | 377 | 9.0           | 997            |
| Saudi Arabian        | 4.90  | 7.51       | 102    | 108 | 8.0           | 32 B.C.        |

Similarly, table 3.8 presents the completeness analysis performed based on the certain magnitude ranges.

Table 3.8: Earthquake completion starting year for each region in EMME project.

| Selection      | $M_w \leq 4.0$ | $4.0 < M_w \leq 4.5$ | $4.5 < M_w \leq 5.0$ | $5.0 < M_w \leq 5.5$ | $5.5 < M_w$ |
|----------------|----------------|----------------------|----------------------|----------------------|-------------|
| Afghan, Pak    | 1981           | 1962                 | 1962                 | 1922                 | 1904        |
| Kopeh, Dagh    | 2001           | 1924                 | 1912                 | 1902                 | 1882        |
| Iran           | 1952           | 1935                 | 1914                 | 1252                 | 852         |
| Makran region  | 1971           | 1974                 | 1965                 | 1952                 | 1922        |
| Zagros region  | 1982           | 1952                 | 1926                 | 1852                 | 1822        |
| Alborz, Azerb. | 1954           | 1802                 | 1682                 | 1442                 | 852         |
| Caucasus area  | 1932           | 1914                 | 1872                 | 1847                 | 1782        |
| Turkey region  | 1961           | 1956                 | 1927                 | 1847                 | 1822        |

This study is further carried by Methiye [Methiye., 2020] and updated with all uncertainties involved for PSHA. The researcher has used OpenQuake program for modelling of seismicity distribution of the region. The input parameters for source typologies include point source, grid source and area sources respectively. The uncertainties and solutions involved in PSHA analysis such as magnitude distribution and recurrence models, activity rate and recurrence relations, maximum magnitude, GMPE's, aleatory variability and epistemic uncertainties along with logic tree application are explained in detail.

### 3.7. Seismic Hazard Analysis of Current Study

PGA and spectral acceleration values are obtained using classical PSHA of EMME model, PSHA by Monte Carlos (MC) of EMME model, classical PSHA of SHARE model, PSHA by Monte Carlos (MC) of SHARE model and by using TBDY 2018 [Methiye., 2020].

Table 3.9. presents the seismic hazard values in the form of PGA,  $S_a(0.1)$  and  $S_a(0.2)$  for the 5 regions along with the latitude and longitude coordinates. The maximum hazard value is observed in the Sakarya region following its nearby regions of İzmit, Sapanca, Düzce and Gebze respectively. The values are obtained using classical PSHA analysis of the EMME model.

Table 3.9: Seismic hazard values using classical PSHA of EMME model.

| District | Longitude | Latitude | PGA   | Sa (0.1) | Sa (0.2) |
|----------|-----------|----------|-------|----------|----------|
| Gebze    | 29.3      | 40.9     | 0.362 | 0.852    | 0.283    |
| İzmit    | 29.9      | 40.7     | 0.861 | 2.004    | 0.528    |
| Sapanca  | 30.3      | 40.7     | 0.838 | 1.974    | 0.525    |
| Sakarya  | 30.5      | 40.7     | 0.939 | 2.219    | 0.584    |
| Düzce    | 31.1      | 40.9     | 0.777 | 1.884    | 0.448    |

Similarly, PSHA is also performed by using MC method of the EMME model. The seismic hazard values for this method are given in table 3.10. By comparing the hazard values of both classical PSHA and PSHA by MC method of the EMME model, it can be noted that both methods are compatible to each other. The values obtained by MC method a little higher and it can be due to the reason that in MC method, more uncertainties are considered during analysis.

Table 3.10: Seismic hazard values using PSHA by MC method of EMME model.

| District | Longitude | Latitude | PGA   | Sa (0.1) | Sa (0.2) |
|----------|-----------|----------|-------|----------|----------|
| Gebze    | 29.3      | 40.9     | 0.363 | 0.860    | 0.287    |
| İzmit    | 29.9      | 40.7     | 0.866 | 1.988    | 0.529    |
| Sapanca  | 30.3      | 40.7     | 0.843 | 2.021    | 0.531    |
| Sakarya  | 30.5      | 40.7     | 0.945 | 2.212    | 0.589    |
| Düzce    | 31.1      | 40.9     | 0.774 | 1.893    | 0.448    |

Further, seismic hazard values using classical PSHA of SHARE model are presented in table 3.11. By comparing the values of Table 3.9 and Table 3.11, a minor difference is obtained in the result output by using classical PSHA of SHARE and EMME models. The values of EMME model are bit in upward trend and the reason behind this is that the EMME catalog was rich of earthquake records than the SHARE model.

Table 3.11: Seismic hazard values using classical PSHA of SHARE model.

| District | Longitude | Latitude | PGA   | Sa (0.1) | Sa (0.2) |
|----------|-----------|----------|-------|----------|----------|
| Gebze    | 29.3      | 40.9     | 0.433 | 0.952    | 0.326    |
| İzmit    | 29.9      | 40.7     | 0.575 | 1.212    | 0.442    |
| Sapanca  | 30.3      | 40.7     | 0.628 | 1.344    | 0.477    |
| Sakarya  | 30.5      | 40.7     | 0.710 | 1.525    | 0.537    |
| Düzce    | 31.1      | 40.9     | 0.683 | 1.454    | 0.455    |

Likewise, seismic hazard values using PSHA by MC method of SHARE model is also performed and the output values are given in table 3.12. The result output by both models has much compatibility as can be seen from tables 3.10 and 3.12.

Table 3.12: Seismic hazard values using PSHA by MC method of SHARE model.

| District | Longitude | Latitude | PGA   | Sa (0.1) | Sa (0.2) |
|----------|-----------|----------|-------|----------|----------|
| Gebze    | 29.3      | 40.9     | 0.432 | 0.959    | 0.326    |
| İzmit    | 29.9      | 40.7     | 0.569 | 1.200    | 0.442    |
| Sapanca  | 30.3      | 40.7     | 0.620 | 1.365    | 0.478    |
| Sakarya  | 30.5      | 40.7     | 0.714 | 1.543    | 0.547    |
| Düzce    | 31.1      | 40.9     | 0.684 | 1.449    | 0.453    |

Additionally, seismic hazard values presented for current region by using TBDY 2018 catalog data are presented in Table 3.13.

Table 3.13: Seismic hazard values using TBDY 2018.

| District | Longitude | Latitude | PGA   | Sa (0.1) | Sa (0.2) |
|----------|-----------|----------|-------|----------|----------|
| Gebze    | 29.3      | 40.9     | 0.409 | 0.897    | 0.224    |
| İzmit    | 29.9      | 40.7     | 0.72  | 1.589    | 0.381    |
| Sapanca  | 30.3      | 40.7     | 0.698 | 1.543    | 0.376    |
| Sakarya  | 30.5      | 40.7     | 0.718 | 1.587    | 0.383    |
| Düzce    | 31.1      | 40.9     | 0.489 | 1.068    | 0.261    |

In a nutshell, it is observed that the analysis values made with the SHARE data and the TBDY 2018 values are quite compatible with each other. However, by comparing EMME data and the TBDY 2018 values, it is seen that values for Düzce are larger than TBDY 2018 values. The reason behind this difference can be due to large source values of the regions, different GMPEs application and differentiation in earthquake catalogues. Figure 3.28 shows the seismic hazard analysis in a summarized form.

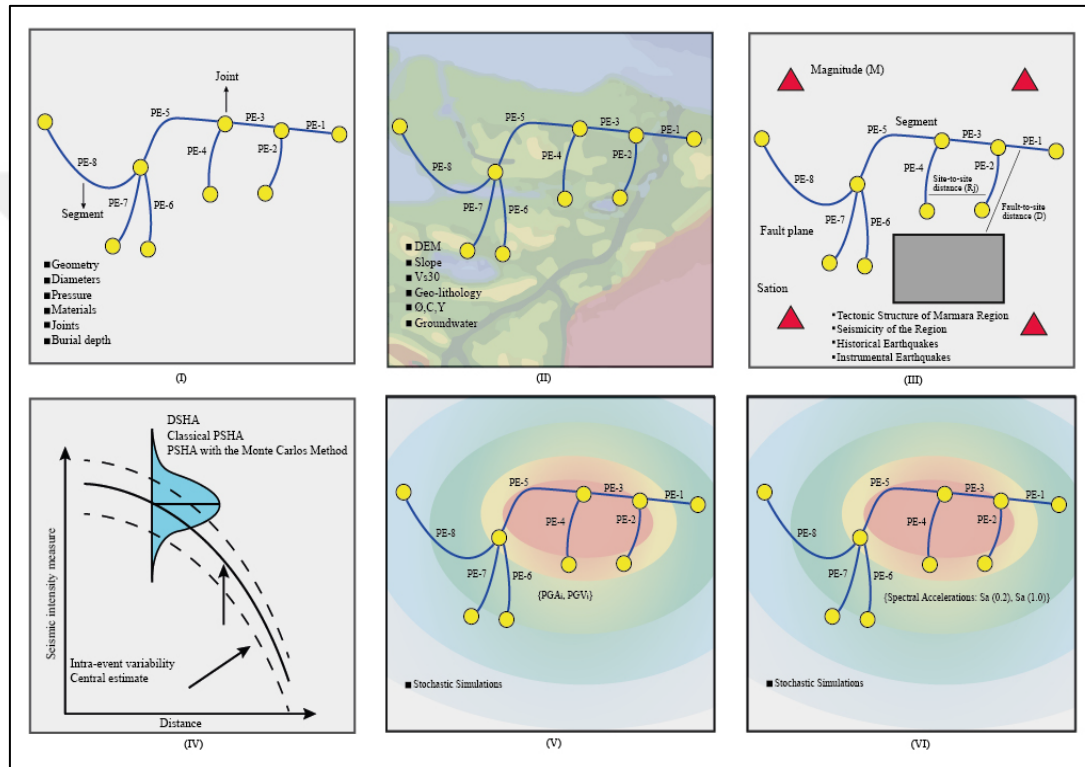


Figure 3.28: Seismic hazard evaluation summary: (I) BCGPs location (II) historical maps (III) hazard involving factors (IV) analysis methods (V) PGA (VI) Sa (0.1), Sa (0.2).

## 4. SEISMIC WAVE PROPAGATION HAZARD

Pipelines cross through diverse environments and face various soil conditions and seismic hazards in their routes of laying. Sometimes, it is not possible to change the path due to unavailability of land or environmental conditions.

### 4.1. Past Damage Due to Seismic Wave Propagation

Pipelines cross through diverse environments and face various soil conditions and seismic hazards in their routes of laying. Sometimes, it is not possible to change the path due to unavailability of land or environmental conditions. Seismic hazards are classified into two categories. One is transient seismic wave propagation (TSWP) hazard and other one is permanent ground deformation (PGD) hazard [Akram and Zulfikar, 2020]. Transient wave propagation in BCGPs generates compressive, tensile, and bending stresses that lead to the failure of system. It has been seen in 1985 Mexico City earthquake that only TSWP hazard caused the failure of buried potable water pipes.



Figure 4.1: Failure of buried potable water pipes during 1985 Mexico City earthquake.

TSWP hazard on buried pipelines in recent major earthquakes have been summarized in Table 4.1.

Table 4.1: WP hazard on buried pipelines in earlier foremost earthquakes.

| <b>Location</b>      | <b>Earthquake<br/>(Year – Mw)</b> | <b>Seismic Damages</b>                                                                                                                                         |
|----------------------|-----------------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------|
| The Great Kanto      | 1923 – 8.2                        | Almost 4000 pipe fractures have been noticed.<br>Small diameter made of cast iron are damaged more.<br>Hazard was due to phase difference of travelling waves. |
| Puget Sound          | 1965 - 6.7                        | Failure at repairs have been noticed. 15% repairs were totally broken and 85% were leaked due to TSWP hazard.                                                  |
| Santa Rosa           | 1969 - 5.7                        | Water pipes were damaged due to seismic TSWP and corrosion phenomena.                                                                                          |
| San Fernando         | 1971 - 6.7                        | 1400 breaks in water, gas and sewage pipes were reported.                                                                                                      |
| Tangshan             | 1976 - 7.8                        | 80% pipe breaks at joints due TSWP hazard.                                                                                                                     |
| Miyagi Ken-Oki       | 1978 - 7.7                        | 237 failures in buried steel pipes, mostly small diameter pipes were damaged.                                                                                  |
| Coalinga             | 1983 - 6.2                        | Many breaks and leakage in buried pipes were reported due to TSWP effects.                                                                                     |
| Mexico City          | 1985 - 8.0                        | Tape water pipes were damaged due to TSWP hazard.                                                                                                              |
| Whittier Narrows     | 1987 - 5.9                        | TWP effect on water pipes have been observed.                                                                                                                  |
| Landers and Big Bear | 1992 - 6.5                        | Failure has been observed in water pipes.                                                                                                                      |
| Indian Ocean         | 2004 - 9.1                        | Driftwood and rubbles due to Tsunami waves caused failure of buried pipelines.                                                                                 |
| Tohoku               | 2011 - 9.0                        | Damage was due to TSWP as well as PGD hazards.                                                                                                                 |

## 4.2. Design Criteria for Seismic Wave Propagation

Ground motions are generally generated as earthquake happens and these motions are in the form of body or surface waves. Body waves propagate more quickly than the surface waves. That is why, it is considered that the superstructures are prone to seismic hazards due to seismic waves as comparing to the substructures. However, pipelines buried at shallow depths have also damaged in recent earthquakes. Therefore, pipelines burial depth parameter is an important parameter in seismic designs.

Following references can be taken as benchmark points for designing of buried pipelines against seismic waves. a). Buried pipelines that are welded well cannot face any serious effect due to seismic wave propagations. The points of serious attraction where any damage to the pipelines may happen can be at i. Change of stiff to soft soil natures, ii. Diffusion of pipelines inside the valve boxes iii. Pipes that are close to the pumping stations iv. Pipes with T connections v. Poor pipe fitting and valves. Hence, a special design criterion for buried pipelines should be adopted before taking them to real applications. b). Corroded pipes and the pipes used in older times especially iron made pipes with bell and spigot joints can be damaged easily by TSWP. c). Assortment of TSWP and their speed of wave movement should be classified on geophysical natures. d). Soil-pipe interaction is an important tool to be studied carefully. The proper use of wrapping/ coating or backfill soil on the buried pipes can be helpful in minimizing the effects of TSWP.

For the evaluation of buried pipes behavior against earthquakes, generally axial strains are calculated and then compared with the limit state values to examine their performance levels. Flexural strains are usually neglected in seismic designing of buried pipes as these values are exceedingly small. Figure 4.1. shows the procedure adopted for TSWP analysis of BCGPs for current case study.

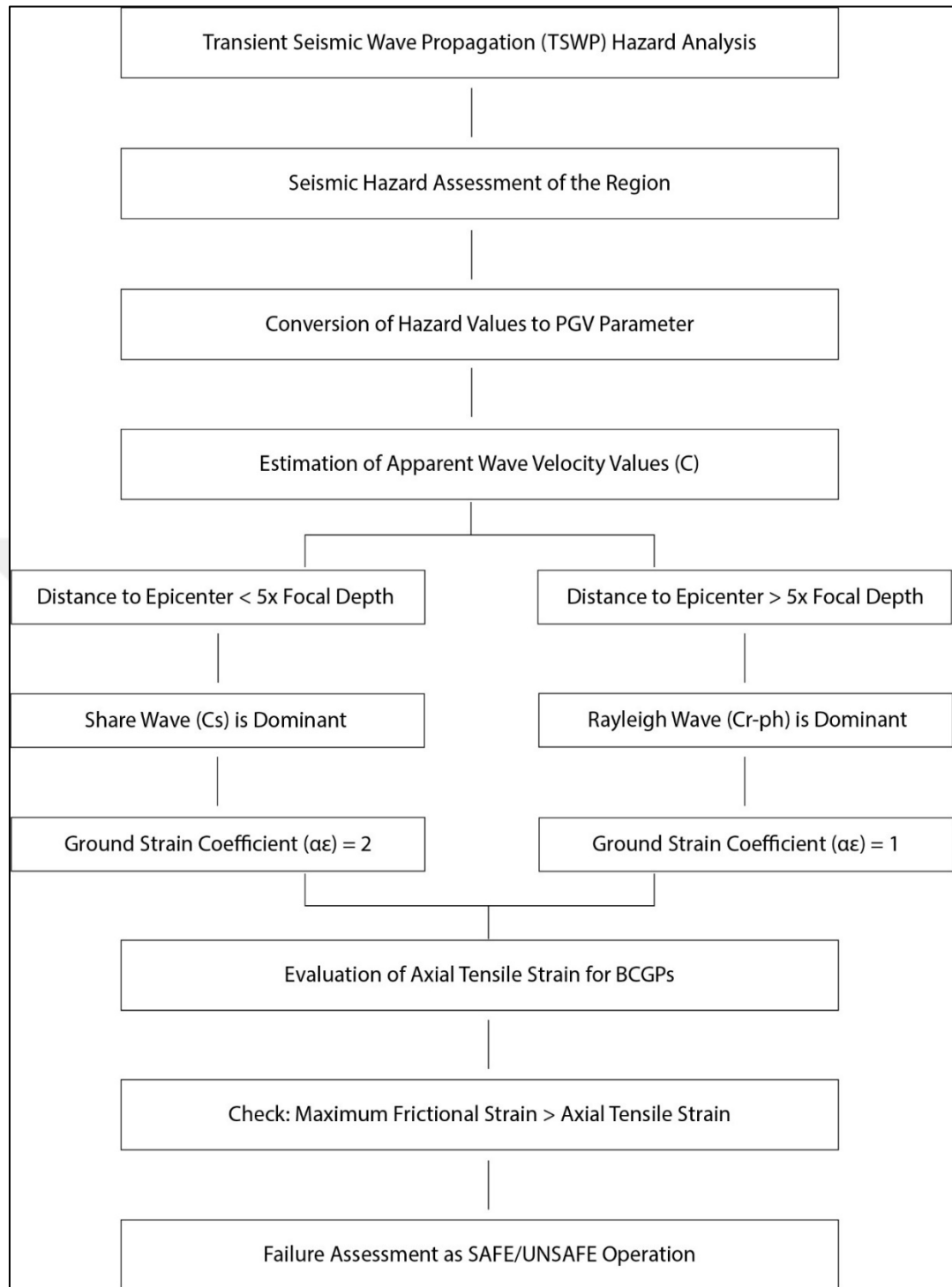


Figure 4.2: Flowchart for strain estimation due to seismic wave propagation hazard.

### 4.3. Apparent Wave Propagation Velocity (AWPV)

An important parameter necessary to evaluate during seismic design of buried pipelines is the AWPV. This parameter is used to find the strain induced in the pipe.

Therefore, AWPV for both surface and body waves should be calculated, as pipelines are placed at shallow depths normally 1-3 m. Body waves contain two types of waves namely S and P waves. Generally, S-waves are preferred for design purposes as they transfer more energy and cause more ground motions as comparing to the P waves. Surface waves are further classified into two types: R and L waves. R waves are usually considered due to the aspect that they generate potential axial strain in buried pipe than bending strain produced by L waves. Designers follow a general rule by considering velocity of R wave for the sites having distance to epicentre greater than 5 times to focal depth as shown in Figure 4.3.

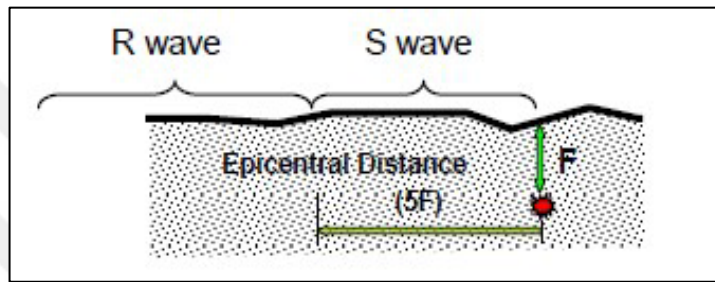


Figure 4.3: Examination of S-wave and R-wave in buried continuous pipeline design.

#### 4.3.1. For S-Waves

In examination of body waves at the ground surface, AWPV of shear waves appears multiple times more than the shear wave velocity of the ground surface material. The origin of seismic waves in deep soils and then their propagation through different soft layers follows a phenomenon of refraction and make a concave travel line. Therefore, body waves that reaches at surface of the ground appears to have a small angle value by comparing it to vertical one as shown in Figure 4.3. Considering  $\theta$  as angle of incidence and  $C_s$  as shear wave velocity of ground surface layer, then AWPV can be calculated as by using equation (4.1).

$$Cs\_apparent = \frac{Cs}{\sin \theta} \quad (4.1)$$

In above equation, when  $\theta$  approaches to zero value, AWPV could be of infinity.

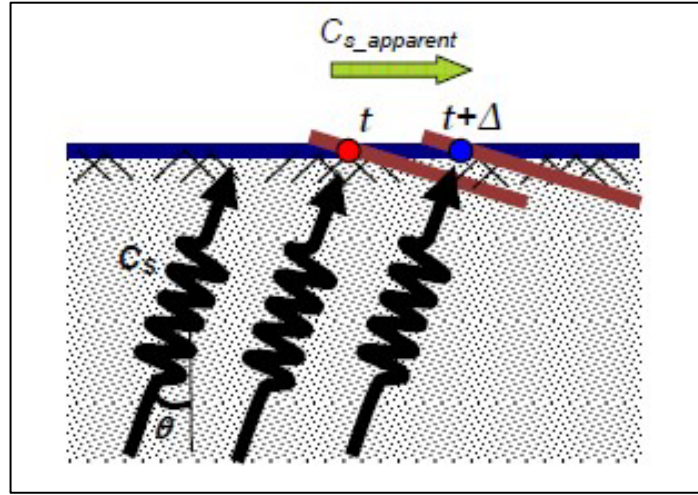


Figure 4.4: Seismic S-wave propagation style.

#### 4.3.2. For R-Waves

From definition, the waves that travel along the parallel surface of ground are considered as R waves, so their AWPV and phase velocities are same. It is extremely complicated to calculate the phase velocity of R-wave. However, in literature, a specific procedure is provided by O'Rourke et al. [O'Rourke et al., 1999] that can be helpful for evaluating it. The AWPV for R-wave can be consider by using equation (4.2).

$$Cr\_apparent = Cr\_ph \quad (4.2)$$

In above equation,  $Cr\_ph$  is the phase velocity of R wave as shown in Figure 4.4.

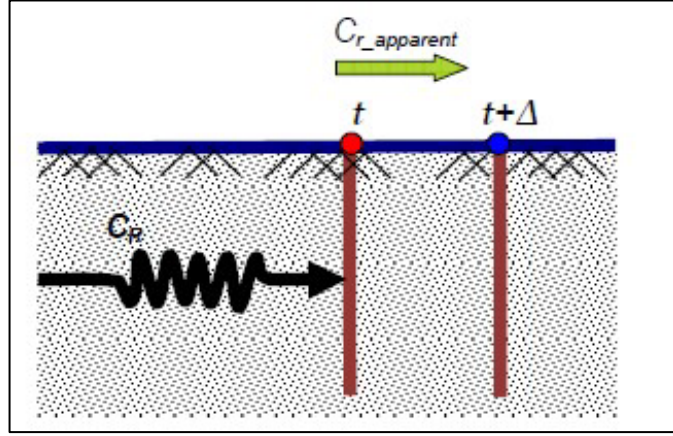


Figure 4.5: Examination of seismic AWPV of R-wave.

#### 4.4. Longitudinal Axial Strain

Among the surface waves, the shear wave (S-wave) is considered as an important wave that is the sole cause of substructure failure. As the ground strain coefficient is dependent on angle of incidence of seismic wave with pipeline and the property of wave itself. So, in case of S-wave the apparent propagation velocity in the direction of pipelines axis is found as using equation (4.3).

$$C_{s\_apparent} = \frac{C_s}{\sin\theta \cdot \cos\gamma} \quad (4.3)$$

The maximum ground strain happens, when  $\Theta$  and  $\Upsilon$  both equals to  $45^\circ$  is determined in equation (4.4).

$$C_{s\_apparent} = \frac{C_s}{\frac{1}{\sqrt{2}} \cdot \frac{1}{\sqrt{2}}} = 2C_s \quad (4.4)$$

Therefore, the ground strain coefficient ( $\alpha\epsilon$ ) value for S-wave is considered as 2. The velocity of seismic wave propagation is 2.0 km/sec for S-waves. Thus, the maximum axial strain in the BCGPs can be evaluated as using equation (4.5).

$$\varepsilon_a = \frac{V_g}{c_{apparent}} = \frac{V_g}{\alpha_e c} \quad (4.5)$$

#### 4.5. Axial Strain Induced Due to Soil Friction

The frictional force ( $t_u$ ) of soil to resist the movement of buried pipes is dependent on few factors such as i. the slippage is taking place across the whole length of the buried pipe, ii. The frictional force is uniform that acts on the surface of buried pipes (O'Rourke et al., 1999). The frictional resistance over BCGPs is shown in Figure 4.6.

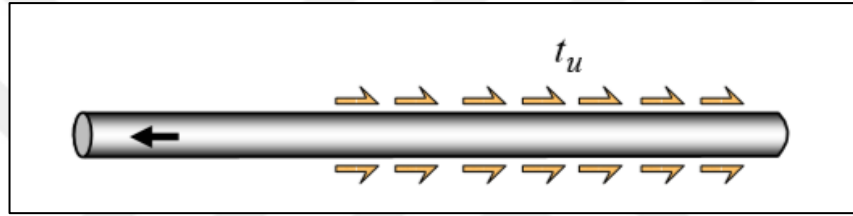


Figure 4.6: Examination of seismic AWPV of R-wave.

$\lambda$  symbol represents the apparent wavelength of seismic wave that is the product of apparent wavelength velocity of seismic wave and natural fundamental period of the ground surface. The mathematical expression to evaluate axial strain in buried continuous pipes due to soil friction is given in equation (4.6).

$$\varepsilon_a \leq \frac{t_u \lambda}{4AE} \quad (4.6)$$

In above equation,  $t_u$  = Frictional forces or axial force due to soil friction on BCGPs,  $\lambda$  = Apparent wavelength of earthquake waves (Generally value is 1.0 km if detailed information is not available),  $A$  = BCGPs cross sectional area, and  $E$  = Modulus of elasticity.

## 4.6. Properties of Soil Spring

To model and analyses soil- buried pipe interaction, it is necessary to study the soil-spring properties in detail. Soil-spring properties for axial, lateral and vertical directions are explained in following section in details. These properties are used for 3D analysis of buried pipe in ZeusNL program.

### 4.6.1. Axial Side

The geotechnical properties of backfill soil used for filling BCGPs trench is an important parameter to estimate the axial soil-spring properties. This assumption is valid in such cases where the response of BCGPs movement to the layers backfill soil in not considerably affected by the soil available outside the trench. The representation of axial soil spring force and related displacement is shown in Figure 4.7.

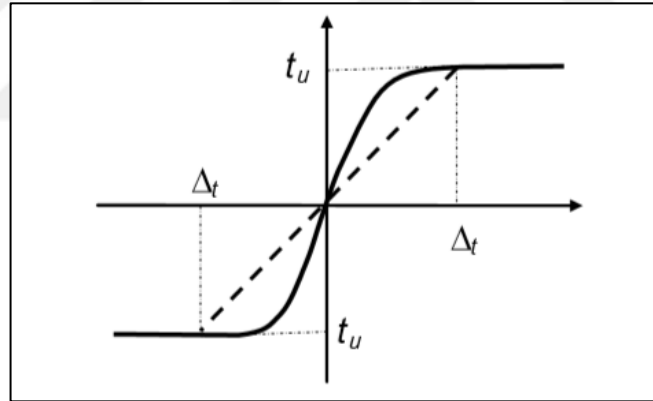


Figure 4.7: Axial soil-spring representation.

Maximum axial soil spring properties in terms of axial force per unit length can be evaluated using equation (4.7).

$$t_u = \pi D c \alpha + \pi D H \bar{\gamma} \frac{1 + K_0}{2} \tan \delta' \quad (4.7)$$

In above equation, D is the exterior diameter of BCGPs, c: Cohesion coefficient of backfilled soil, H: Vertical depth of soil layers over the central length of BCGPs,  $\bar{\gamma}$ :

Effective soil unit weight,  $\delta'$ : Interface frictional angle b/w pipe and soil =  $f\phi$ ,  $\phi$ : Soil internal friction angle,  $f$ : Frictional factor for various kinds of soils as given in table 4.2.

Table 4.2:  $f$ -value for various kinds of pipe coatings.

| Pipe External Coating | $f$ - value |
|-----------------------|-------------|
| Concrete Material     | 1.00        |
| Coal Tar Layer        | 0.90        |
| Rough Upper Steel     | 0.80        |
| Smooth Upper Steel    | 0.70        |
| Epoxy bonded          | 0.60        |
| HDPE                  | 0.60        |

$K_0$  is the coefficient of soil pressure at rest. There are two available options to determine this coefficient. One is by using Jacky's equation as given (4.8).

$$K_0 = 1 - \sin \phi \quad (4.8)$$

The other way is based on drainage and compaction conditions of soil, and coefficient values for soil pressure at rest can be taken from the below Table 4.3.

Table 4.3:  $K_0$  value for various kinds of soils.

| Soil Type                      | $K_0$ - value |
|--------------------------------|---------------|
| Loose Compacted Soil           | 0.50 – 0.60   |
| Dense Compacted soil           | 0.30 – 0.50   |
| Clayey Soil (Drained Cond.)    | 0.50 – 0.60   |
| Clayey Soil (Un-Drained Cond.) | 0.80 – 1.10   |
| OCS                            | 1.00 – 1.30   |

$\alpha$  = Soil adhesion factor and this factor can be calculated as in equation (4.9).

$$\alpha = 0.608 - 0.123c - \frac{0.274}{c^2+1} + \frac{0.695}{c^3+1} \quad (4.9)$$

In above equation  $c$  is the cohesion coefficient of backfilled soil in terms of kPa/100.

#### 4.6.2. Lateral Side

The relationship of maximum lateral force and corresponding displacement is graphically shown in Figure 4.8.

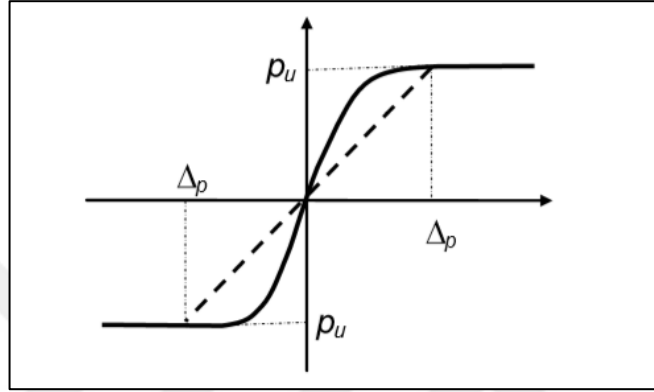


Figure 4.8: Lateral soil-spring representation.

Maximum lateral soil spring properties in terms of lateral force per unit length can be evaluated using equation (4.10).

$$P_u = N_c c D N_q \bar{\gamma} H D \quad (4.10)$$

In above equations,  $N_{ch}$  and  $N_{qh}$  represents the horizontal bearing capacity factors for clay and sandy soils respectively and these factors can be calculated by using equation (4.11) and (4.12) respectively.

$$N_q = a + bx + cx^2 + dx^3 + ex^4 \quad (4.11)$$

$$N_c = a + bx + \frac{c}{(x+1)^2} + \frac{d}{(x+1)^3} \leq 9 \quad (4.12)$$

Whereas  $x$  factor is the ratio of burial depth to diameter of the BCGPs ( $H/D$ ). In ALA 2001,  $N_c$  and  $N_q$  values are presented and are summarized in Table 4.4.

Table 4.4: Bearing capacity factor for lateral side of soil.

| Factor | f    | a      | b      | c       | d                       | e                       |
|--------|------|--------|--------|---------|-------------------------|-------------------------|
| $N_c$  | 0.0  | 6.753  | 0.064  | -11.065 | 7.118                   |                         |
| $N_q$  | 20.0 | 2.398  | 0.441  | -0.031  | $1.058 \times 10^{-3}$  | $-1.751 \times 10^{-5}$ |
| $N_q$  | 25.0 | 3.331  | 0.842  | -0.089  | $5.604 \times 10^{-3}$  | $-1.321 \times 10^{-4}$ |
| $N_q$  | 30.0 | 4.564  | 1.235  | -0.087  | $4.273 \times 10^{-3}$  | $-9.158 \times 10^{-5}$ |
| $N_q$  | 35.0 | 6.815  | 2.0121 | -0.145  | $7.649 \times 10^{-3}$  | $-1.679 \times 10^{-4}$ |
| $N_q$  | 40.0 | 10.957 | 1.779  | 0.044   | $-5.423 \times 10^{-3}$ | $-1.149 \times 10^{-4}$ |
| $N_q$  | 45.0 | 17.655 | 3.311  | 0.047   | $-6.439 \times 10^{-3}$ | $-1.298 \times 10^{-4}$ |

### 4.6.3. Vertical Side

In analysis of vertical soil spring properties for two cases namely uplift and bearing, two different soil properties are to be considered. For bearing, local soil present at the study site are considered. But for uplift, nature of backfill soil properties are examined. Vertical soil spring properties are presented in the Figure 4.9.

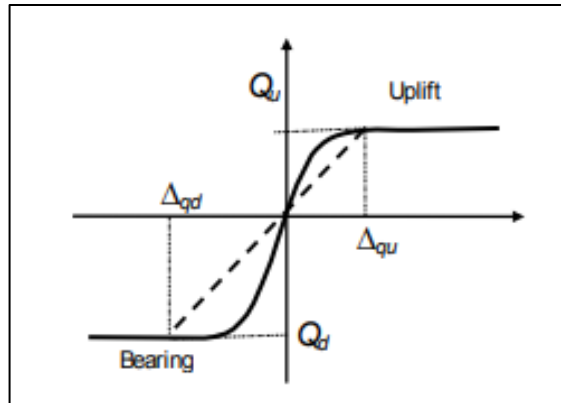


Figure 4.9: Vertical soil-spring representation.

#### 4.6.3.1. Vertical uplift

Maximum vertical soil spring properties in terms of vertical uplift per unit length of buried pipe can be evaluated as equation (4.13).

$$Q_u = N_{cv} cD + N_{qv} \bar{\gamma}HD \quad (4.13)$$

In equation (4.13);  $N_{cv}$  denotes the vertical uplifting factor for clayey soil and  $N_{qv}$  vertical factor for sandy soil and these factors can be calculated using equations (4.14) and (4.15).

$$N_{cv} = 2 \left( \frac{H}{D} \right) \leq 10 \quad (4.14)$$

$$N_{qv} = \left( \frac{\phi H}{44D} \right) \leq N_{qv} \quad (4.15)$$

#### 4.6.3.2. Bearing uplift

Maximum vertical soil spring properties in terms of bearing uplift per unit length of buried pipe can be evaluated as.

$$Q_d = N_c cD + N_q \bar{\gamma}HD + N_\gamma \gamma \frac{D^2}{2} \quad (4.16)$$

In above equation  $N_c$ ,  $N_q$ , and  $N_\gamma$  represents bearing capacity factors that can be calculated from the below equations (4.17), (4.18) and (4.19).

$$N_c = [\cot(\phi + 0.001)] \left\{ \exp[\pi \tan(\phi + 0.001)] \tan^2 \left( 45 + \frac{\phi + 0.001}{2} \right) - 1 \right\} \quad (4.17)$$

$$N_q = \exp(\pi \tan \phi) \tan^2 \left( 45 + \frac{\phi}{2} \right) \quad (4.18)$$

$$N_\gamma = \exp(0.18\phi - 2.5) \quad (4.19)$$

Bearing capacity factors are presented in the Figure 4.10.

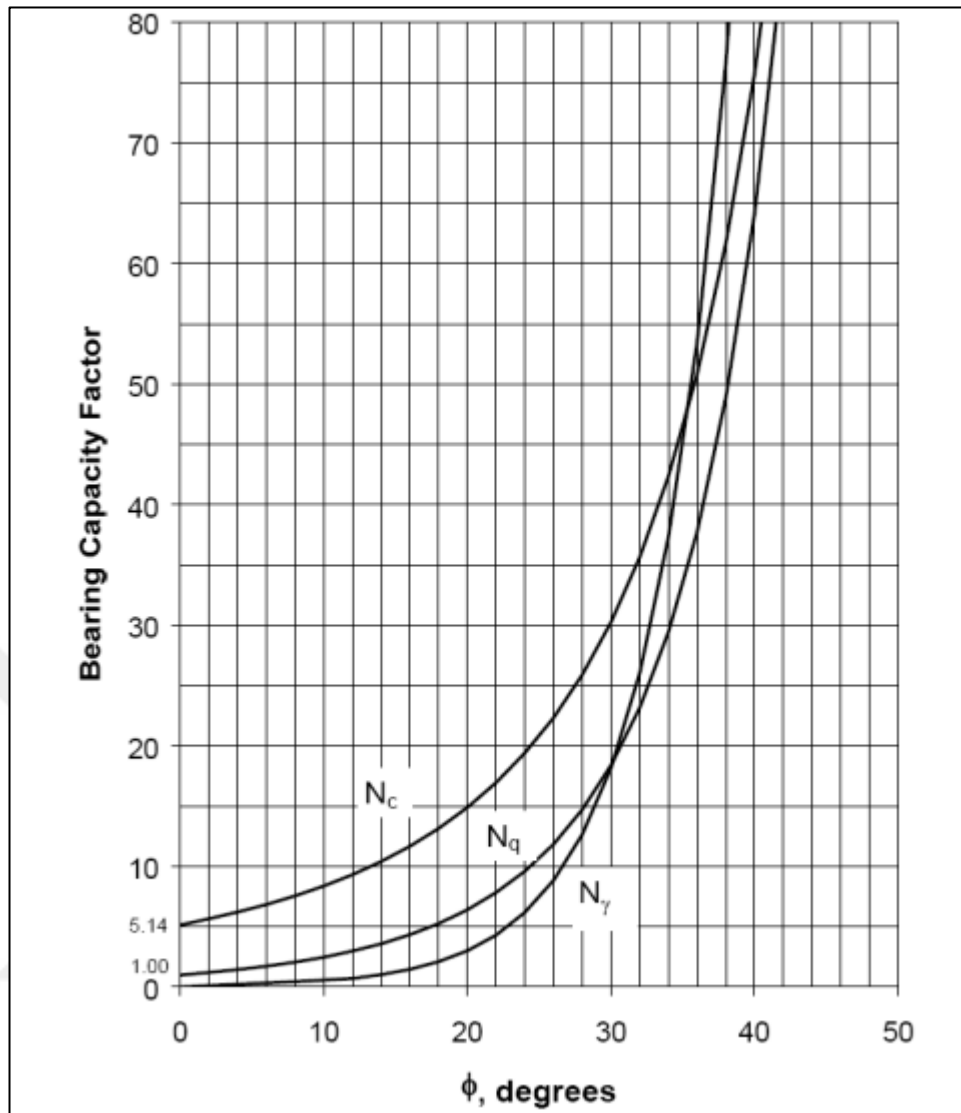


Figure 4.10: Bearing capacity factors.

#### 4.7. Seismic Safety Analysis Against TSWP: Current Study

Earthquake is one of the natural disasters that has huge impact on the lifeline facilities such as BCGPs [O'Rourke and Palmer, 1996]. Research on seismic safety of BCGPs against TSWP has taken an important consideration in recent years although there is still need of development to overcome the future challenges [Porras and Najafi 2010]. Many methodologies are explained in literature, for instance Lanzano et al. [Lanzano et al., 2015] collected database information on the damaged BCGPs after the earthquakes. The researchers believe that PGD hazard has more impact, however TSWP can also cause damage to BCGPs [Wham, O'Rourke, 2015]. Material

composition of BCGPs can also change the damage behavior [Edkins et al., 2016]. In previous studies, experimental and numerical workings are explained in well manner, however there is still a space available for quick and reliable methodology to be developed. In current research, an algorithm is developed for fast and reliable safety check of BCGPs against TSWP hazard as explained in Figure 4.1.

The chosen study area of current research is shown in Figure 4.11. Pipe is divided into 8 segments (PE-1 to PE-8) and observation is made at joints levels. The soil is considered as uniform dense/stiff soil and is classified as D type soil in NEHRP soil category. The values of shear wave velocity of the soil are taken as  $180 < V_s \leq 360$ . All 8 segments of pipes are considered as continuous buried gas steel pipes with pipe importance factor as 1.5. The seismic hazard values computed in chapter-3 for the current regions are converted to PGV for TSWP analysis. The shear wave is considered as dominant seismic wave with velocity of propagation as 2 kms/sec. As S-wave is dominant, therefore ground strain co-efficient value is taken as 2.0. The burial depth of all 8 segment of pipes is taken as constant value of 2.5m from the top surface layer. Additionally, the contribution of other factors such as internal gas pressure and change in temperature are also considered. In current study, the value of internal gas pressure is considered as 7.5 MPa. The temperature at the installation of pipes is taken as 25°C and during operation as 40°C.

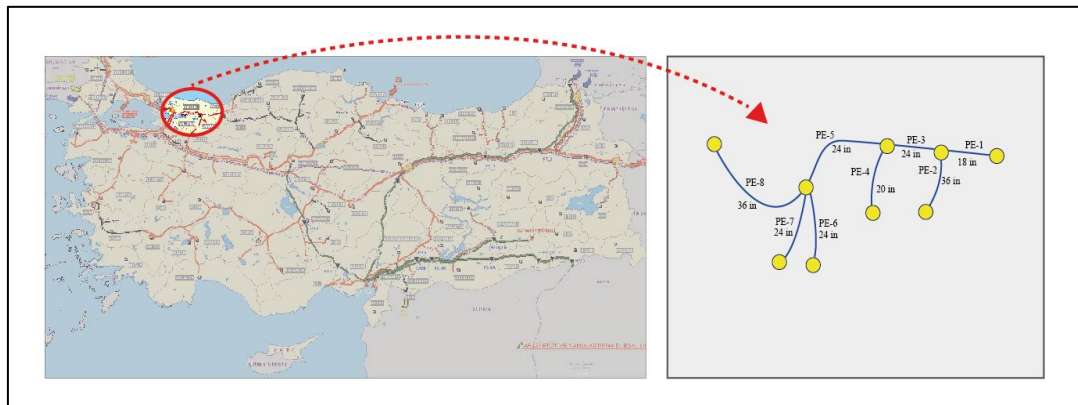


Figure 4.11: Pipe network for current study.

The algorithm presented in Figure 4.1. is applied to check the TSWP analysis. Longitudinal axial strain is computed in all 8 segments of the current study as given in Table 4.5 and the results are then compared with the threshold limit values (ALA,

2001) to check the safe operation. From Table 4.5. the analysis performed on the 8 segments of pipes show safe operation.

Table 4.5: Safety evaluation due to TWP hazard.

| Loc. | Computed Tension | Allowable Tension | Status      |
|------|------------------|-------------------|-------------|
| PE-1 | 0.0012           | 0.03              | Operational |
| PE-2 | 0.0013           | 0.03              | Operational |
| PE-3 | 0.0012           | 0.03              | Operational |
| PE-4 | 0.0013           | 0.03              | Operational |
| PE-5 | 0.0012           | 0.03              | Operational |
| PE-6 | 0.0010           | 0.03              | Operational |
| PE-7 | 0.0010           | 0.03              | Operational |
| PE-8 | 0.0011           | 0.03              | Operational |

## 4.8. Parametric Studies

The current study is further extended by changing the various parameters to examine the behavior of BCGPs. The comparison of factorial outputs is explained in the following sections.

### 4.8.1. Effect of Ground Strain Coefficient for S and R Wave

It is noted that computed seismic strain due to S-wave are less than R-Wave as S-Waves have larger ground strain coefficient. The effect of ground strain coefficient for S and R waves is shown in Figure 4.12.

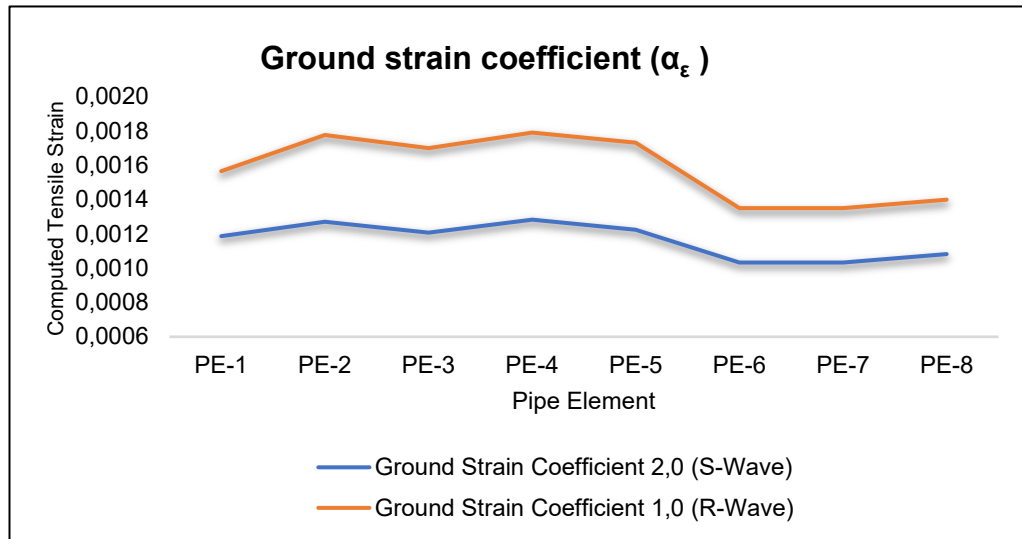


Figure 4.12: Effect of ground strain coefficient for S and R waves.

#### 4.8.2. Effect of Pipe Class Importance Factors

Pipe class importance factors are used as per their functional requirement and in current study, it is observed that lower the  $I_p$  values induce less tensile strains. The effect of pipe class importance factor is shown in Figure 4.13.

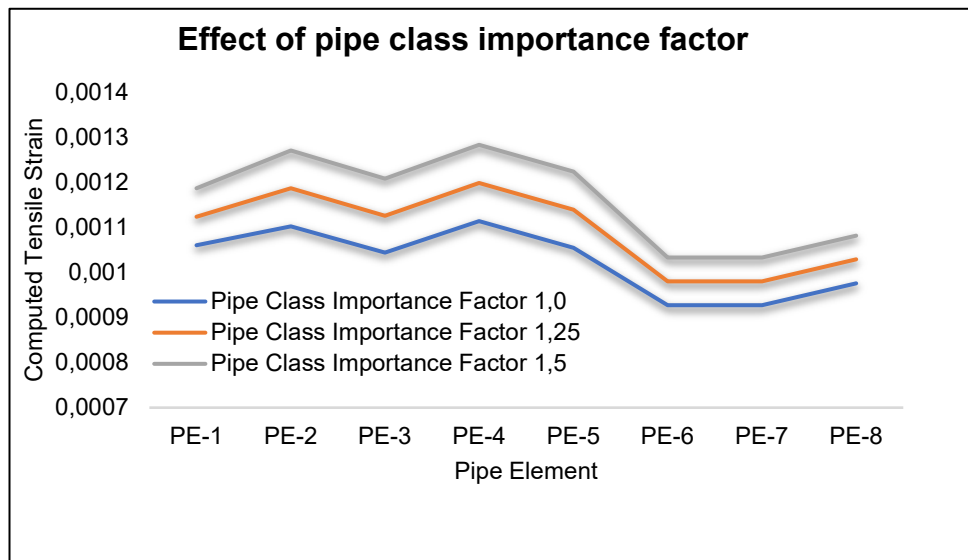


Figure 4.13: Effect of pipe class importance factor.

### 4.8.3. Variation of Internal Gas Pressure

For oil, gas and other fluid transporting pipes, internal pressure is classified in many seismic design codes [JSCE, 2000b], [ALA, 2001-2005], [EU-CODE, 2006], [IIT Kanpur, 2007], [ASME, 2012]. The effect of the variation of internal gas pressure from 0 MPa to 10 MPa is evaluated for the current case study. It is noted in Figure 4.14. that as the gas pressure is increased to higher levels, it causes instability in buried continuous pipelines.

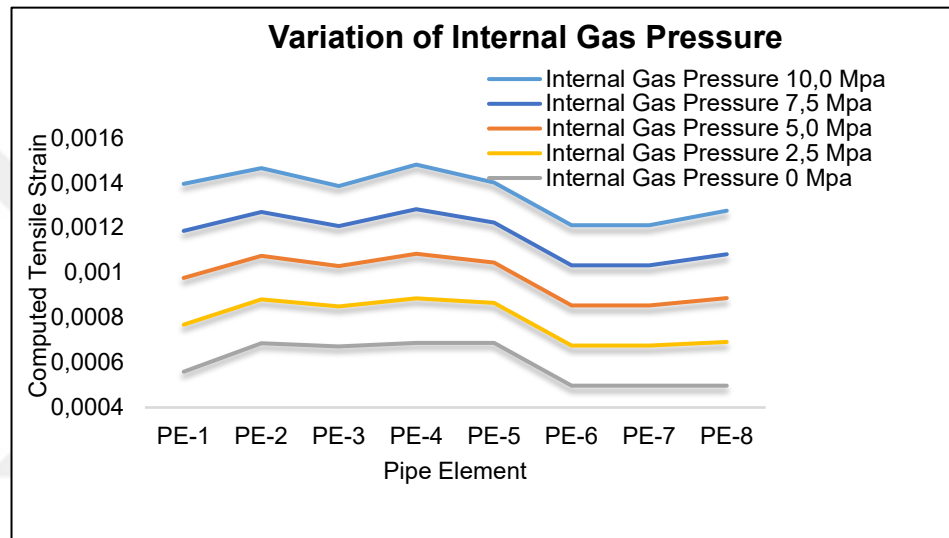


Figure 4.14: Variation of internal gas pressure.

### 4.8.4. Soil Class Effects

Different soil classes as given in NEHRP [NEHRP, 2004] are taken into consideration for current study. It is examined that seismic strain induced in the buried continuous pipelines is less for very dense soil (C-class) and increases with soil class levels (D-E Classes).

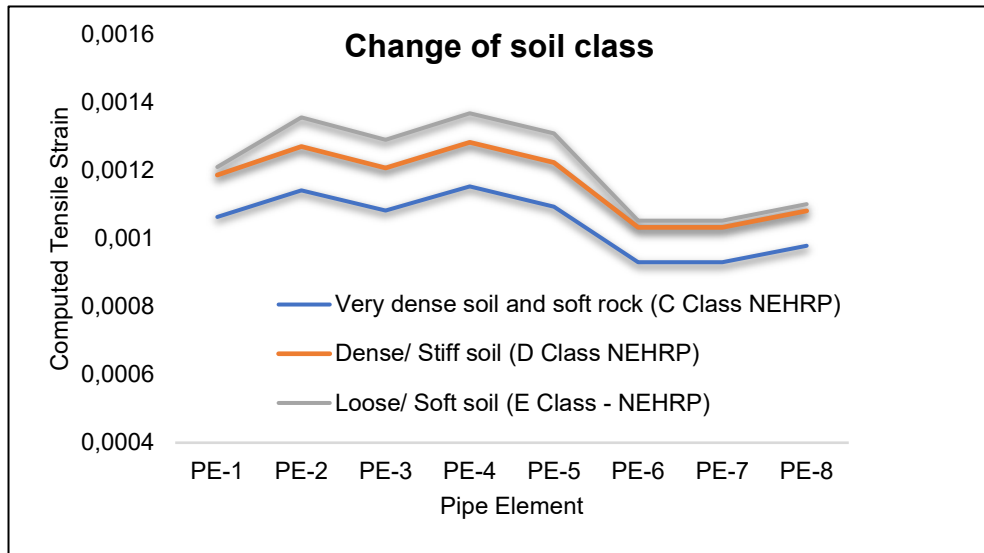


Figure 4.15: Soil class effects.

## 5. PERMANENT GROUND DEFORMATION (PGD) HAZARD

PGD is one of the prominent failures and type of soil deformation that cannot be recovered back, and it usually happens due to landslide, liquefaction induced lateral spreading, or faulting crossing of buried gas pipelines.

### 5.1. Past Damages Due to PGD

PGD hazard on buried pipelines in earlier foremost earthquakes are presented in Table 5.1.

Table 5.1: PGD hazard on buried pipelines in earlier foremost earthquakes.

| Location        | Earthquake<br>(Year – $M_w$ ) | Hazard Details                                                                                |
|-----------------|-------------------------------|-----------------------------------------------------------------------------------------------|
| San Francisco   | 1906 - 7.8                    | Soil liquefaction and lateral spreading caused the damage of buried water pipelines.          |
| Long Beach      | 1933 - 6.4                    | Nearly 50 breaks at welded section of buried gas high pressure pipelines.                     |
| Kern County     | 1952 - 7.3                    | Buried oil pipe ruptured due to fault and buried gas pipeline was deformed at fault crossing  |
| Alaska          | 1964 - 9.2                    | Due to land sliding, 200 breaks in buried gas and 100 breaks in water pipelines were noticed. |
| Niigata         | 1964 - 7.6                    | Failure at welded sections of buried pipes.                                                   |
| Haicheng        | 1975 - 7.3                    | Liquefaction caused immense damage to buried pipelines.                                       |
| Tangshan        | 1976 - 7.8                    | Due to surface fault at Tangshan city, all buried pipes crossing fault were totally damaged.  |
| Miyagi Ken-Oki  | 1978 - 7.7                    | 237 small diameter buried pipes of different kinds were damaged.                              |
| Imperial Valley | 1979 - 6.4                    | 3 buried gas pipelines and 2 high pressure oil pipes were damaged at fault crossing.          |
| Nihonkai-Chubu  | 1983 - 7.8                    | Liquefaction effected buried utilities.                                                       |

Table 5.1: (Continue) PGD hazard on buried pipelines in earlier foremost earthquakes.

| Location             | Earthquake<br>(Year – Mw) | Hazard Details                                                                          |
|----------------------|---------------------------|-----------------------------------------------------------------------------------------|
| Edgecumbe            | 1987 - 6.5                | Damages were reported to sewage pipelines.                                              |
| Loma Prieta          | 1989 - 6.9                | Large failures were observed in water and gas pipelines due to liquefaction.            |
| Landers and Big Bear | 1992 - 6.5                | At rupture area, water pipes were damaged.                                              |
| Northridge           | 1994 - 6.7                | Pipes laid at inclined surfaces were damaged.                                           |
| Kobe                 | 1995 - 6.9                | Huge quantity of BCGPs and water pipes were damaged due to ground failure.              |
| Chi-Chi              | 1999 - 7.6                | PGD due to land sliding, liquefaction and settlement cause the failure of buried pipes. |
| Izmit                | 1999 - 7.6                | Water pipe was reported as ruptured at different points of traversing.                  |
| Bhuj                 | 2001 - 7.7                | Liquefaction and lateral spreading caused the failure of different pipelines.           |
| Denali               | 2002 - 7.9                | Small fractures were noticed on main pipeline.                                          |
| Cheutsu              | 2004 - 6.8                | Land slide and liquefaction caused the displacement of buried pipes.                    |
| Peru                 | 2007 - 8.0                | Land sliding due to liquefaction caused damage to more than 100 buried water utilities. |
| Christchurch         | 2011 - 6.3                | Water pipes failed due to liquefaction and lateral spreading.                           |
| Central Italy        | 2016 - 6.2                | Land sliding in seismic active areas caused pipelines failure.                          |
| Alaska               | 2018 – 7.2                | 800-mile trans-Alaska pipeline gets affected due to land sliding during earthquake.     |
| North Central Peru   | 2019 – 8.0                | Landslide caused failure of buried oil pipes.                                           |

From literature studies of Table 4.1 and 5.1, It has been seen that PGD hazard to buried pipes is more than TWP hazard. The comparison of seismic hazards to buried pipes in previous major earthquakes have been presented in Figure 5.1.

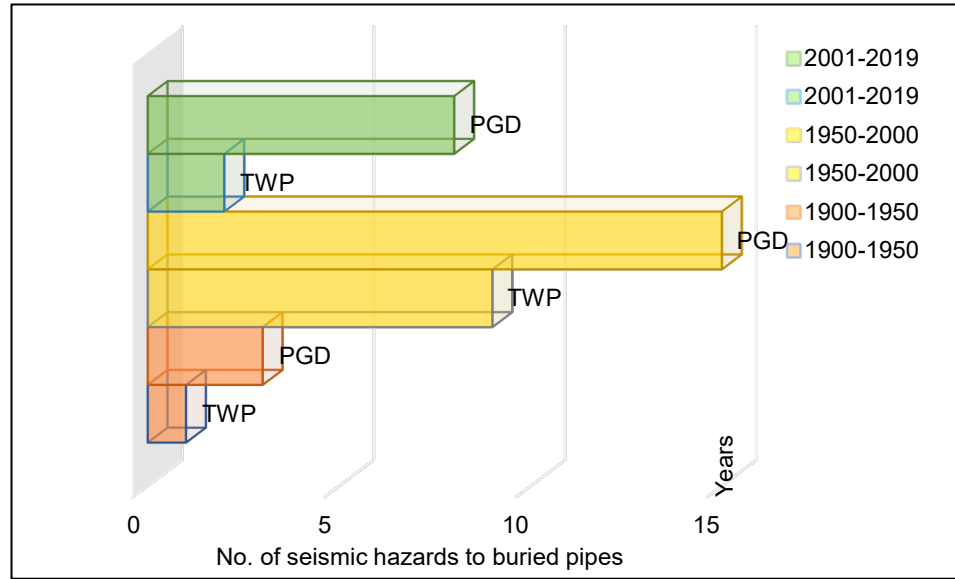


Figure 5.1: Comparison of seismic hazards to buried pipes in previous major earthquakes.

## 5.2. Failure Modes of BCGPs

Failure modes classification in BCGPs is the rudimentary step to establish suitable performance measures and assortment of effective analysis methodology. Excessive field survey reports, research publications and technical notes are available that categorize the failure modes of BCGPs. Generally, six categories of damage states for BCGPs have been observed in recent earthquakes, shown in Figure 5.2.

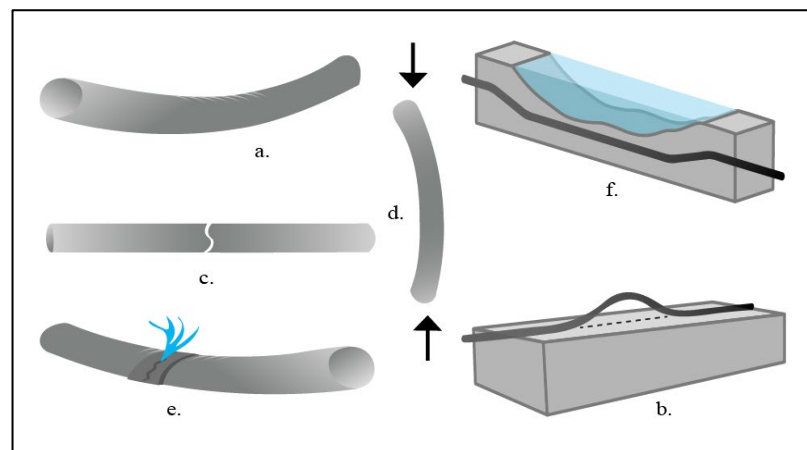


Figure 5.2: Failure modes of BCGPs a). local buckling b). beam buckling form c). tensile fracture type d). flexural failure mode e). welding failure f). fatigue failure condition.

### 5.2.1. Local buckling

Failure mode of BCGPs allied with stability loss due to compressive load or bending moment is known as shell type or local buckling failure. Parameters, such as big diameter, more R/t ratio and more deep covers are the main reasons of local buckling failure [Yun and Kyriakides, 1990]. This is a most often failure mode observed in recent earthquakes. Examples are summarized in Table 5.3. Hall and Newmark [Hall and Newmark, 1977] based on their experimental workings have established a strain criterion for local buckling as in equation (5.1).

$$0.15 \left( \frac{t}{R} \right) \leq \epsilon_{cr} \leq 0.20 \left( \frac{t}{R} \right) \quad (5.1)$$

In equation 5.1, t is the pipe wall thickness, R is the radius of pipe and  $\epsilon_{cr}$  is the critical buckling strain. Further, [O'Rourke and Liu, 1999] commented on Equation 5.1. and explained that it is applicable only to thin-walled cylinders. Recently, Vazouras et al. [Vazouras and Karamanos, 2010] developed a “no local buckling” condition for BCPs crossing strike-slip faults as given in equation (5.2).

$$\left( \frac{R}{t} \right) \leq 0.05a \left( \frac{L}{R} \right)^2 \quad (5.2)$$

In equation (5.2), R, t, L and a are radius, thickness, length of deformed segment and parameter of pipe material and initial imperfection of BCPs, respectively.

### 5.2.2. Beam buckling

Compressive loads of ground on pipelines buried at shallow depths can cause beam mode buckling or upheaval buckling. Parameters such as, small diameter, low R/t ratio and shallow depths are the main reasons for beam buckling [Yun and Kyriakides, 1990]. Meyerson and O'Rourke [Meyerson and O'Rourke, 1991] pointed out that BCPs covered by backfill soil of partial uplift resistance may fail due to beam mode buckling. Further, they explained that if the burial depth of BCPs is more than their critical cover depth, it is more likely to be a local buckling that can occur before

beam mode failure. Mitsuya et al. [Mitsuya et al., 2013] developed a critical strain formula based on elastic stability and tangent modulus theories and is given as (5.3).

$$\varepsilon_{cr} = \left( \frac{4K_L I}{A^2} \cdot \frac{\varepsilon_{0.2}^n}{\sigma_{0.2}} n \right)^{\frac{1}{(n+1)}} \quad (5.3)$$

In equation (5.3), A, K<sub>L</sub>, I and n symbolize cross sectional area, the lateral spring constant, second moment of inertia and work hardening exponent.  $\varepsilon_{0.2}$  is the corresponding strain value for 0.2% steel proof stress.

### 5.2.3. Tensile fracture

This form of failure mode is commonly observed in old steel pipes with weak gas welded joints due to accumulation of axial tension that exceeds the limiting threshold values. O'Rourke T.D and O'Rourke M.J [Rourke T.O. and Rourke M.O, 1995] described a case history of 1994 Northridge event based on joint differences. They observed that four side arc welded steel pipes with butt joints didn't fail when subjected to longitudinal PGD. But three gas welded pipes with slip joints faced tensile fracture at the same hazard condition. To find the tensile strains induced in a BCPs, Ramberg- Osgood's [ Ramberg and Osgood, 1943] equations are usually preferred.

$$\varepsilon = \frac{\sigma}{E} \left( 1 + \frac{n}{1+r} \cdot \left( \frac{\sigma}{\sigma_y} \right)^r \right) \quad (5.4)$$

In equation 5.4,  $\sigma$ , E,  $\sigma_y$  signify uniaxial tensile stress, initial young modulus, and apparent yield stress. Where  $n_i$  and  $r_i$ , are Ramberg and Osgood parameters given in Table 5.2.

Table 5.2: Values of Ramberg-Osgood's parameters.

| Steel grade  | B-Grade | X-42  | X-52  | X-60  | X-70  |
|--------------|---------|-------|-------|-------|-------|
| Yield stress | 227.0   | 310.0 | 358.0 | 413.0 | 517.0 |
| $n_i$        | 10.0    | 15.0  | 9.0   | 10.0  | 5.50  |
| $r_i$        | 100.0   | 32.0  | 10.0  | 12.0  | 16.60 |

#### 5.2.4. Flexural Failure

Failure associated with excessive flexural stresses, bending strain or compressive load is termed as flexural failure. As BCPs are of high ductile in nature, so there is an infrequent chance of such failure to happen.

#### 5.2.5. Welding Failure

Failure at connections of BCPs due to improper weld or inexperience personnel workings may lead to welding failure (WF). This type of failure is not so common in BCPs due to modern welding techniques and laboratory weld strength testing before laying but it has been observed in old BCPs due to unfitted welds.

#### 5.2.6. Fatigue Failure

Repetitive loading during earthquake may cause fatigue failure in BCPs. Pipelines were failed due to accumulation of tensile and compressive forces during 1983 Nihonkai-Chubu event [Kitaura and Miyajima 1988]. Failure modes with their number of failures based on years have been shown in Figure 5.3.

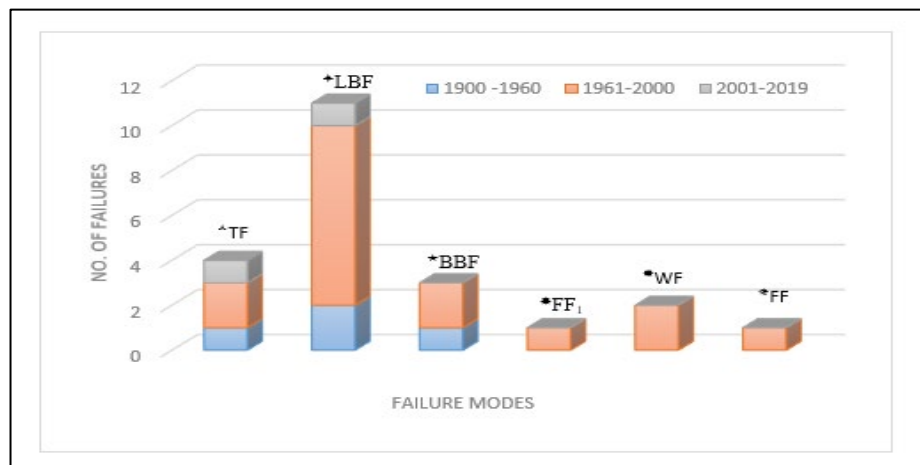


Figure 5.3: Failure modes with their number of failures based on years.

Failure modes observed in recent major earthquake events by various researchers are summarized in Table 5.3. Local buckling type failure mode is the prominent one in previous major earthquakes enlisted in Table 5.3.

Table 5.3: Values of Ramberg-Osgood's parameters.

| Location        | Earthquake<br>(Year – $M_w$ ) | Failure Modes in BCPs |                  |                  |                              |                 |                 |
|-----------------|-------------------------------|-----------------------|------------------|------------------|------------------------------|-----------------|-----------------|
|                 |                               | <sup>^</sup> TF       | <sup>+</sup> LBF | <sup>*</sup> BBF | <sup>*</sup> FF <sub>1</sub> | <sup>*</sup> WF | <sup>*</sup> FF |
| San Franc. EQ   | 1906 – 7.8                    | •                     | •                | •                |                              |                 |                 |
| Kern Count. EQ  | 1952 – 7.3                    |                       | •                |                  |                              |                 |                 |
| Alaska EQ       | 1964 – 9.2                    |                       | •                |                  |                              |                 |                 |
| San Fern. EQ    | 1971 – 6.7                    |                       | •                | •                | •                            | •               |                 |
| Tangshan        | 1976 – 7.8                    |                       | •                |                  |                              |                 |                 |
| Imperial V. EQ  | 1979 – 6.4                    |                       |                  | •                |                              |                 |                 |
| Nihonkai-Chubu  | 1983 – 7.8                    |                       | •                |                  |                              |                 | •               |
| Michoacán       | 1985 – 8.0                    |                       | •                |                  |                              |                 |                 |
| Ecuador         | 1987 – 7.1                    | •                     |                  |                  |                              |                 |                 |
| Costa Rica. EQ  | 1991 – 7.7                    |                       | •                |                  |                              |                 |                 |
| Northridge. EQ  | 1994 – 6.7                    | •                     | •                |                  |                              | •               |                 |
| Chi.Chi EQ      | 1999 – 7.6                    |                       | •                |                  |                              |                 |                 |
| Central Italian | 2006 – 6.2                    | •                     |                  |                  |                              |                 |                 |
| Peru            | 2019 – 8.0                    |                       | •                |                  |                              |                 |                 |

<sup>^</sup>Tensile fracture, <sup>+</sup>Local buckling failure, <sup>\*</sup>Beam buckling failure, <sup>\*</sup>Flexural failure,  
<sup>\*</sup>Welding failure, <sup>\*</sup>Fatigue failure

### 5.3. Seismic Safety Assessment of BCGPs: Current Case Study

In this section, seismic safety assessment is completed against PGD hazard for current case study. The PGD hazards involved in current study are lateral spreading (longitudinal PGD and transverse PGD), buoyancy due to liquefaction and strike-slip fault crossing.

### 5.3.1. PGD due to Lateral Spreading

Lateral spreading can be categorized as longitudinal PGD and transverse PGD. The developed algorithm can be applied either on the critical parts of the buried gas pipe (where the pipe is assumed to face lateral spreading) or on the whole network of the system. The recommendation is to apply only on the critical parts by pre-survey of the site. Three images are shown in Figure 5.4. that describes the case of pipe crossing through landslide area. The pipeline may cross the landslide zone in any arbitrary direction (Figure.5.4a). However, designing the pipeline for critical response due to permanent ground deformation (PGD), two conditions such as parallel crossing (Figure 5.4b) and perpendicular crossing (Figure 5.4c) are considered. The developed algorithm considers these two situations as the pipeline is subjected to longitudinal PGD and transverse PGD respectively.

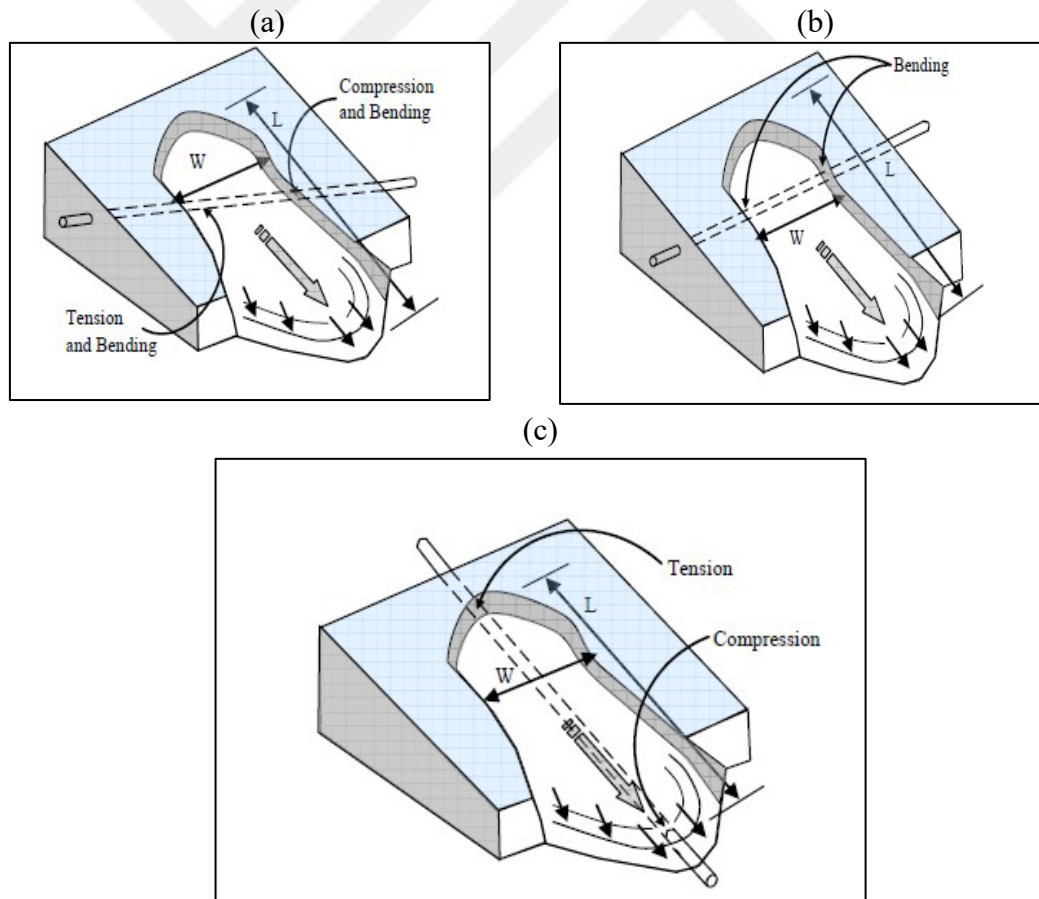


Figure 5.4: Modes of lateral spreading (a). arbitrary direction (b). perpendicular crossing (c). parallel crossing.

The methodology adopted for analysis is explained in Figure 5.5. Firstly, length of PGD zones and its corresponding longitudinal PGD is evaluated.

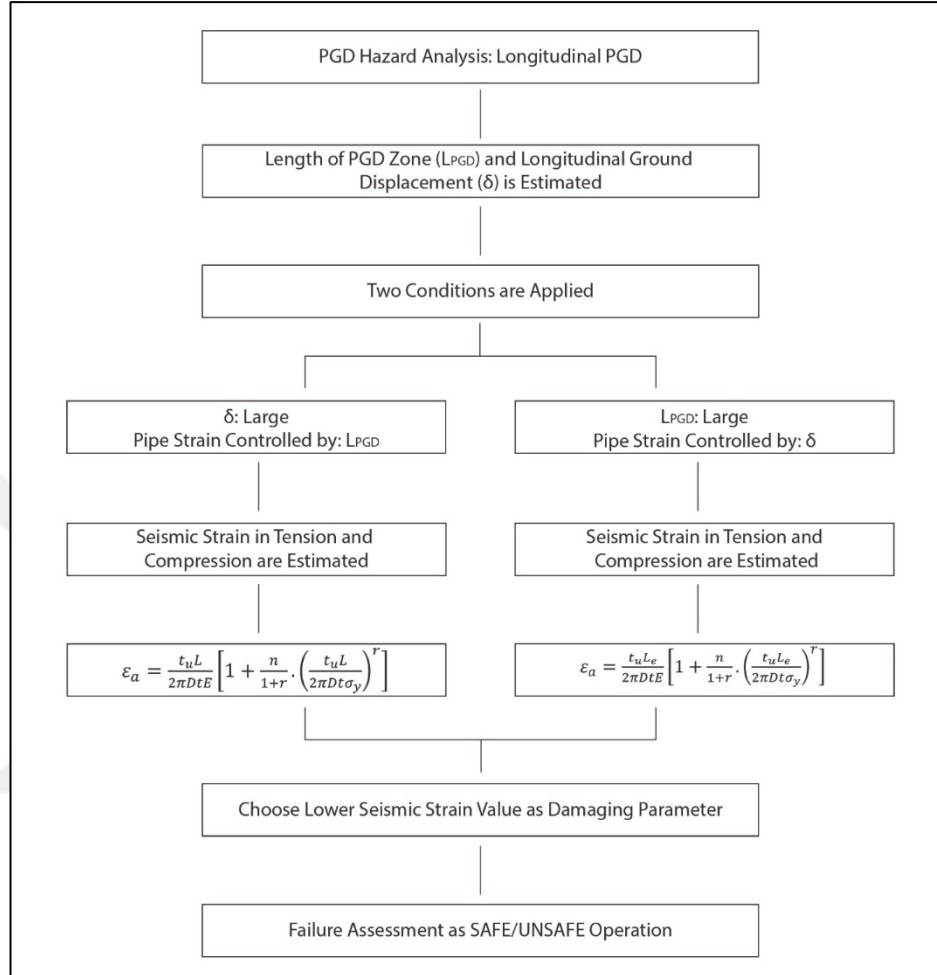


Figure 5.5: Flow chart for PGD hazard analysis due to longitudinal PGD.

To find design strain, a minimum value is taken between two cases explained in Figure 5.6. Two cases of inelastic model for longitudinal PGD are explained there. Firstly, it is assumed that the expected amount of permanent ground movement will be of 3m. Then the design ground movement can be evaluated from equation (5.5).

$$\delta^l_{design} = \delta^l \cdot I_p \quad (5.5)$$

Where, IP is the Importance Factor for different classes of pipeline. From Figure 5.4. maximum axial strain in a pipe can be evaluated from equation (5.6).

$$\varepsilon_a = \frac{t_u L}{2\pi D t E} \left[ 1 + \frac{n}{1+r} \cdot \left( \frac{t_u L}{2\pi D t \sigma_y} \right)^r \right] \quad (5.6)$$

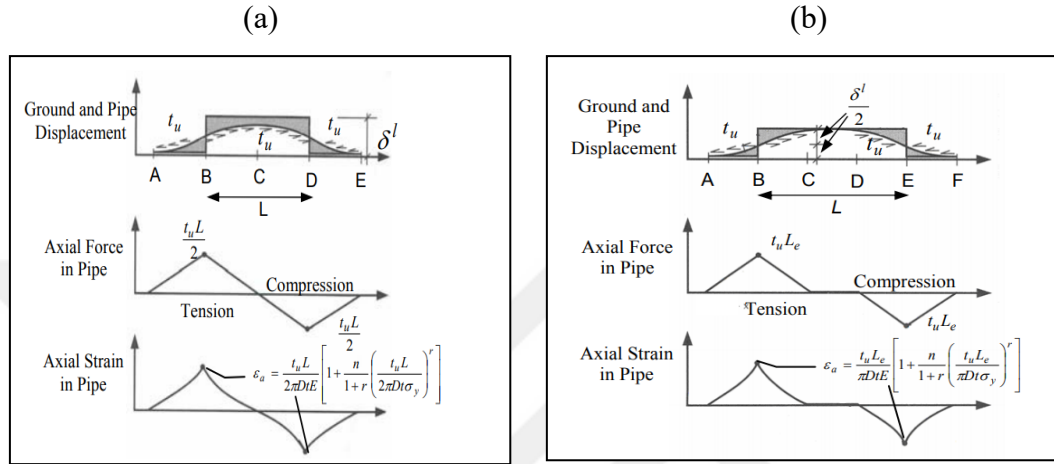


Figure 5.6: Inelastic model for longitudinal PGD. (a) Case-I, (b) Case-II.

Maximum seismic strain due to effective length “Le” can be calculated from equation (5.7).

$$\varepsilon_a = \frac{t_u L_e}{2\pi D t E} \left[ 1 + \frac{n}{1+r} \cdot \left( \frac{t_u L_e}{2\pi D t \sigma_y} \right)^r \right] \quad (5.7)$$

Le = Effective length of pipeline where the friction force (Tu) acts and is found by equation (5.8).

$$\delta^l_{design} = \frac{t_u L_e^2}{\pi D t E} \left[ 1 + \frac{2}{2+r} \cdot \frac{n}{1+r} \cdot \left( \frac{t_u L_e}{\pi D t \sigma_y} \right)^r \right] \quad (5.8)$$

The dimensions adopted for current study is shown in Figure 5.7. for lateral spreading evaluation due to longitudinal PGD.

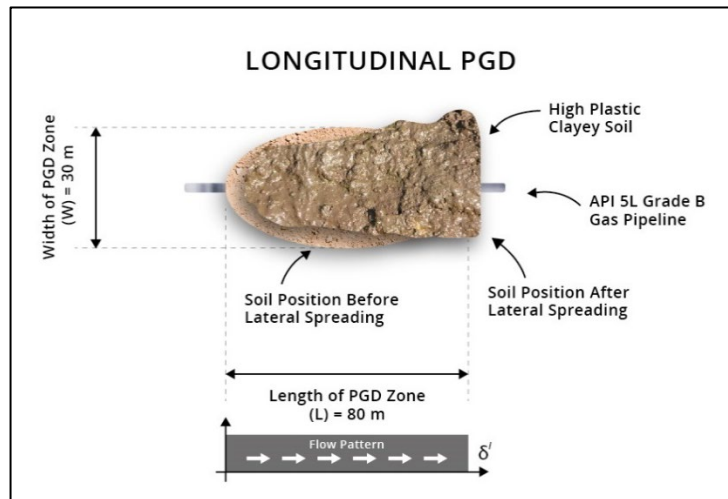


Figure 5.7: Longitudinal PGD: current case study.

Table 5.4. shows the seismic strains due to lateral spreading in longitudinal sides computed vs. allowable limits to check the status of buried gas pipes of current study for safety operations.

Table 5.4: Longitudinal PGD: current case study results.

| Pipe Element | $\epsilon_{t\text{-computed}}$ | $\epsilon_{t\text{-allowable}}$ | Status | $\epsilon_{c\text{-computed}}$ | $\epsilon_{c\text{-allowable}}$ | Status |
|--------------|--------------------------------|---------------------------------|--------|--------------------------------|---------------------------------|--------|
| PE-1         | 0,002402                       | 0,03                            | S      | 0,00085                        | 0,00373                         | S      |
| PE-2         | 0,001430                       | 0,03                            | S      | 0,00005                        | 0,00373                         | S      |
| PE-3         | 0,001754                       | 0,03                            | S      | 0,00028                        | 0,00373                         | S      |
| PE-4         | 0,002051                       | 0,03                            | S      | 0,00067                        | 0,00373                         | S      |
| PE-5         | 0,001764                       | 0,03                            | S      | 0,00027                        | 0,00373                         | S      |
| PE-6         | 0,000960                       | 0,03                            | S      | -0,00042                       | 0,00373                         | S      |
| PE-7         | 0,001708                       | 0,03                            | S      | 0,00033                        | 0,00373                         | S      |
| PE-8         | 0,001430                       | 0,03                            | S      | 0,00005                        | 0,00373                         | S      |

\*S: Safe

Following below assumptions are considered for evaluation of tensile strain in current study. Soil class: D (NEHRP) - Dense/ Stiff soil -  $180 < V_s \leq 360$ ; pipe class importance factor: 1,5; length of PGD Zone= $L= 80$  m; effective length =  $L_e= 64$  m;  $H$  = Depth of soil above the center of the pipeline (2,5 m); internal gas pressure =  $P_i= 7.5$  MPa, Installation temp.=  $25^{\circ}\text{C}$ , and Operating temp.=  $40^{\circ}\text{C}$ .

The methodology adopted for seismic strains analysis due to lateral spreading in transverse direction is explained in Figure 5.8.

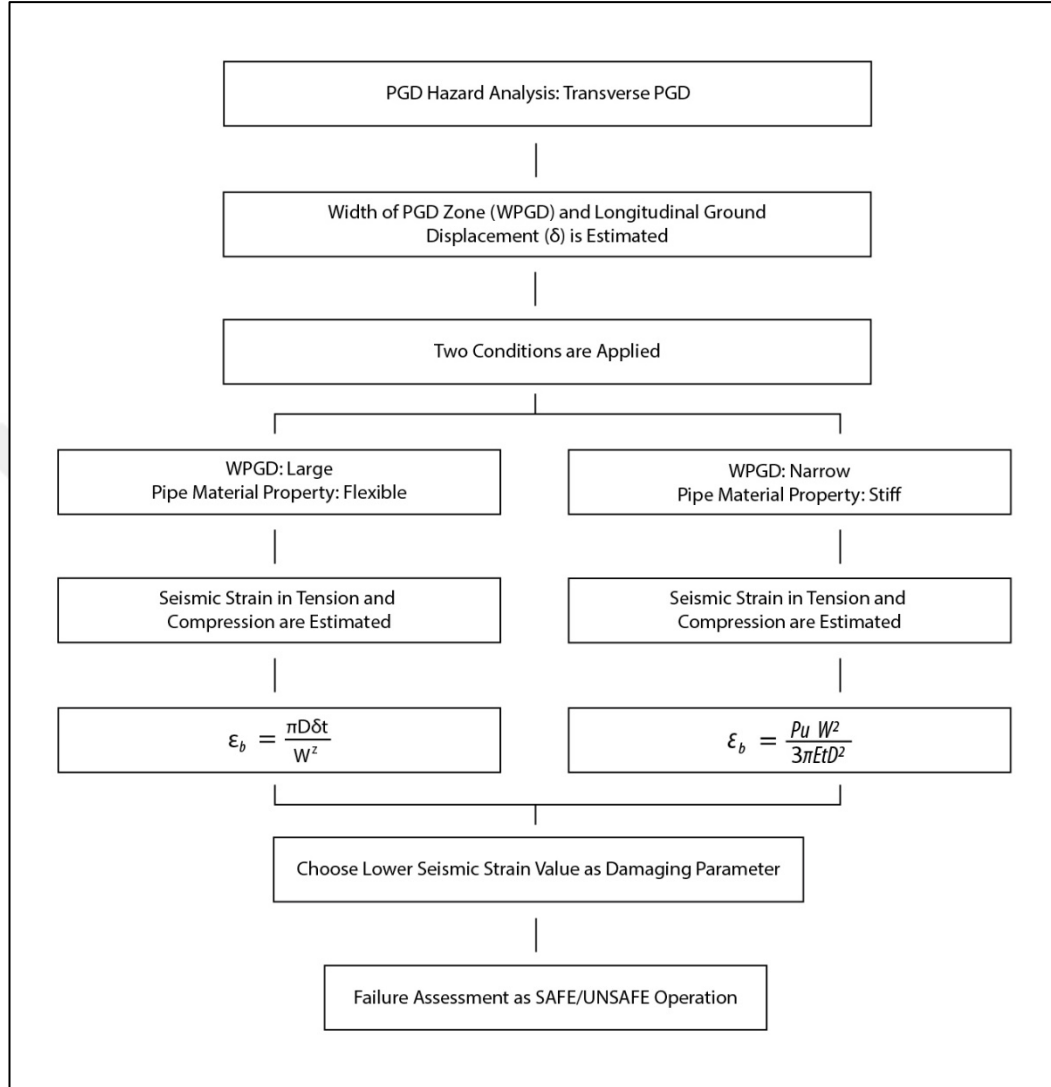


Figure 5.8: Flow chart for PGD hazard analysis due to transverse PGD.

The dimensions adopted for current study is shown in Figure 5.9. for lateral spreading evaluation due to longitudinal PGD.

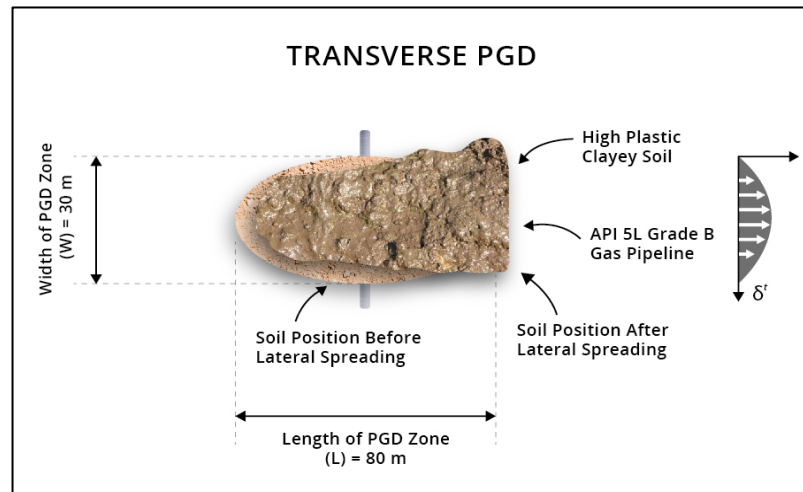


Figure 5.9: Transverse PGD: current case study.

Table 5.5. shows the seismic strains due to transverse PGD computed vs. allowable limits to check the status of buried gas pipes of current study for safety operations.

Table 5.5: Transverse PGD: current case study results.

| Pipe element | $\epsilon_{t\text{-computed}}$ | $\epsilon_{t\text{-allowable}}$ | Status | $\epsilon_{c\text{-computed}}$ | $\epsilon_{c\text{-allowable}}$ | Status |
|--------------|--------------------------------|---------------------------------|--------|--------------------------------|---------------------------------|--------|
| PE-1         | 0,000999                       | 0,03                            | S      | -0,00056                       | 0,00373                         | S      |
| PE-2         | 0,000762                       | 0,03                            | S      | -0,00062                       | 0,00373                         | S      |
| PE-3         | 0,000811                       | 0,03                            | S      | -0,00066                       | 0,00373                         | S      |
| PE-4         | 0,001017                       | 0,03                            | S      | -0,00036                       | 0,00373                         | S      |
| PE-5         | 0,001063                       | 0,03                            | S      | -0,00043                       | 0,00373                         | S      |
| PE-6         | 0,000864                       | 0,03                            | S      | -0,00052                       | 0,00373                         | S      |
| PE-7         | 0,000814                       | 0,03                            | S      | -0,00057                       | 0,00373                         | S      |
| PE-8         | 0,000780                       | 0,03                            | S      | -0,00060                       | 0,00373                         | S      |

\*S: Safe

Following below assumptions are considered for evaluation of tensile strain in current study. Soil class: D (NEHRP) - Dense/ Stiff soil -  $180 < V_s \leq 360$ ; pipe class importance factor: 1,5; width of PGD Zone= $W= 30$  m; Effective length =  $Le= 64$  m;

H = Buried depth of pipe from topsoil layer (2,5 m); internal gas pressure =  $P_i = 7,5$  Mpa, installation temp.=  $25^{\circ}\text{C}$ , and operating Temp.=  $40^{\circ}\text{C}$

### 5.3.1.1 Parametric Studies (Lateral Spreading)

From Figure 5.10; It is observed that the length of PGD zones is directly proportional to the failure of BCGPs. Displacement absorption capacity (DAC) of pipe should be checked in terms of exceed/decreased limit values. If the expected ground movement exceeds DAC value, then other alternative options should be adopted such as improvement of soil layer surrounding the pipe etc.

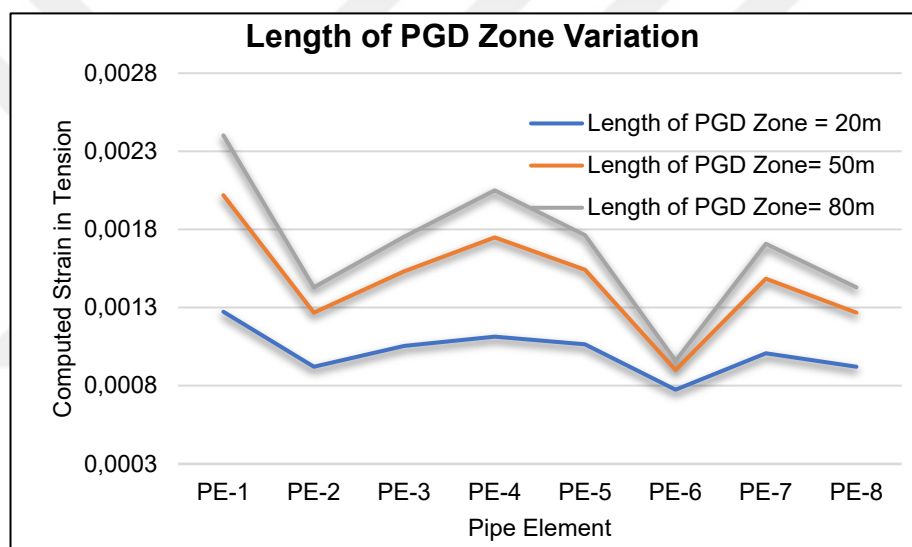


Figure 5.10: Length of PGD zone variation.

However, as observed from Figure 5.11; the width of PGD zone is inversely proportional to the accumulation of seismic strains. Therefore, to avoid this condition, the strength of soil layers surrounding the buried pipes should be improved to lessen the movement of lateral soil. If the pipe is buried at shallow depth of soil, then densification procedure of soil is more appropriate. However, if the burial depth is deeper, then stabilized soil buttresses technique can be utilized to strengthen the surrounding soil around the buried pipe.

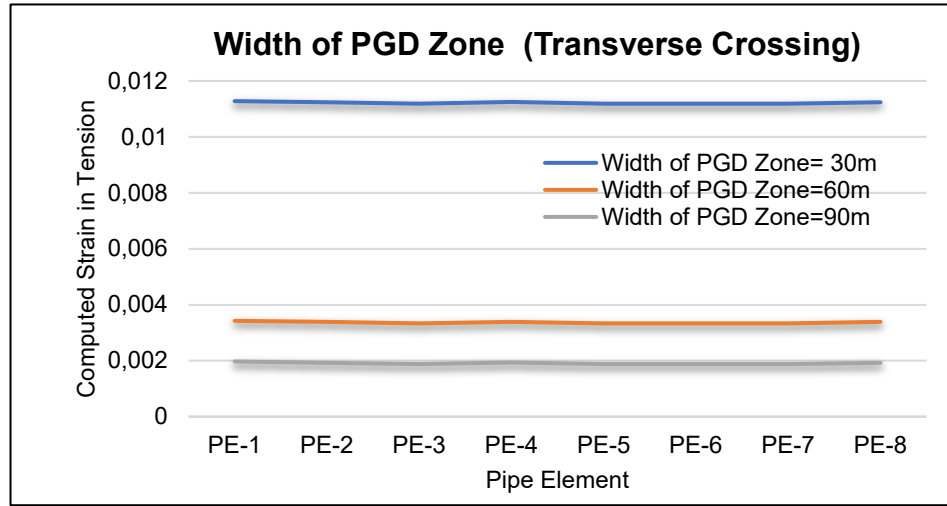


Figure 5.11: Width of PGD zone variation.

### 5.3.2. Buoyancy Due to Liquefaction

The extent of liquefaction zone in current study is expected as 30 m. Net upward force per unit length of BCGPs can be computed by using equation (5.9).

$$F_b = \left[ \frac{\pi D^2}{4} \cdot (\gamma_{sat} - \gamma_{con}) \right] - \pi D t \gamma_{pipe} \quad (5.9)$$

Bending stress in the BCGPs due to upward force is computed by using equation (5.10).

$$\sigma_{bf} = \frac{F_b \cdot L_b^2}{10Z} \quad (5.10)$$

Whereas Z in Equation 5.10. is the section modulus of pipe cross section that can be found by using equation (5.11).

$$Z = \frac{\pi(D^4 - d^4)}{32D} \quad (5.11)$$

The maximum strain value accumulated in the buried pipe due to the bending stress can be calculated as equation (5.12).

$$\varepsilon = \frac{\varepsilon_{bf}}{E} \left[ 1 + \frac{n}{1+r} \right] \left[ \frac{\sigma_{bf}}{\sigma_y} \right]^r \quad (5.12)$$

The methodology adopted for seismic strains analysis due to lateral spreading in transverse direction is explained in Figure 5.12.

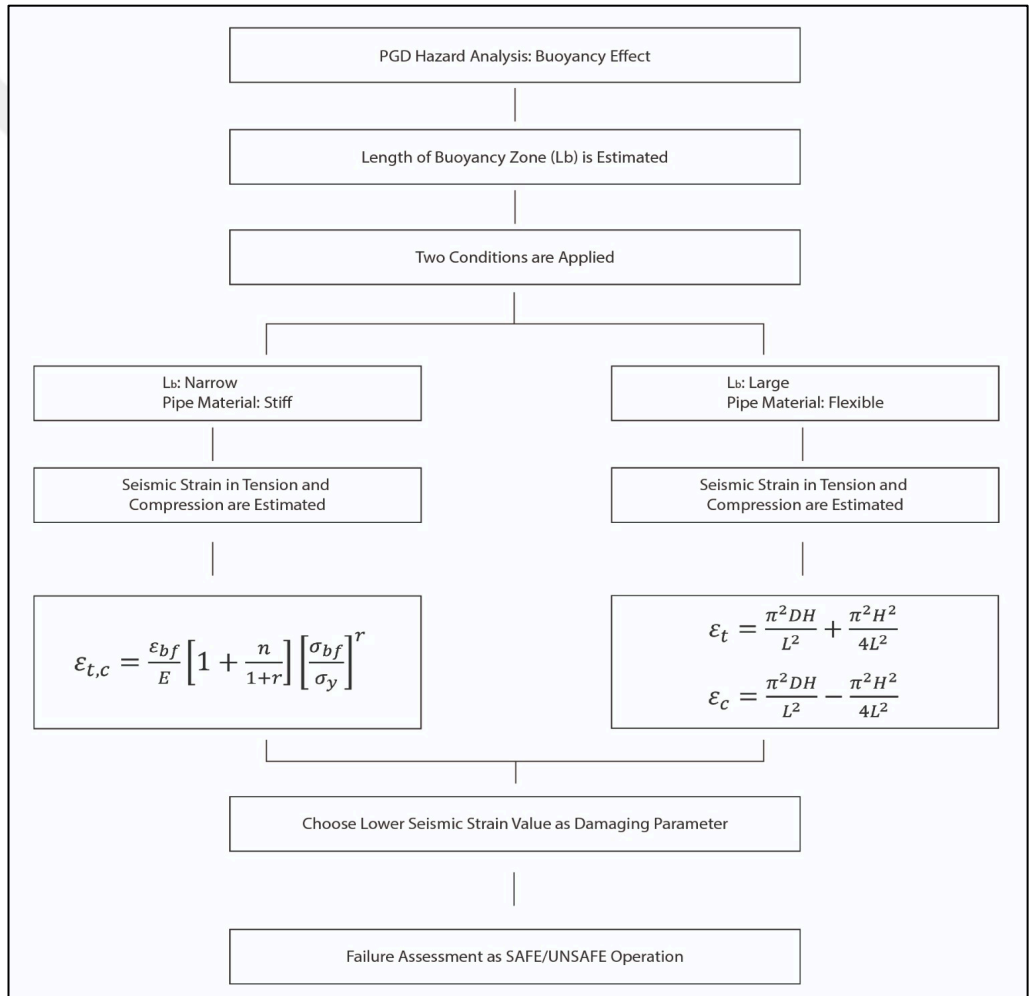


Figure 5.12: Flow chart for PGD hazard analysis due to buoyancy effect.

As the extent of liquefaction ( $L_b$ ) is of narrow value that is taken as 30m. Therefore, condition 1 is applied to check the seismic safety of BCGPs due to

buoyancy effect. Longitudinal section of the pipeline showing the forces acting on it due to buoyancy is presented in Figure 5.13.

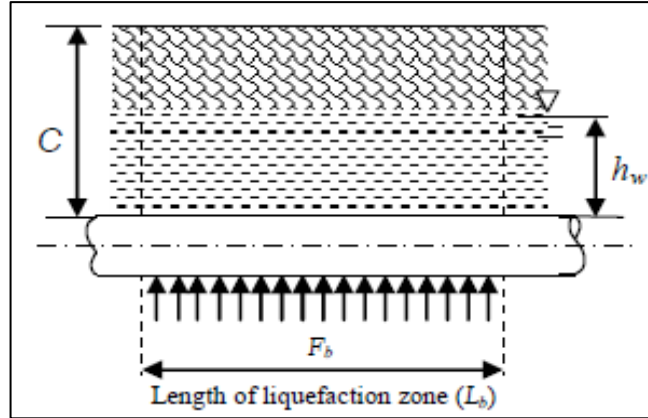


Figure 5.13: Longitudinal section of the pipeline showing the forces acting on it due to buoyancy.

The algorithm is applied, and the results are shown in Table 5.6. The computed tensile strain values are lower than their upper bound threshold limit values, thus all pipes behave safe under buoyancy effect related PGD condition. The computed tensile strain values are given in Table 5.6 and the condition used for analysis are presented in the next section to the Table 5.6.

Table 5.6: Buoyancy due to Liquefaction: current case study results.

| Pipe element | $\epsilon_{t\text{-computed}}$ | $\epsilon_{t\text{-allowable}}$ | Status | $\epsilon_{c\text{-computed}}$ | $\epsilon_{c\text{-allowable}}$ | Status |
|--------------|--------------------------------|---------------------------------|--------|--------------------------------|---------------------------------|--------|
| PE-1         | 0,001538                       | 0,03                            | S      | -0,00002                       | 0,003139                        | S      |
| PE-2         | 0,001041                       | 0,03                            | S      | -0,00034                       | 0,003368                        | S      |
| PE-3         | 0,001211                       | 0,03                            | S      | -0,00026                       | 0,003675                        | S      |
| PE-4         | 0,001335                       | 0,03                            | S      | -0,00005                       | 0,003307                        | S      |
| PE-5         | 0,001221                       | 0,03                            | S      | -0,00027                       | 0,003675                        | S      |
| PE-6         | 0,001164                       | 0,03                            | S      | -0,00022                       | 0,003675                        | S      |
| PE-7         | 0,001164                       | 0,03                            | S      | -0,00022                       | 0,003675                        | S      |
| PE-8         | 0,001041                       | 0,03                            | S      | -0,00034                       | 0,003368                        | S      |

\*S: Safe

Following below assumptions are considered for evaluation of tensile strain in current study. Soil class: D (NEHRP) - Dense/ Stiff soil -  $180 < V_s \leq 360$ ; pipe class importance factor: 1,5; length of pipeline in Buoyancy Zone= $L_b=20\text{m}$ ; unit weight of buried pipe ( $\gamma_{\text{pipe}}$ ) is considered as  $78560 \text{ N/m}^3$ ;  $H$  = Burial depth of pipe from the top layer of soil (2,5 m); internal gas pressure =  $P_i=7,5 \text{ Mpa}$ , installation temp.=  $25^\circ\text{C}$ , and operating temp.=  $40^\circ\text{C}$ .

### 5.3.2.1. Parametric Studies (Liquefaction)

In section 5.3.2; It is noted that the selected pipe for current study behaves as safe operation as computed strain value is less than the threshold limit values. Further, the liquefaction study is expanded by considering some parameters change and their effect on the pipe behavior. In Figure 5.14; as the extent of liquefaction is increased, the strain accumulated in the buried pipe also increased. PE-1 segment at  $L_b$  value of 30m shows a sharp increment in strain that display the earlier failure mode of the pipe segment.

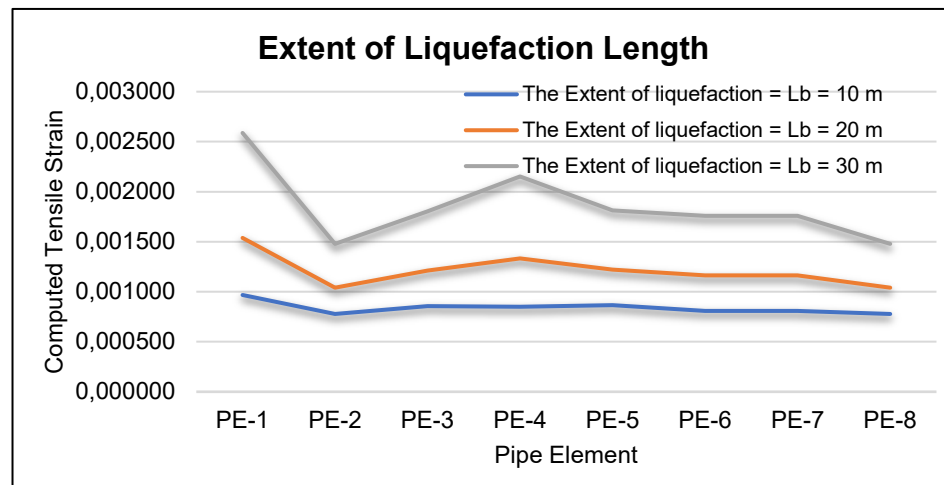


Figure 5.14: Extent of liquefaction length.

From literature studies, the below points are collected to minimize the effect of buoyancy on the buried pipes to avoid them from failures. Researchers suggest encasing the buried pipe with concrete or termed as concrete pipes to lessen the

buoyancy effect. Extra consolidation in the form of concrete weight or soil blankets in the form of gravel fills can be used to make an extra resistance to the buoyancy. It is noted from past studies that buoyancy effect can also be decreased by placing pipe at shallower depths or placing above the water table. If the uplifting phenomena of buried pipe is the main concern, then anchors in the form of tie or hooks can be applied.

### 5.3.3. Due to Fault Crossing

From previous earthquakes, it is observed that BCGPs crossing seismic faults gain higher amount of tensile strain that results in the form of failure when a strong seismic event happens. There are different kind of seismic faults based on the geometry of their plate's movement. Usually for BCGPs design against seismic faults, three kinds of faults are considered namely strike-slip, normal and reverse faults. For seismic analysis of BCGPs against these fault crossings, various empirical based approaches are available. For current study, as the BCGPs are close to the North Anatolian Fault (NAF), therefore strike-slip fault is considered for the analysis. The graphical representation of pipeline crossing strike-slip fault is shown in Figure 5.15.

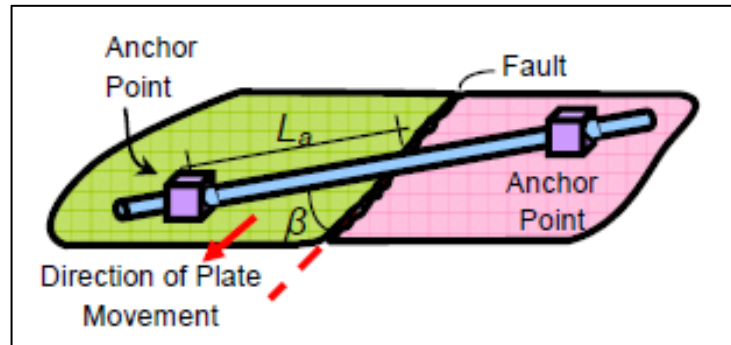


Figure 5.15: Pipeline crossing strike slip fault.

For strike-slip fault crossing, two well-known approaches developed by Newmark and Hall and Kennedy et al., are usually preferred. The reason behind to use these approaches is due to their compatibility with the selected site and precision of results. For current analysis, Newmark, and Hall, 1975 approach is used for the analysis and this approach is described in the form of flow chart in Figure 5.16.

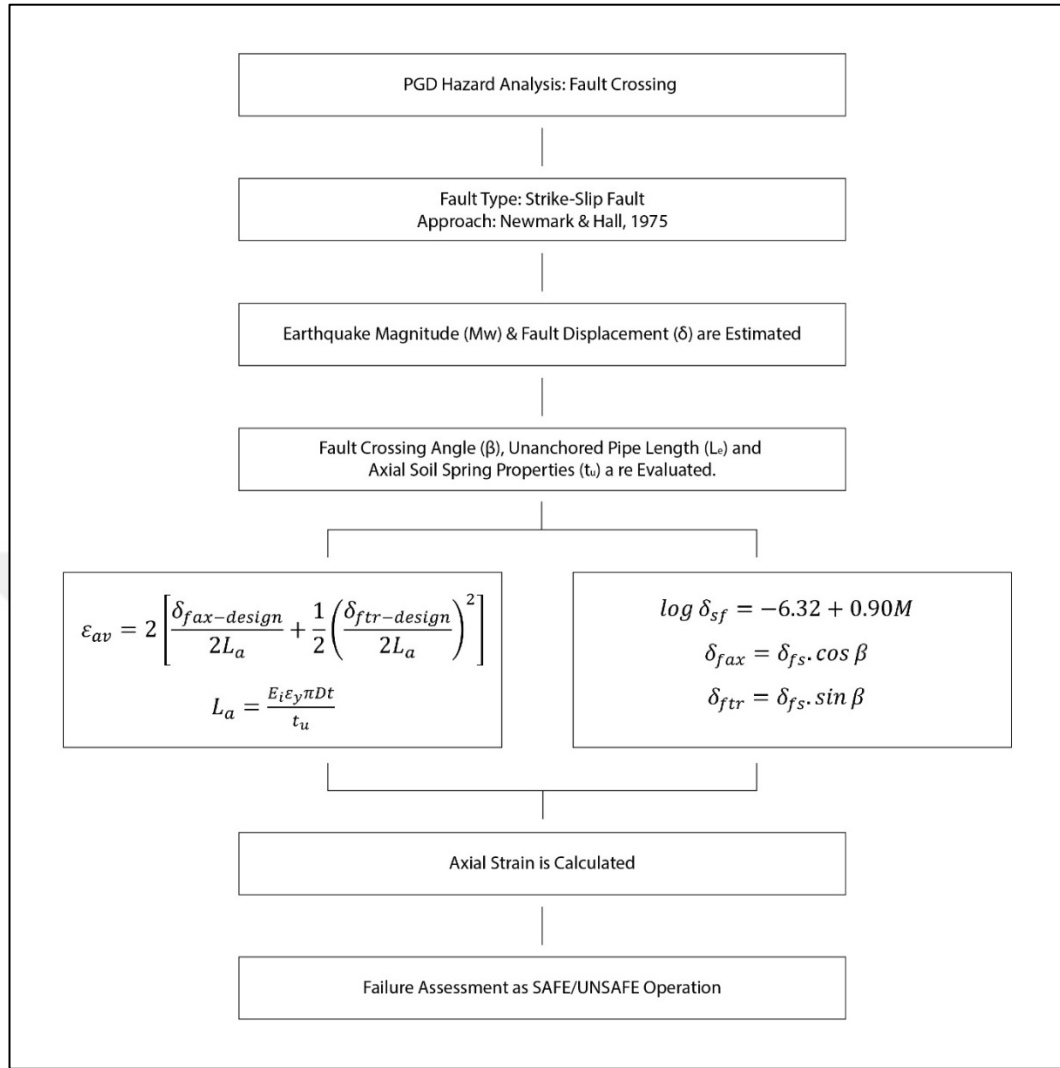


Figure 5.16: Flowchart for Strike-Slip fault crossing by Newmark & Hall approach.

Firstly, expected fault displacement is essential to evaluate based on rigorous empirical based analysis. Many researchers have contributed well to this domain. In current study, widely used empirical equations developed by [Copper and Smith, 1994] are preferred and are given below. For strike slip fault displacement equation (5.13) is used.

$$\log \delta_{sf} = -6.32 + 0.90M \quad (5.13)$$

Then, component of fault displacement in both axial and transverse directions are evaluated. Strike slip fault displacement in axial and transverse sides of BGPs are given in equations (5.14) and (5.15).

$$\delta_{fax} = \delta_{fs} \cdot \cos \beta \quad (5.14)$$

$$\delta_{ftr} = \delta_{fs} \cdot \sin \beta \quad (5.15)$$

Design fault displacement in axial and transverse sides of BGPs are computed by equations (5.16) and (5.17).

$$\delta_{fax-design} = \delta_{fax} \cdot I_p \quad (5.16)$$

$$\delta_{ftr-design} = \delta_{ftr} \cdot I_p \quad (5.17)$$

The average pipe strain due to fault movement in axial direction is evaluated as equation (5.18).

$$\varepsilon_{av} = 2 \left[ \frac{\delta_{fax-design}}{2L_a} + \frac{1}{2} \left( \frac{\delta_{ftr-design}}{2L_a} \right)^2 \right] \quad (5.18)$$

$L_a$  = Effective unanchored length of the pipeline in the fault zone and is computed as in equation (5.19).

$$L_a = \frac{E_t \varepsilon_y \pi D t}{t_u} \quad (5.19)$$

The output of current study related to strike-slip fault crossing is given in Table 5.7.

Table 5.7: Seismic safety evaluation due to fault crossing.

| Pipe element | $\epsilon_{t\text{-computed}}$ | $\epsilon_{t\text{-allowable}}$ | Status | $\epsilon_{c\text{-computed}}$ | $\epsilon_{c\text{-allowable}}$ | Status |
|--------------|--------------------------------|---------------------------------|--------|--------------------------------|---------------------------------|--------|
| PE-1         | 0,042021                       | 0,03                            | US     | 0,04041                        | 0,00373                         | US     |
| PE-2         | 0,019937                       | 0,03                            | S      | 0,01841                        | 0,00373                         | US     |
| PE-3         | 0,027092                       | 0,03                            | S      | 0,02566                        | 0,00373                         | US     |
| PE-4         | 0,035964                       | 0,03                            | US     | 0,03441                        | 0,00373                         | US     |
| PE-5         | 0,027092                       | 0,03                            | S      | 0,02566                        | 0,00373                         | US     |
| PE-6         | 0,004744                       | 0,03                            | S      | 0,00331                        | 0,00373                         | US     |
| PE-7         | 0,027092                       | 0,03                            | S      | 0,02566                        | 0,00373                         | US     |
| PE-8         | 0,019937                       | 0,03                            | S      | 0,01841                        | 0,00373                         | US     |

\*S: Safe; US: Unsafe

From Table, 5.7, PE-1 and PE-4 behave as unsafe condition due to the over increments of tensile strain in them than their threshold limit values. Also, all elements of pipe behave as unsafe in compression side when supposed to strike-slip fault crossing, so special design precautions should be considered for BCGPs in compression side. Following assumptions are considered for evaluation of tensile strain in current study. Soil class: D (NEHRP) - Dense/ Stiff soil -  $180 < V_s \leq 360$ ; Kuzey Anadolu Fay = North Anatolian Fault = Strike - Slip Fault is considered. Earthquake Magnitude= EQ = 7,5; Dip angle =  $\psi = 45^\circ$ ; The angle b/w buried pipe and fault line =  $\beta = 30^\circ$ ; H = Depth of soil above the center of the pipeline (2,5 m); Internal gas pressure =  $P_i = 7,5$  Mpa, Installation temp.=  $25^\circ\text{C}$ , and operating temp.=  $40^\circ\text{C}$

### 5.3.3.1. Parametric Studies (Fault Crossing)

Firstly, the effect of change in earthquake magnitudes are noted. It is observed that as the magnitude of earthquake is increased the strain value is directly increased as shown in Figure 5.17. Further, a peak increase is observed as the magnitude is increased from 6.5 to 7.0. However, there is slight increment between magnitudes of 6.0 to 6.5 in comparison to the higher levels.

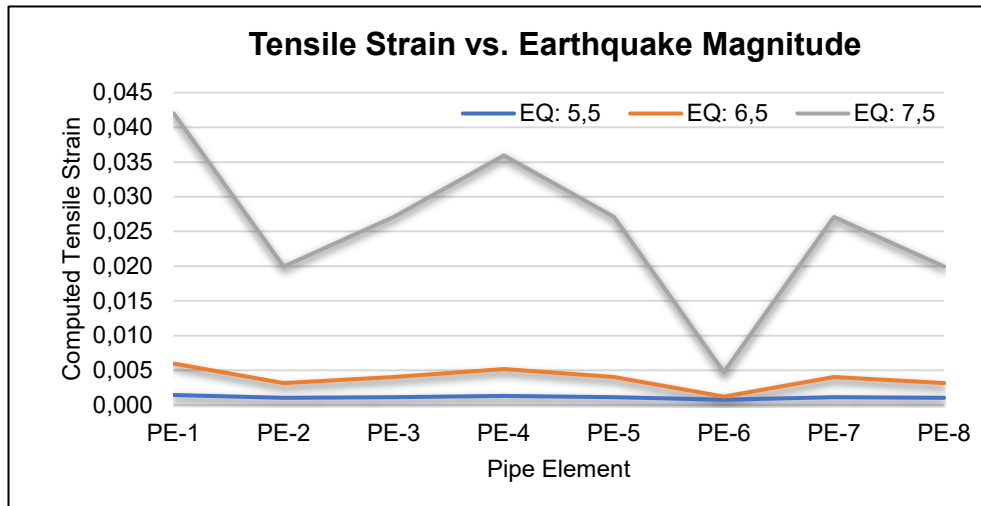


Figure 5.17: Tensile strain vs. earthquake magnitude.

Additionally, the relationship of angle between pipe and fault plane are examined in this study. The graphical results of Figure 5.18 reveal that the value of angle between these two parameters should be kept low to avoid the damage during the earthquakes.

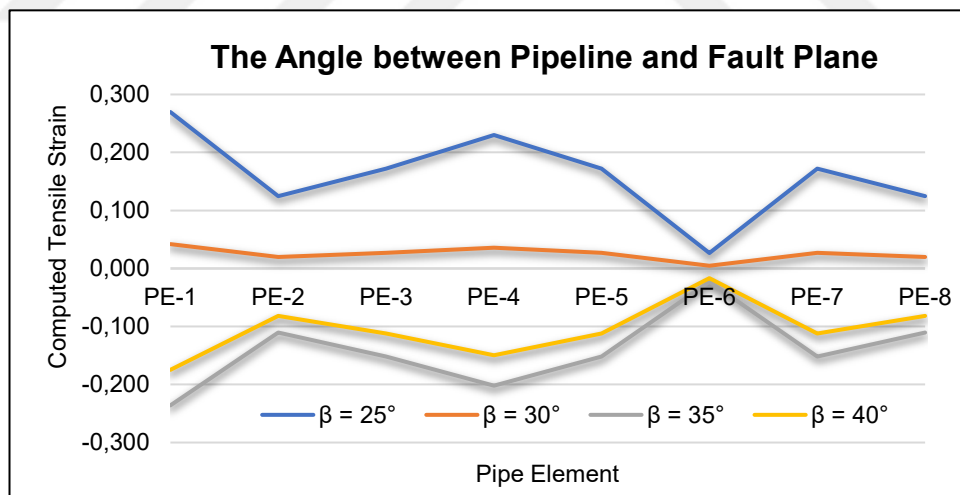


Figure 5.18: The angle between pipe and fault plane.

The effect of dip angle is also analyzed to check the contribution in seismic designing of buried pipelines. It is observed that there is no considerable change in values of dip angle for different pipe segments as shown in Figure 5.19. However, pipe

segments nearby to the fault crossing are prone to damage higher than the longer distances.

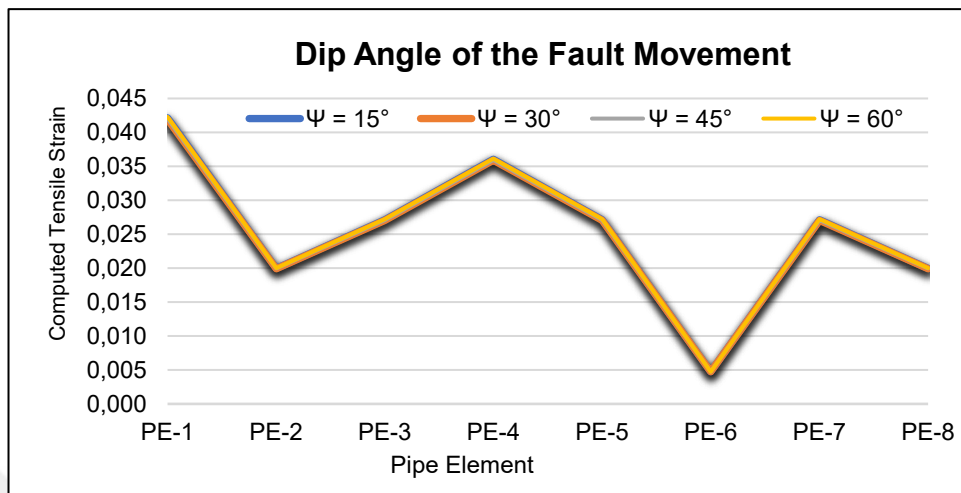


Figure 5.19: Dip angle and fault movement.

## 6. RISK ASSESSMENT

Urban risk assessment of spatially distributed civil engineering structures such as transportation networks and underground pipelines, needs a). identification of critical segments of the infrastructure, type of failure modes and the concerned vulnerability model shapes; b). mechanical properties of the structural system and geotechnical properties of underground soil c). hazard evaluation at the field site d). development of risk assessment tool.

### 6.1. General Overview

As subject research is based on buried gas continuous pipelines, therefore for such systems the information needed are characterized as, the geometrical configuration of the systems, burial depth of the pipelines, internal gas pressure, diameter of pipe, joint and material compositions. These parameters are essential to select the appropriate vulnerability model from the literature studies (e.g., [ALA, 2001], [Piccinelli and Krausmann 2013], [Lanzano et al. 2015]).

After collecting all information related to geometry of pipelines, the network is divided into segments in the form of small elements denoted with length ( $L$ ). These segments are small enough to ignore the property or material differentiation along the whole network. The next step is to assemble the topographical, geological, and geotechnical properties. This information can be taken from the local availability maps such as digital elevation model (DEM) maps, lithological maps, slope maps, shear wave velocity ( $V_{s30}$ ) maps, geotechnical soil maps for angle of friction, cohesion values, unit weight of surrounding soil, and ground water table maps. To develop risk-based assessment soon after the earthquake, the complete information regarding earthquake is also compulsory to collect such as magnitude, epicenter location, fault planes etc. In addition to this, additional data that can be required to develop hazard models include strong ground motion data records that can be taken from the seismic stations. Based on the availability of this information, hazard models can be developed by using GMPEs developed by experts. These GMPEs output the hazard models in the form of peak ground acceleration and peak ground velocities form.

## 6.2. Permanent Ground Deformation (PGD) Model

For the risk assessment of buried gas pipeline systems, PGD model is one of the essential steps because several fragility models are presented in this term. PGA and PGV values evaluated from GMPEs in chapter 3 are not enough to calculate PGD of buried gas pipelines. In simple words, GMPEs can be used for PGD evaluation because no such GMPE is available yet in the literature that can directly assist in PGD calculation. This can be due to the lack of support for the recorded lower frequency strong ground motion records [Malhotra, 2015]. PGD can be computed from the four main parameters such as 1. Co-seismic 2. Land-sliding 3. Faulting 4. Liquefaction. In this study, land sliding, fault crossing, and liquefaction phenomena are selected by ignoring the co-seismic effect as it has no concern on localized displacement concerns.

This research presents three new PGD models such as land sliding, fault crossing and liquefaction. All these three models have similar three phase assessment procedure. 1. Susceptibility analysis 2. Triggering phenomena 3. Total PGD induced. In susceptibility analysis, general failure is looked up based on the empirical equation failure type domains such as strength of earthquake, source-site distance etc. The next phase of triggering is implemented on the locations where susceptibility of failure is expected. For triggering assessment, large scale semiempirical methodologies are applied. In the last step of PGD evaluation, it is completed for such parts/elements where triggering indicates the geotechnical failure. For each section of analysis, final PGD value is the maximum value that is obtained either of three PGD models.

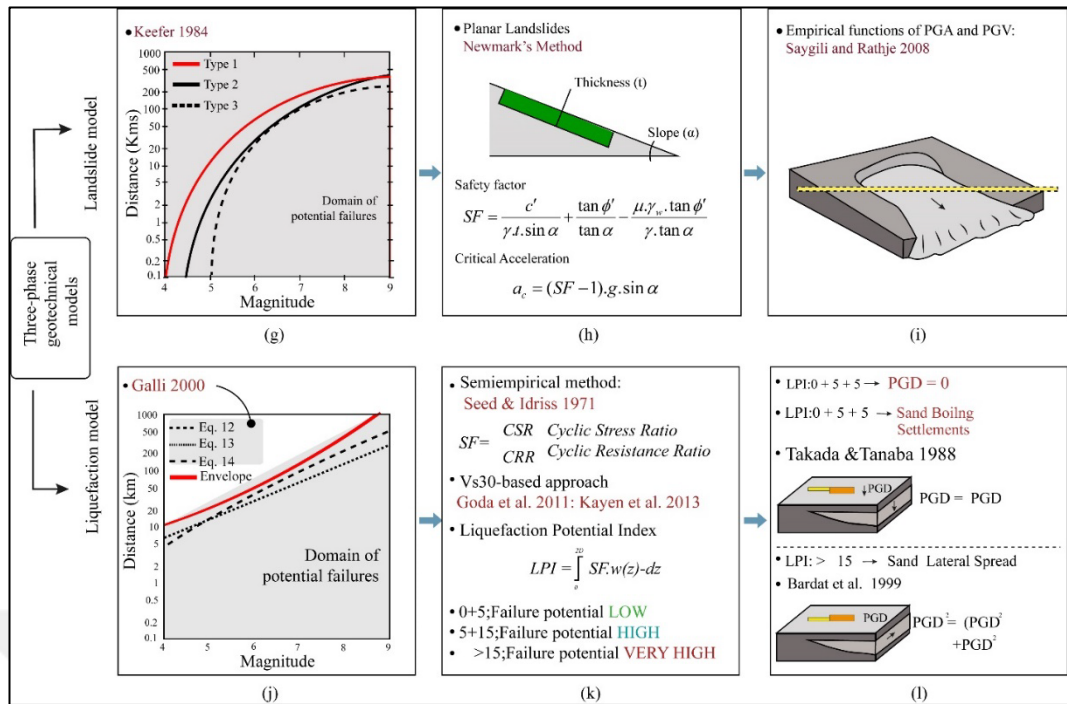


Figure 6.1: (g) Landslide susceptibility by Keefer (1984); (h) triggering analysis; (i) PGD induced due to landslide (j) liquefaction susceptibility (k) triggering analysis and (l) PGD induced due to liquefaction.

### 6.2.1. Landslide Model

The first step of landslide modelling is susceptibility analysis. In current research susceptibility type analysis is completed based on the failure curve type domains presented by Keefer. [Keefer, 1984]. Three different types of failure domains are proposed for three different type of land sliding typologies as shown in Figure (g). In Figure, the susceptibility analysis is completed based on earthquake magnitude and distance in kms. If the results are under the failure domain, then triggering process is completed otherwise PGD value due to land sliding is taken as zero.

This research analysed the triggering analysis considering only translational movement of landslides, i.e., landslides movement along a planar line or undulating rupture floor, sliding out over the unique floor surface [Varnes, 1978]; rotational type landslides is unnoticed due to the fact their protection aspect (SF) is usually more than the SF for planar type landslides [Ferentinou et al., 2006]. The triggering analysis is completed by using Newmark's method [Newmark, 1965]. This version reacts the landslide as a inflexible block that moves on an inclined mode plane [Figure 6.1(h)] while the crucial acceleration  $a_c$  (i.e., the edge base acceleration required to exceed the

surface resistance and provoke sliding) is evaluated. For analyses on the local level, an easy restriction equilibrium version of a limitless slope in cloth having each friction and cohesive energy can be applied, below those hypotheses, the aspect of protection.

$$SF = \frac{c'}{\gamma \cdot t \cdot \sin \alpha} + \frac{\tan \phi'}{\tan \alpha} - \frac{\mu \cdot \gamma_w \cdot \tan \phi'}{\gamma \cdot \tan \alpha} \quad (6.1)$$

The next step is to evaluate the critical acceleration as given in Equation 6.2. by using the SF values computed in equation (6.1). [Saulnier et al., 1997], [Tesfa et al., 2009], [Catani et al., 2010], [Shafique et al., 2011]. The symbol of g denotes the value of seismic acceleration.

$$a_c = (SF - 1) \cdot g \cdot \sin \alpha \quad (6.2)$$

Finally, PGD can be found out by using the well-known Equation given in equation (6.3) developed by [Saygili and Rathje, 2008].

$$\ln P_{GD} = a_1 + a_2 \left( \frac{a_c}{PGA} \right) + a_3 \left( \frac{a_c}{PGA} \right)^2 + a_4 \left( \frac{a_c}{PGA} \right)^3 + a_5 \left( \frac{a_c}{PGA} \right)^4 + a_6 \ln P_{GA} + a_7 \ln P_{GV} \quad (6.3)$$

### 6.2.2. Liquefaction Model

In liquefaction model, the first step is to find the safety factor value that is the ratio of cyclic stress ratio (CSR) to cyclic resistance ratio (CRR) and is given in equation (6.4).

$$SF = \frac{CSR}{CRR} \quad (6.4)$$

The following below equations (6.5) and (6.6). [Goda et al., 2011] and [Kayen et al., 2013] are used to evaluate the SF factor.

$$CSR = 0.65 \frac{PGA}{g} \cdot \frac{\sigma_v(z)}{\sigma'_{v(z)}} \cdot \frac{r_d(z)}{MSF(M)} \quad (6.5)$$

$$CRR = \exp \left\{ \frac{[(0.0073Vs_1)^{2.8011} - 2.6168 \ln(M) - 0.0099 \ln \sigma'_v + 0.0028FC - 0.480\phi^{-1}(PL)]}{1.946} \right\} \quad (6.6)$$

Factor of safety values from the equation (6.4). are then utilized to compute the liquidity Potential Index (LPI). LPI is the decision parameter for PGD check of buried pipelines against the liquefaction hazard.

$$LPI = \int_{20}^0 SF(z) \cdot w(z) \cdot dz \quad (6.7)$$

LPI value is the deciding factor for the liquefaction induced PGD. If the computed value is less than 5, then PGD due to liquefaction is null (zero). Similarly, the values between 5 and 12–15, then vertical settlement is expected to occur along with sand boiling phenomena. Further, the values above the 15, then lateral spreading of the soil would also be considered to occur and in this case SRSS method is used to find the lateral spreading.

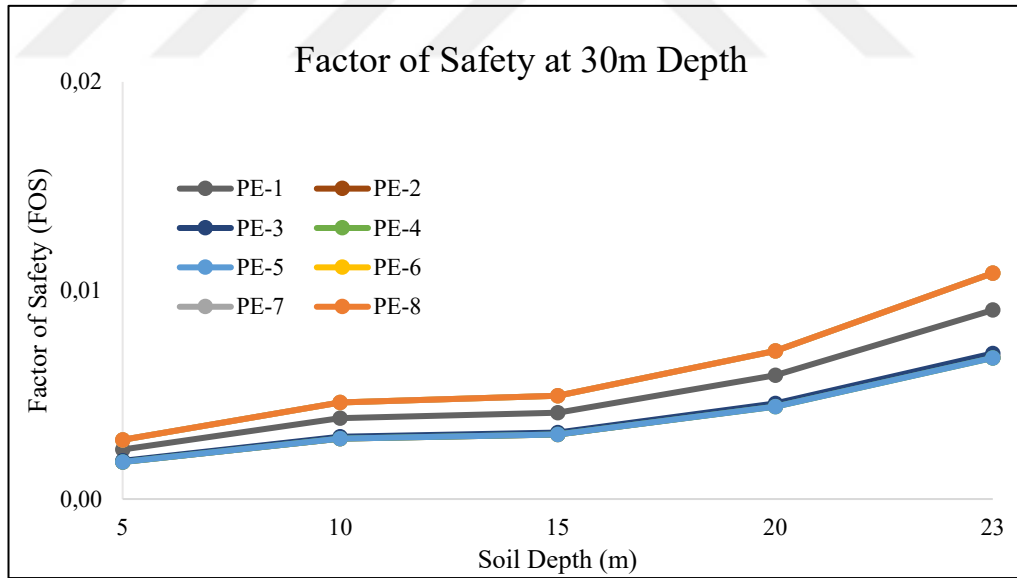


Figure 6.2: FS values computed at 30m depth in current case study.

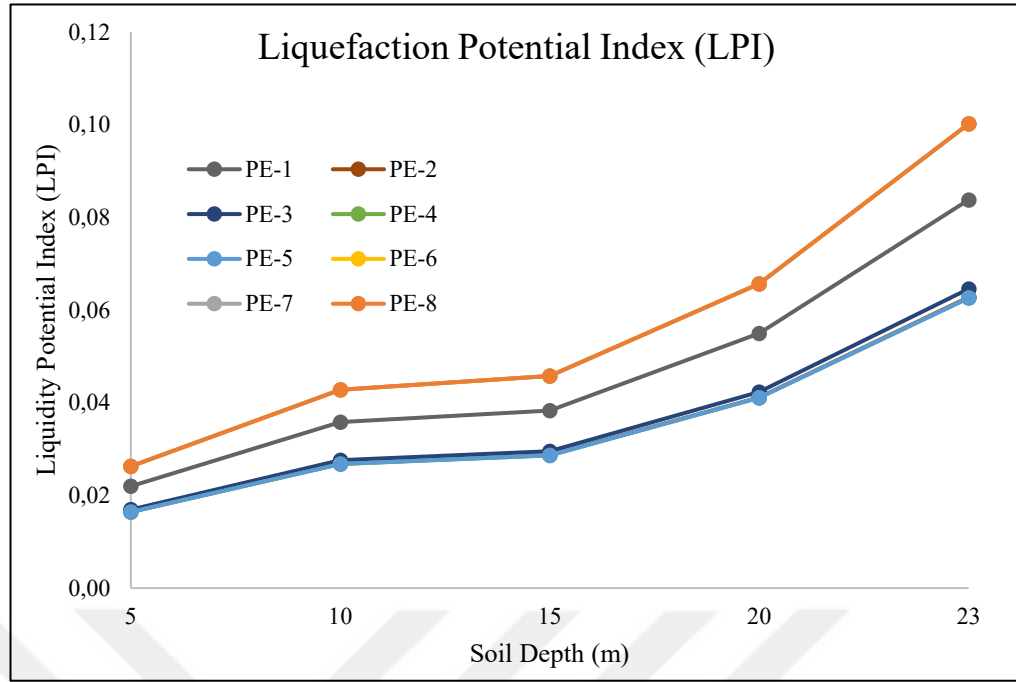


Figure 6.3: LPI values computed at 30m depth in current case study.

Following below assumptions are considered for evaluation of tensile strain in current study. Soil class: D (NEHRP) - Dense/ Stiff soil -  $180 < V_s \leq 360$ ; liquefaction measured at soil depths of 5m, 10m, 15m, 20m and 23m; water table is considered at 5m depth; earthquake Magnitude= EQ = 7,6; for current study, calculated LPI is less than 5, so the liquefaction is not triggered and the PGD due to liquefaction is equal to 0.

### 6.3. Vulnerability Functions and Loss Curves for Natural Gas Pipelines

For buried structures such as NG pipe networks, two types of damage states such as leaks and breaks are generally considered [HAZUS, 2006]. These damage classifying limits are based on TWP or PGD hazards. PGD hazard can be due to liquefaction, lateral spreading, or fault crossing. According to FEMA, if the damage is considered due to TWP hazard, then %age of leaks contributes as 20% and %age of breaks as 80%. Similarly, if damage to failure is characterized by the PGD hazard, then then %age of leaks contributes as 80% and %age of breaks as 20%.

The ALA [ALA, 2001] identifies damage equations for BGCPs damaged due to TWP and PGD hazards that allows the calculation of repair rate values per km length and are given below as.

$$R_{R,SGS} = K_1 \cdot 0.002416 \cdot PGV \quad (6.8)$$

$$R_{R,GF} = K_2 \cdot 11.223 \cdot PGD^{0.319} \quad (6.9)$$

$$\#ofleaks = \sum_i (0.8 \cdot R_{R,SGSi} + 0.2 \cdot R_{R,SGSi}) \cdot L_i \quad (6.10)$$

$$\#ofbreaks = \sum_i (0.2 \cdot R_{R,SGSi} + 0.8 \cdot R_{R,SGSi}) \cdot L_i \quad (6.11)$$

Table 6.1: Vulnerability functions: RR vs. PGV

| Vulnerability relation                                                                                                                                                                                                                                                                                                                                                       | Year |
|------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|------|
| $RR = 10^{b+6.39\log PGA}$                                                                                                                                                                                                                                                                                                                                                   | 1975 |
| $RR = 1.698 \times 10^{-16} \times PGA^{6.06}$                                                                                                                                                                                                                                                                                                                               | 1982 |
| $RR = (PGV/50)^{2.50}$                                                                                                                                                                                                                                                                                                                                                       | 1989 |
| $RR = 0.0001PGV^{2.25}$                                                                                                                                                                                                                                                                                                                                                      | 1993 |
| $RR = K_1 \cdot 0.0001658PGV^{1.98}$                                                                                                                                                                                                                                                                                                                                         | 1995 |
| $RR = 10^{1.25 \times \log PGA - 0.63}$                                                                                                                                                                                                                                                                                                                                      | 1998 |
| $RR = (V_{max}/266)^{1.22}$                                                                                                                                                                                                                                                                                                                                                  | 1999 |
| $RR = 3.11 \times 10^{-3} (PGV - 15)^{1.30}$                                                                                                                                                                                                                                                                                                                                 | 2000 |
| $RR = 0.00187PGV$                                                                                                                                                                                                                                                                                                                                                            | 2001 |
| $RR = \begin{cases} 0 & \text{If } 0 < PGV < 5.35 \\ 0.1172 + 0.7281\varphi(PGV: 51.7964, 19.7811) & \text{If } 5.35 \leq PGV < 95 \\ 0.00137PGV + 0.70458 & \text{If } PGV \geq 95 \end{cases}$ <p>Where, <math>\varphi(PGV; \mu, \sigma) = \int_{-\infty}^{PGV} \frac{1}{\sqrt{2\pi}\sigma} e^{-\left(\frac{1}{2}\right)\left[\frac{v-\mu}{\sigma}\right]^2} dv</math></p> | 2003 |
| $RR = 0.034PGV^{0.92}$                                                                                                                                                                                                                                                                                                                                                       | 2004 |
| $RR = 0.0035PGV^{0.92}$                                                                                                                                                                                                                                                                                                                                                      | 2004 |
| $RR = 513\epsilon_g^{0.89}$                                                                                                                                                                                                                                                                                                                                                  | 2004 |

Table 6.1: (Continue) Vulnerability Functions: RR vs. PGV

|                                       |      |
|---------------------------------------|------|
| $RR = 724\epsilon_g^{0.92}$           | 2004 |
| $RR = 1905\epsilon_g^{1.12}$          | 2009 |
| $RR = 10^{-4.52} \text{GMPGV}^{2.38}$ | 2014 |
| $RR = 0.0839\epsilon_g + 0.41$        | 2014 |
| $RR = 2951\epsilon_g^{1.16}$          | 2015 |

According to Lanzano et al., [Lanzano et al., 2015], the most widely used IMs for the assessment of vulnerability function is the PGV following PGA and MMI as data given in Figure 6.4.

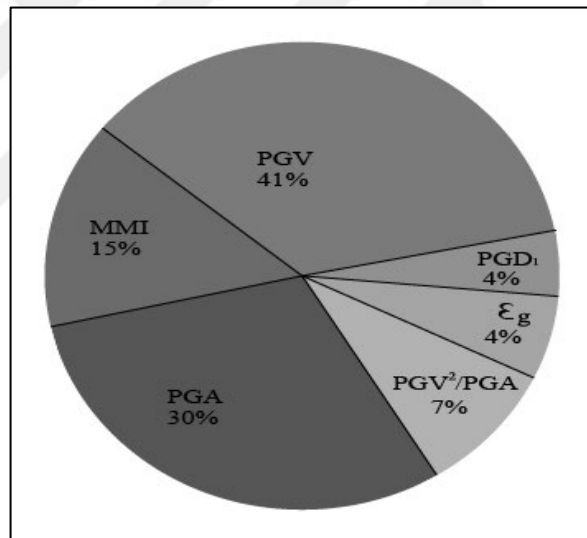


Figure 6.4: Percentage of various seismic intensity parameters used for development of fragility relation.

The Equation presented in table 6.1. are used in current study to analyze the repair rate per km. for each pipe segment.

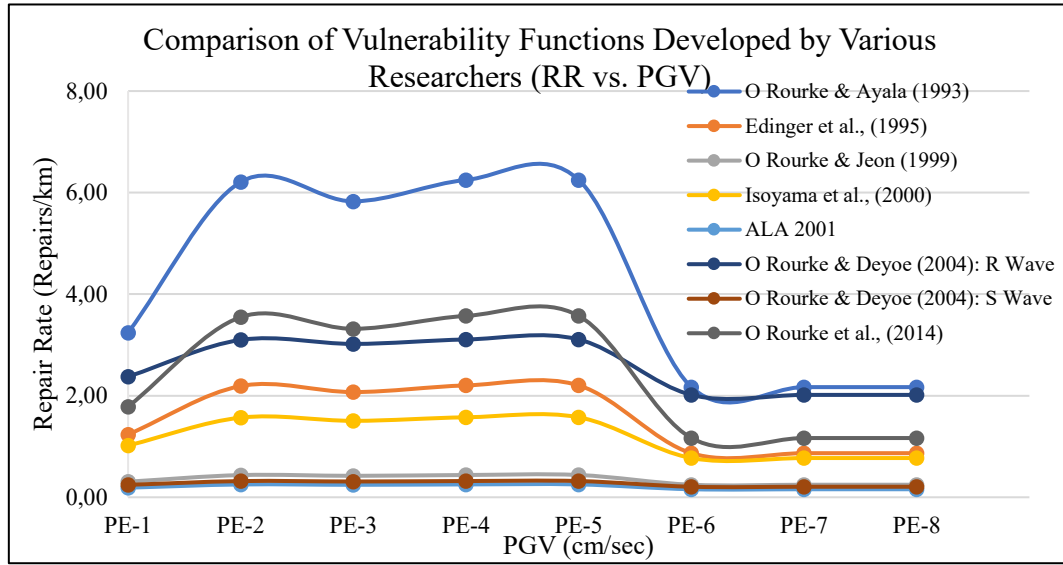


Figure 6.5: Vulnerability functions by various researchers: current study.

It is observed that results of O'Rourke and Ayala [O'Rourke and Ayala, 1993] and O. Rourke and Deyoe [O. Rourke and Deyoe, 2004] for R-waves are overestimated. Edinger et al. [Edinger et al., 1995] curve have limited range as it is applicable only for small diameter segmented pipes. O'Rourke and Jeon [O'Rourke and Jeon, 1999] curve is based only on one earthquake i.e., Northridge earthquake. Isoyama et al. [Isoyama et al., 2000], used six earthquakes' records but the curve is only valid for specific region of Japan as it is based on data collected from 1995 Kobe, Japan Earthquake. ALA [ALA, 2001] have exceptionally low values as compared to other researchers and does not include the pipe and soil properties. Vulnerability functions developed by O'Rourke and Deyoe [O. Rourke and Deyoe, 2004] for R-wave (surface waves) and S-wave (body waves) are based on damage data from five earthquakes. The modification factor termed as  $K_1$  is developed by [ALA, 2005] to add with the repair rate Equation to justify the precision in results and is given in equation (6.12).

$$R_{R,SGS} = K_1 \cdot 0.002416 \cdot PGV \quad (6.12)$$

The multiplier factor considers pipe connection, soil type, pipe material and pipe diameters as values given in Table 6.2.

Table 6.2: Modification factor (K<sub>1</sub>) values for different soil-pipe conditions.

| Identity | Material | Connection     | Soil condition      | Diameter | Multiplier (K <sub>1</sub> ) |
|----------|----------|----------------|---------------------|----------|------------------------------|
| A1       | CI       | C              | All soils           | Small    | 1.00                         |
| A2       | CI       | C              | Corrosive soils     | Small    | 1.40                         |
| A3       | CI       | C              | Non-Corrosive soils | Small    | 0.70                         |
| A4       | CI       | RG             | All soils           | Small    | 0.80                         |
| A5       | WS       | LAW            | All soils           | Small    | 0.60                         |
| A6       | WS       | LAW            | Corrosive soils     | Small    | 0.90                         |
| A7       | WS       | LAW            | Non-Corrosive soils | Small    | 0.30                         |
| A8       | WS       | LAW            | All soils           | Large    | 0.15                         |
| A9       | WS       | RG             | All soils           | Small    | 0.70                         |
| A10      | WS       | S <sub>c</sub> | All soils           | Small    | 1.30                         |
| A11      | WS       | R <sub>v</sub> | All soils           | Small    | 1.30                         |
| A12      | AC       | RG             | All soils           | Small    | 0.50                         |
| A13      | AC       | C              | All soils           | Small    | 1.00                         |
| A14      | CS       | LAW            | All soils           | Large    | 0.70                         |
| A15      | CS       | C              | All soils           | Large    | 1.00                         |
| A16      | CS       | RG             | All soils           | Large    | 0.80                         |
| A17      | PVC      | RG             | All soils           | Small    | 0.50                         |
| A18      | DI       | RG             | All soils           | Small    | 0.50                         |

CI = Cast Iron, WS= welded steel, AC= Asbestos cement, CS= Concrete with cylinder, PVC= Polyvinyl chloride, DI= Ductile iron, C= Cement, RG= Rubber gasketed, S<sub>c</sub>=Screwed, R<sub>v</sub>= Riveted, LAW= Lap arc welded, small diameter= 0.1-0.3m, Large = 0.4m

It is observed that geometric, physical, and mechanical properties of both pipelines and soil can influence their vulnerability. Cast iron pipes of small diameters with cement- based connection (A2) perform poorly under corrosive soil condition than all other combinations. Similarly, welded steel pipes of small diameters with screwed and riveted connection (A10-A11) also perform poorly in all soil conditions. However, welded steel pipes of large diameter with lap arc welded connection (A8) perform very well among all other combination.

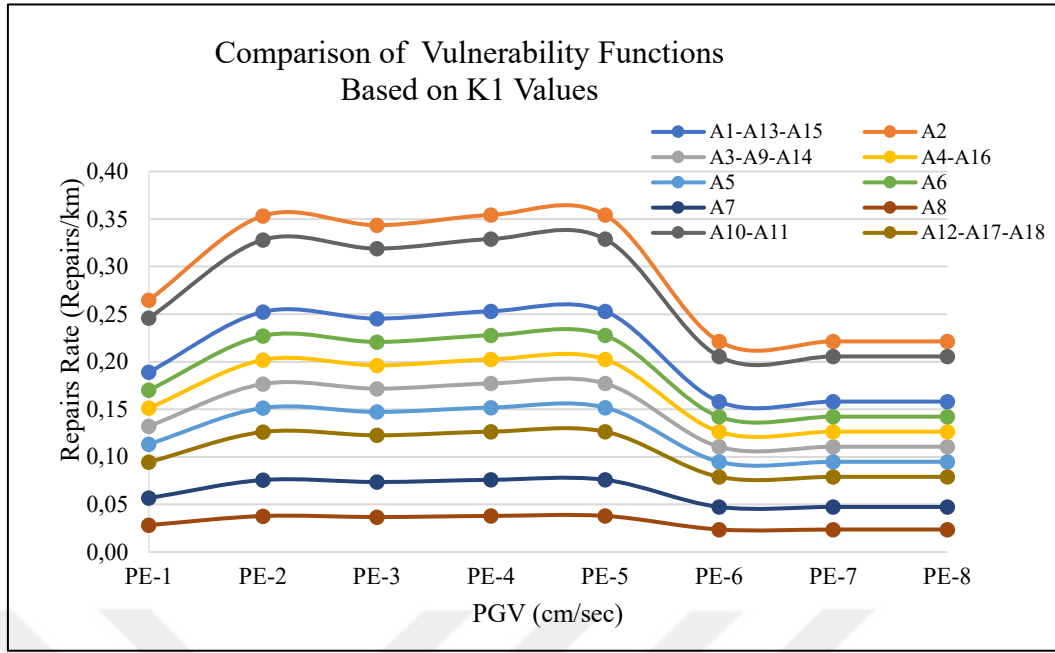


Figure 6.6: Comparison of vulnerability functions based on K1 values.

Loss curve is an important parameter for risk assessment of BGCPs. When the values of # of leaks and # of breaks from above equations are evaluated, then after multiplying these equations with repair cost value gives the loss curve figures. For example, repair cost value for NGP1 type pipe is US\$1,000 as given in HAZUS manual. There are also other manuals and standards that provide more repair costs values for different types of BGCPs [INGA, 2016].

$$losscurve = repair\ cost * \#ofleaks = [US\$1,000]. \sum_i (0.8.R_{R,SGSi} + 0.2.R_{R,SGSi}). L_i \quad (6.13)$$

$$losscurve = repair\ cost * \#ofbreaks = [US\$1,000]. \sum_i (0.2.R_{R,SGSi} + 0.8.R_{R,SGSi}). L_i \quad (6.14)$$

In the following section, considering the literature studies, loss figures in terms of leaks and breaks for current study are developed that yield to expect the damage cost of buried gas pipes. Damage costs are presented in terms of maintenance and repair costs.

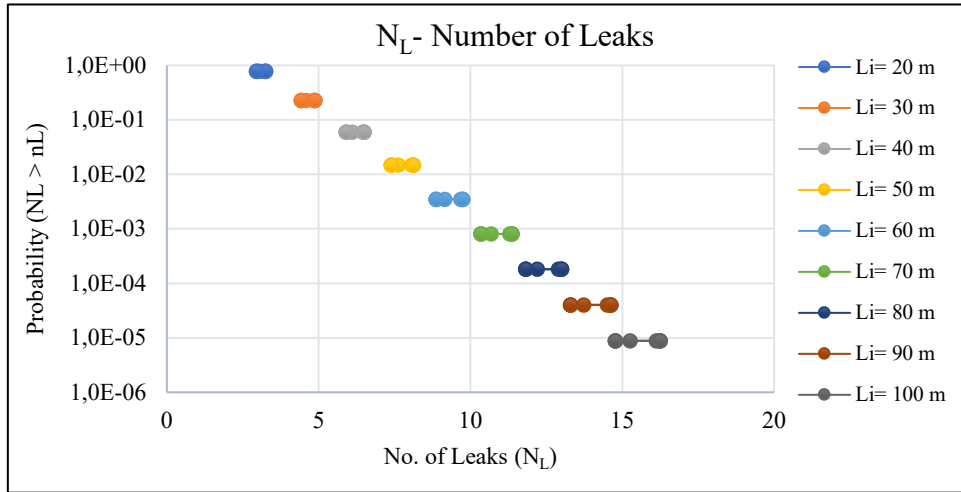


Figure 6.7: Loss figures in terms of number of leaks.

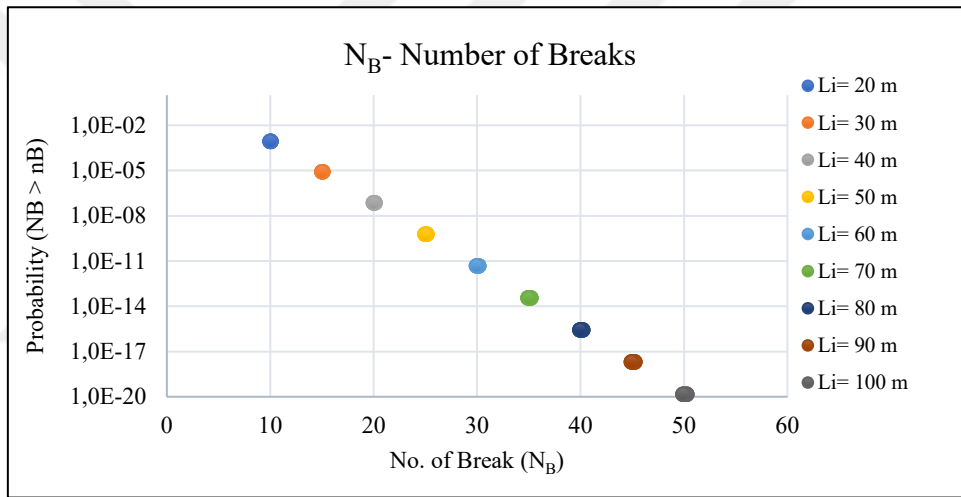


Figure 6.8: Loss figures in terms of number of breaks.

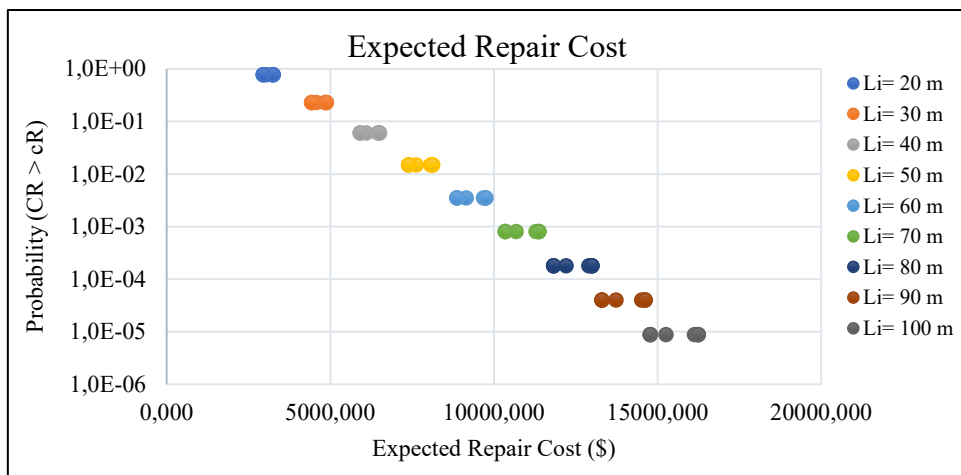


Figure 6.9: Expected repair cost in terms of leaks.

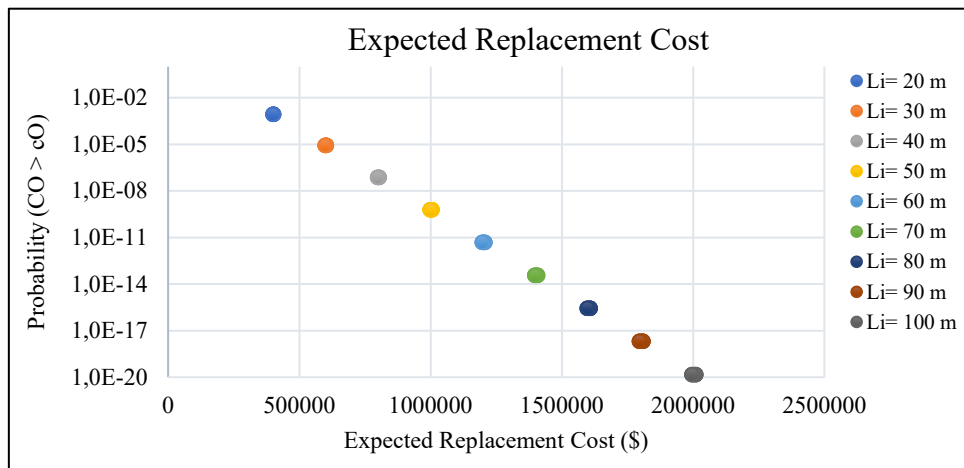


Figure 6.10: Expected replacement cost in terms of breaks.

## 7. NUMERICAL ANALYSIS OF BCGPs

In this chapter, seismic analysis is performed for API grade B 5L steel pipe, a kind of continuous gas pipeline commonly used in Turkey. Three main parameters such as soil type, burial depth and pipe boundary end conditions are studied in detail. Total 10 strong ground motion records are used as input earthquake data that are further scaled from 0.1g to 1.5g to get behavior of selected pipe at various intensity levels. Results from numerical analysis are then compared with the nonlinear fiber element analysis to check the compatibility of the model and its response. The damage to the buried pipe in the form of maximum axial strain are evaluated. The results of current research give a useful insight for seismic performance evaluation of API grade B 5L steel pipe to the researchers of this field. The input parameters and output results are explained in detail in the following sections.

### 7.1. Properties of Steel Pipe and Soil

API grade B 5L steel pipe with diameter of 0.1524 m (6 inches) is used in current analysis. The reason behind this selection is due to its widely usage in gas industry and economical availability at urban level. Material properties of the chosen BCGPs is given in the Table 7.1.

Table 7.1: Material properties of steel pipe.

|                        |                           |
|------------------------|---------------------------|
| <b>Material</b>        | API Grade B 5L Steel Pipe |
| <b>Diameter</b>        | 6 inches (0.1524m)        |
| <b>Density</b>         | 7850 kg/m <sup>3</sup>    |
| <b>Young's Modulus</b> | 207 GPa                   |
| <b>Poisson ratio</b>   | 0.3                       |

Regarding soil profiles, DE class of soil is chosen as it represents the real in-situ geotechnical properties of the region [Özge and Zülfikar, 2014]. In NEHRP soil classification, the shear wave velocity for DE types of soil are classified as 180 m/sec. The layer properties of each DE type soil are given in Table 7.2.

Table 7.2: Properties of DE type of soil class for numerical modelling use.

| Soil Type                | Layer Thickness (m) | Net Unit Weight (kg/m <sup>3</sup> ) | Bulk Modulus (Pa)    | Shear Modulus (Pa)   | Cohesion (Pa)      | Internal friction angle (φ) |
|--------------------------|---------------------|--------------------------------------|----------------------|----------------------|--------------------|-----------------------------|
| Fill                     | 4.0                 | 1400                                 | 2.10x10 <sup>8</sup> | 3.61x10 <sup>7</sup> | 0                  | 33                          |
| High Plasticity Clay     | 1.5                 | 1835                                 | 2.76x10 <sup>9</sup> | 4.16x10 <sup>7</sup> | 0                  | 40.25                       |
| Fine-Medium Grained Sand | 5.0                 | 1848                                 | 2.92x10 <sup>9</sup> | 5.28x10 <sup>7</sup> | 0                  | 39                          |
| Sandstone 1              | 1.5                 | 1873                                 | 3.28x10 <sup>9</sup> | 8.13x10 <sup>7</sup> | 40x10 <sup>4</sup> | 41                          |
| Sandstone 2              | 1.5                 | 1970                                 | 4.94x10 <sup>9</sup> | 2.96x10 <sup>8</sup> | 50x10 <sup>4</sup> | 41                          |
| Sandstone 3              | 3.5                 | 2041                                 | 6.48x10 <sup>9</sup> | 6.27x10 <sup>8</sup> | 60x10 <sup>4</sup> | 41                          |

The missing parameters such as shear modulus, bulk modulus and elastic modulus are calculated by using the below equations (7.1), (7.2) and (7.3).

$$G = \rho \cdot V_s^2 \quad (7.1)$$

In equation (7.1). G is the shear modulus in pascal units. Further, G can also be calculated by using equation (7.2).

$$G = \frac{E}{2(1+\nu)} \quad (7.2)$$

The bulk modulus (K) can be found using the equation (7.3).

$$K = \frac{E}{3(1-2\nu)} \quad (7.3)$$

In above Equations, E is the elastic modulus, and  $\nu$  is the Poisson ratio of the concerned soil layer. The elastic modulus (E) and Poisson ratio ( $\nu$ ) values for each layer is given in Table 7.3.

Table 7.3: Elastic modulus and Poisson ratio values.

| Soil Type                | Elastic Modulus E (Pa) | Poisson ratio |
|--------------------------|------------------------|---------------|
| Fill                     | $1.02 \times 10^8$     | 0.418         |
| High Plasticity Clay     | $1.18 \times 10^8$     | 0.418         |
| Fine-Medium Grained Sand | $1.50 \times 10^8$     | 0.418         |
| Sandstone_1              | $2.31 \times 10^8$     | 0.418         |
| Sandstone_2              | $8.39 \times 10^8$     | 0.418         |
| Sandstone_3              | $1.78 \times 10^9$     | 0.418         |

## 7.2. 3D Finite Element Modelling and Boundary Conditions

Soil-pipe interaction is considered as one of the complex problems for geotechnical engineers since both models undergoes a series of complex interactions when earthquake nonlinear time history loads are applied. Further, if such problems are analyzed as 3D analysis, it requires more efforts due to bundle of variables and takes an extra time gap in comparison with the 2D analysis. In current research, 3D FE analysis program, ABAQUS/CAE is used due to its results accuracy, refined and detailed output analysis and benefits of examining failure modes more easily than its competitive analysis programs. According to Hugel [Hugel, 2008], the features available in ABAQUS/CAE enable the geotechnical engineers to use this package more often at worldwide scale for sub-structure related problems.

For analysis of soil-pipe interaction under ABAQUS/CAE package, the following below assumptions are considered. The weld provided at connection or joint section throughout the pipe length is not considered in this study. The properties of soil are taken as elasto-plastic as described by Mohr Coulomb's theory and buried pipe as elastic in nature and entirely plastic. Buried pipe and surrounding soil are completely connected with each other and the boundary between pipe and soil is ideal with no imperfections.

This dissertation presents the response of BCGPs response in accordance with the change of soil type, boundary conditions and burial depths. The method adopted for analysis is the finite element method (FEM).

The dimensions of soil column are taken as 6m (L) x 6m (W) x 16.5m (H). The pipe is buried at the depth of 1.5m from the top surface layer of soil. The dimensions of soil layers and pipe placement is shown in Figure 7.1. Gas pipe with diameter of 6

inches is used in current study due to its widely usage in household gas transfer at urban level.

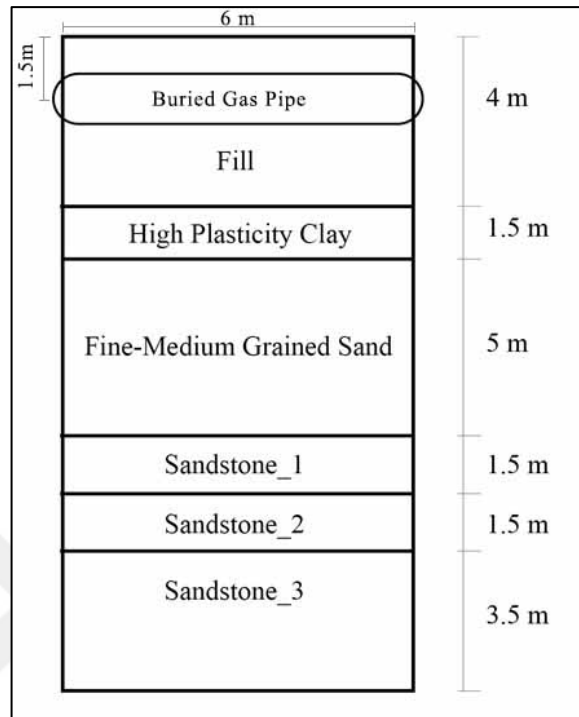


Figure 7.1: Elevation of soil layers and pipe placement.

3D finite element modelling for both soil and buried pipe is completed by using ABAQUS software package. Firstly, parts of each section are created separately. Total 6 parts for different soil layers and one part for API 5 L Grade B pipe is created. After parts step, the properties of each part are assigned in the properties section. The material properties for BCGPs and each soil layers are given in Tables 7.1. – 7.3. The 3D FE model for both soil and pipe are shown in Figure 7.2.

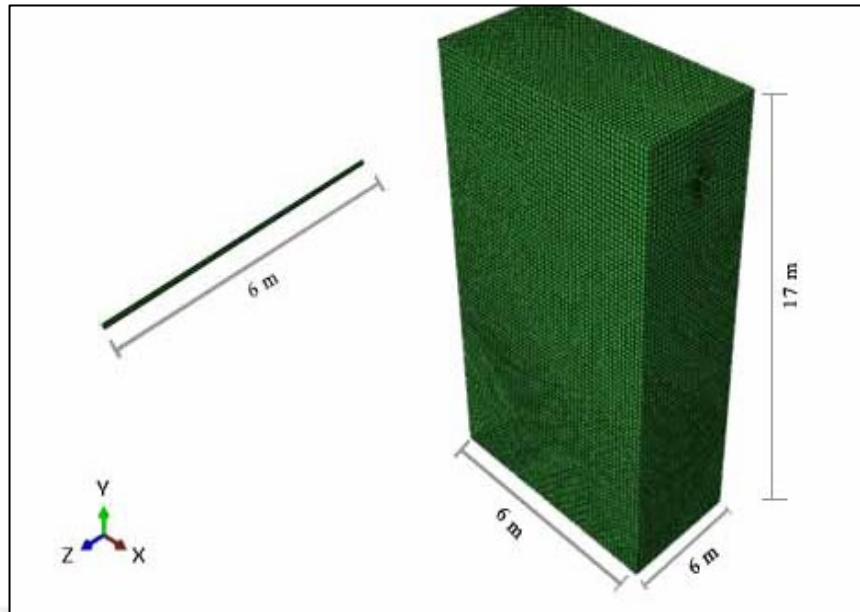


Figure 7.2: 3D finite element models using FE program package of ABAQUS/CAE.

The most important part is the assembly and contact position for two separate models. The contact surface between pipe and soil is created as fully tied to satisfy the condition of third basic assumption of this study as the two FE models are completely bonded to each other and the interface of these two models is entirely perfect without having any imperfection.

The elastic and plastic deformation in the soil-pipe models can be caused by the constraint in between them. In ABAQUS software, there are number of constraint options are available such as tie constraint, rigid body, displaying body, the phenomena of shell to solid coupling, and other constraint equations. In the subject working, tie as constraint is selected as this option connects the buried pipe with the soil layer inside of it as a whole and provides the options of different 3D meshing.

For 3D analysis of soil-pipe in ABAQUS/CAE, generally two types of boundary conditions for soils model are considered as: bottom surface of soil model and four beside surfaces. In current study, the bottom surface is assumed as fixed surface and concerned fixed boundary conditions are applied. The reason of fixed boundary is to restrain its movement in horizontal and vertical sides by considering bottom as rock surface. For beside surfaces of soil, the roller boundary conditions are used as shown in Figure 7.3. It is adjustable to assume rollers for beside boundary of soil because

infinite or semi-infinite medium of soil can be imagined moving vertical side by considering substantial amount of the soil body [Rao, 1999].

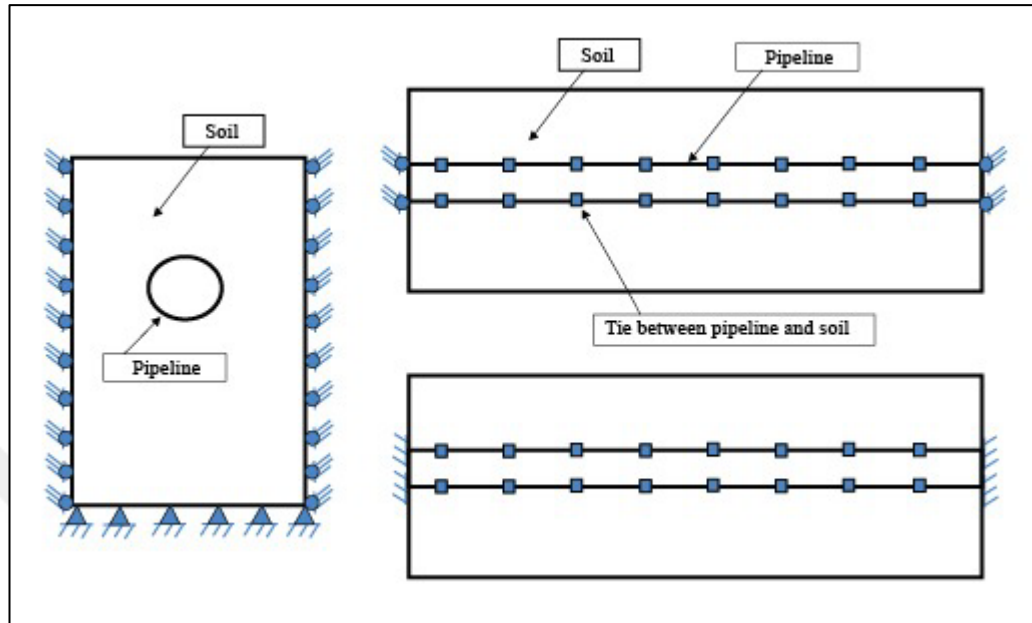


Figure 7.3: Schematic diagram of BCs: (a). soil end BCs; (b). pipe pinned-pinned BCs; (c). pipe fixed-fixed BCs.

In FE analysis of soil-pipe models, meshing plays an important role. The adaptive meshing is applied on each model of soil and pipe. In this step, the type of element, shape of element and number of elements are important to consider as it gives the accuracy to the computed results. For soil, the type of element used is the 3D reduced integration continuum element with 8 nodes as a first order (linear) interpolation (C3D8R). Similarly, for pipe, reduced shell element with four nodes as a first order (linear) interpolation (S4R) is used. The size of element is also particularly important for the accuracy and quality of analysis. In literature, there is no optimal value for meshing to be recommended. However, from available computational capacity one can investigate the meshing from coarse to fine and can observe the size of element for each meshing type. The procedure is also known as ‘h-refinement’ as one can find the adaptive mesh size element by dividing the elements into smaller ones [Lo et al., 2010]. After applying boundary conditions and adaptive meshing to the models, the model is submitted for the analysis.

### 7.3. Seismic Loads and Model Parameters

The type of loads that buried pipes and soil can face are divided into two types as static and dynamic loads. In current analysis, seismic load in the form of real time earthquake ground motions is considered. Total 10 different nonlinear time history data of SGM are chosen for this study. The characteristics with PGA, magnitude, and fault type for each SGM are given in Table 7.4.

Table 7.4: Descriptions of selected ground motions.

| No. | Earthquake  | Symbol | Recording Station  | PGA (g) | Magnitude | Fault Type         |
|-----|-------------|--------|--------------------|---------|-----------|--------------------|
| 1   | Kocaeli     | KOEQ   | YARIMCA(KOERI330)  | 0.349   | 7.51      | Strike-Slip        |
| 2   | Chi -Chi    | CHEQ   | TCU045             | 0.361   | 7.62      | Reverse Oblique    |
| 3   | Imperial V. | IVEQ   | USGS STATION 5115  | 0.315   | 6.53      | Strike-Slip        |
| 4   | Loma Prieta | LPEQ   | CDMG STATION 47381 | 0.367   | 6.93      | Reverse Oblique    |
| 5   | Friuli      | FEQ    | TOLMEZZO           | 0.3513  | 6.50      | Thrust             |
| 6   | Hollister   | HEQ    | USGS STATION 1028  | 0.1948  | 5.6       | Strike-Slip        |
| 7   | Kobe        | KEQ    | KAKOGAWA(CUE90)    | 0.3447  | 6.9       | Strike-Slip        |
| 8   | Landers     | LEQ    | SCE STATION 24     | 0.6418  | 7.3       | Strike-Slip        |
| 9   | Northridge  | NEQ    | CDMG STATION 24278 | 0.5683  | 6.7       | Blind Thrust Fault |
| 10  | Trinidad    | TEQ    | CDMG STATION 1498  | 0.1936  | 6.2       | Strike-Slip        |

Nonlinear acceleration time histories for above earthquakes are given in Figures 7.4. to 7.13.

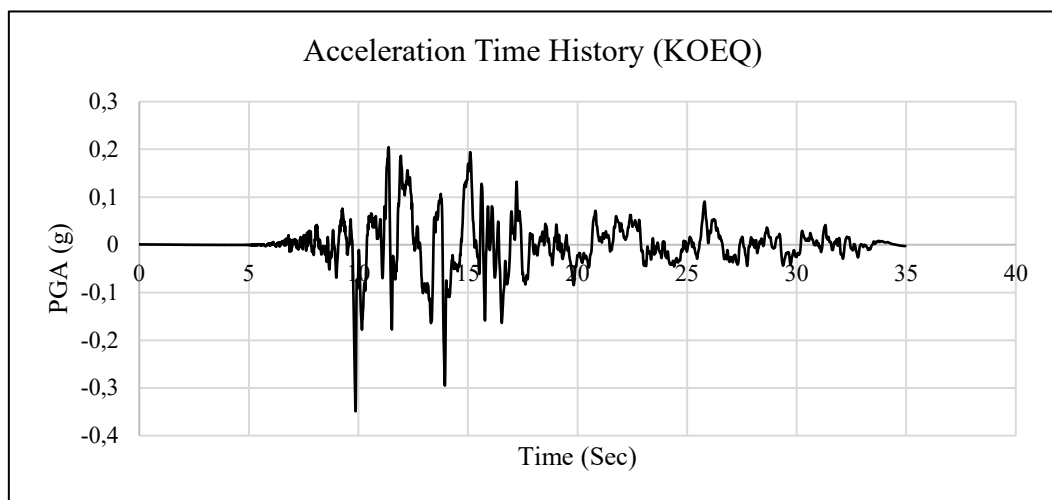


Figure 7.4: The accelerogram of Kocaeli earthquake.

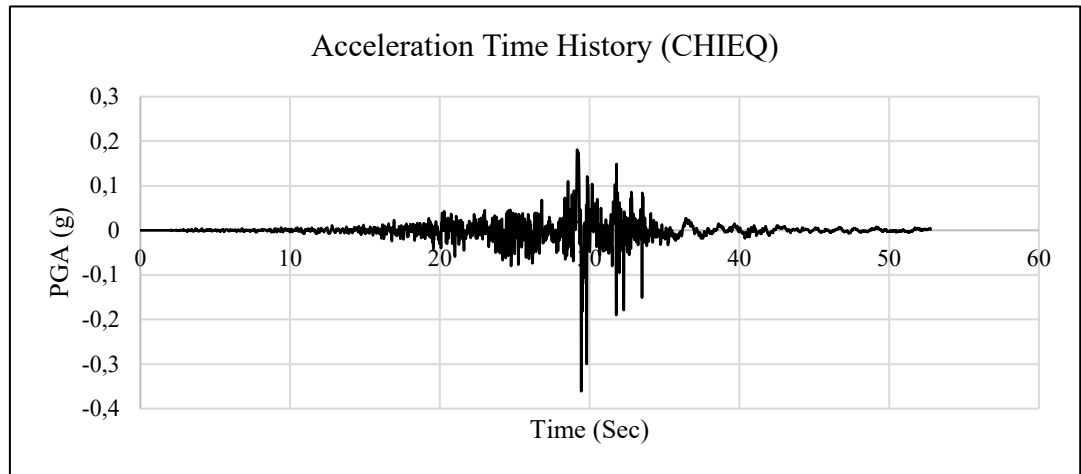


Figure 7.5: The accelerogram of Chi-Chi earthquake.

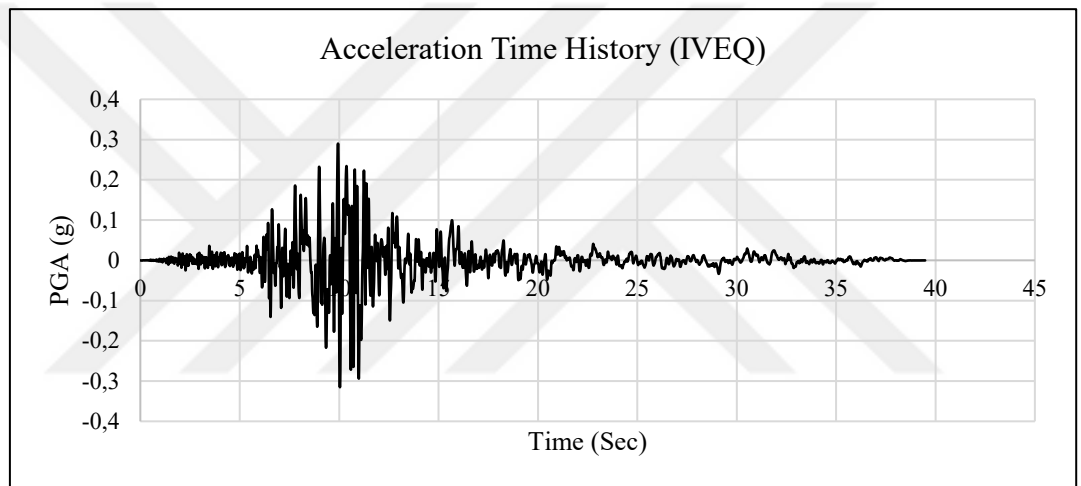


Figure 7.6: The accelerogram of Imperial Valley earthquake.

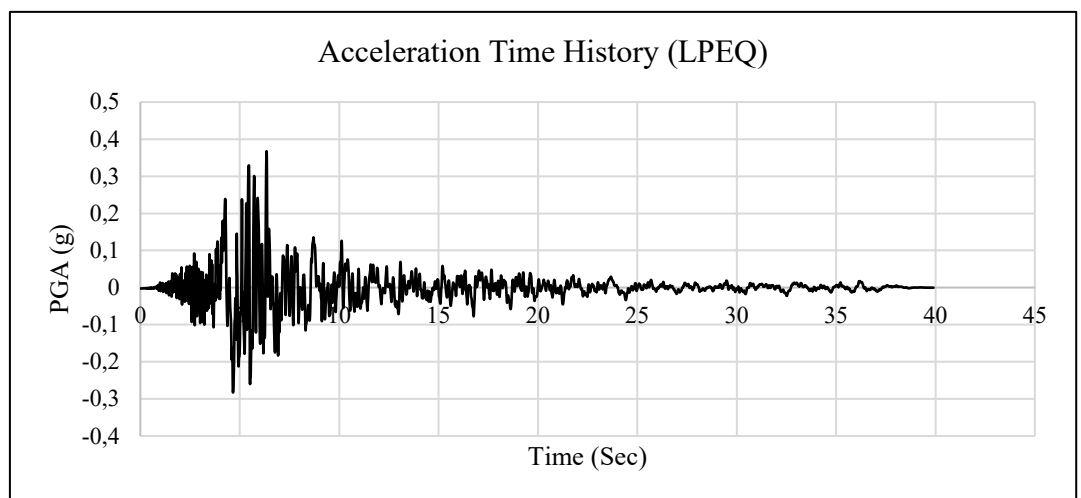


Figure 7.7: The accelerogram of Loma Prieta earthquake.

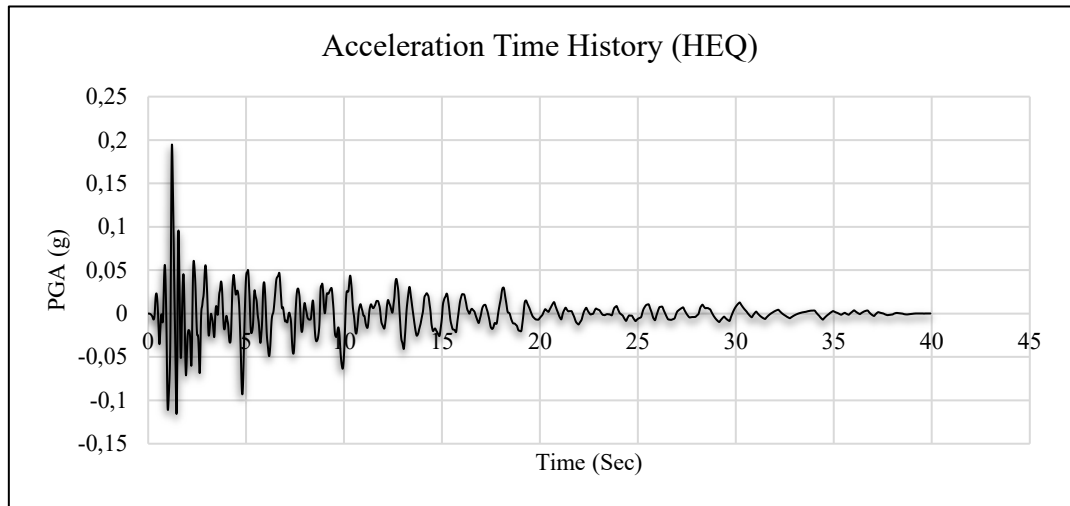


Figure 7.8: The accelerogram of Fruili earthquake.

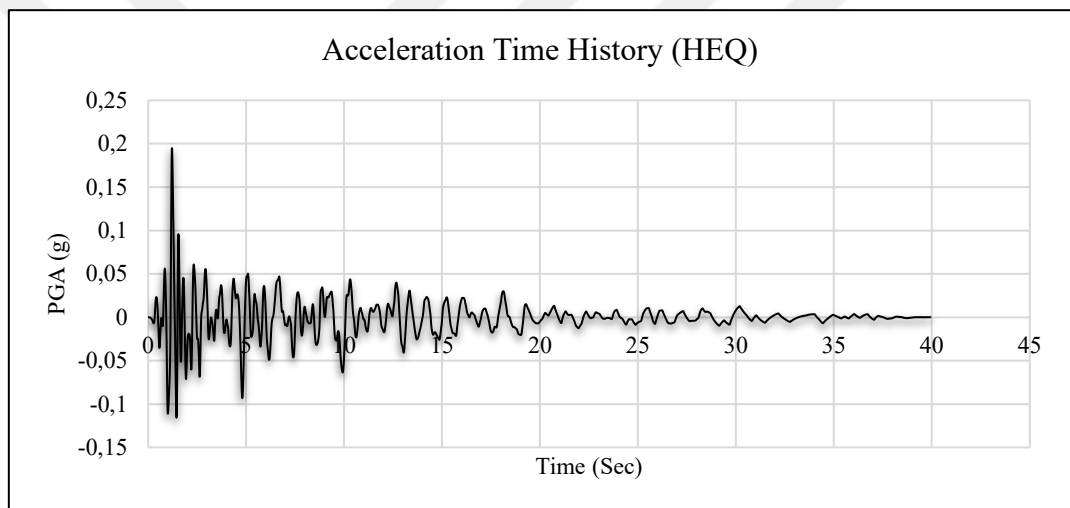


Figure 7.9: The accelerogram of Holliester earthquake.

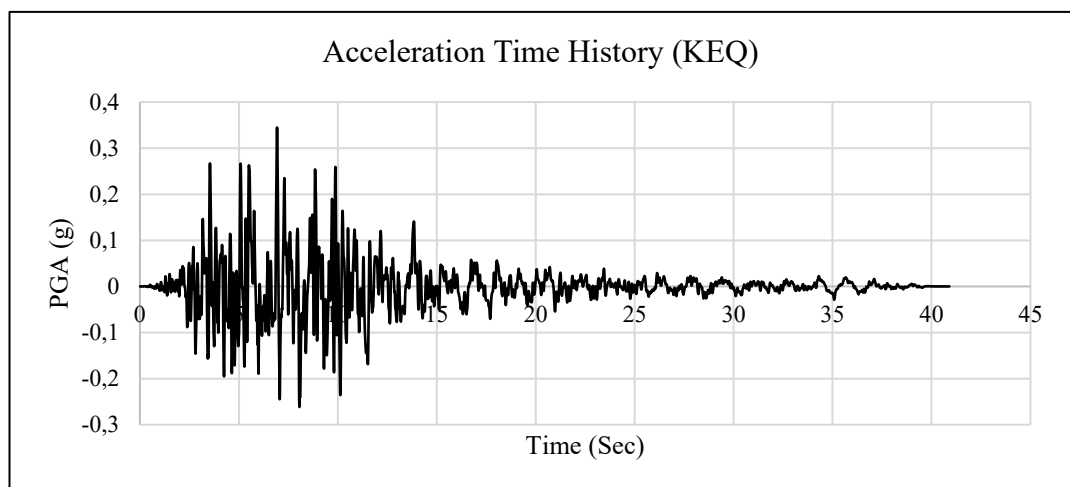


Figure 7.10: The accelerogram of Kobe earthquake.

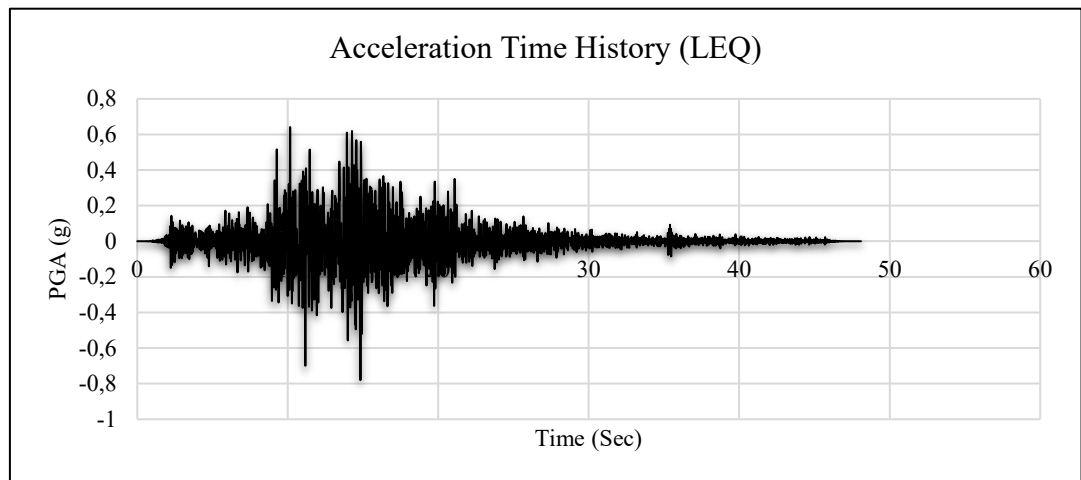


Figure 7.11: The accelerogram of Landers earthquake.

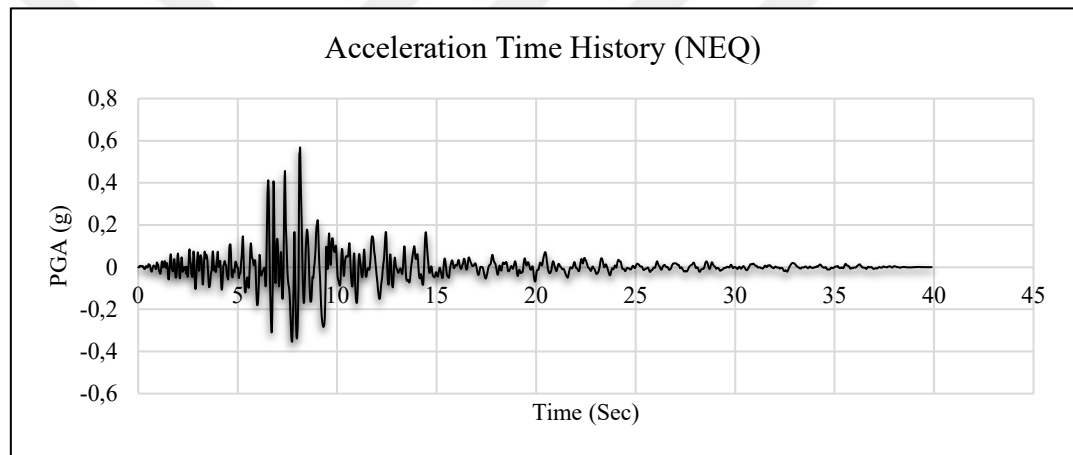


Figure 7.12: The accelerogram of Northridge earthquake.

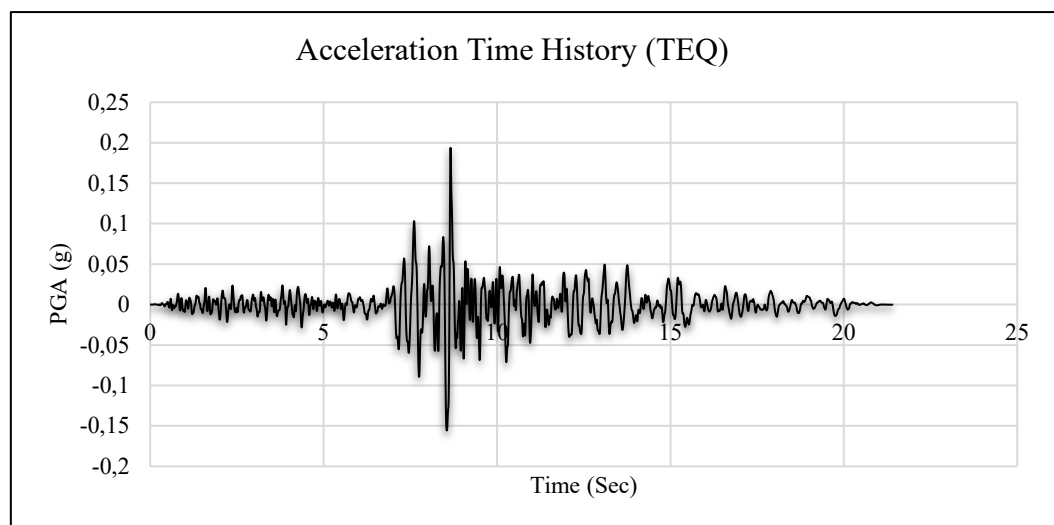


Figure 7.1: The accelerogram of Trinidad earthquake.

Response spectra of selected nonlinear acceleration time histories is shown in Figure 7.14. with an assumed damping ratio of 5%. The Figure 7.14. shows that the selected 10 SGM records are reasonable to use because the maximum spectral acceleration values ( $S_{a_{max}}$ ) and its corresponding period values are comparatively very well distributed on the acceleration response spectrum graph.

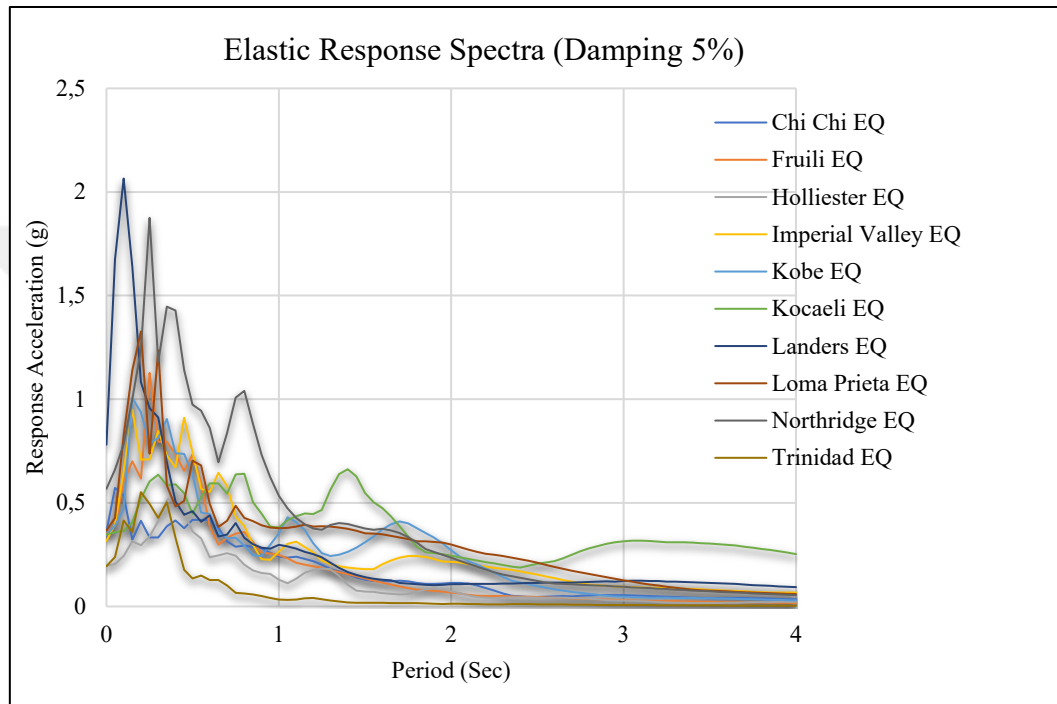


Figure 7.14: Response spectra of selected time histories.

Total 30 different models are presented in this study as described in Table 7.5. The models are based on the variation of 3 main parameters such as change in top layer of soil (P1-P10), changes in pipe burial depth (P11-P20), and pipe boundary end conditions (P21-P30). The effect of pipe burial depths is observed at 1.0m, 1.5 and 2m respectively. Additionally, three type of boundary end conditions such as pinned-pinned (P-P), pinned-fixed (P-F) and fixed-fixed (F-F) are taken in consideration for analysis.

Table 7.5: Model parameters of current study.

| Parameter               | EQ    | Mark | Burial Depth     | Soil Condition | Boundary Condition |
|-------------------------|-------|------|------------------|----------------|--------------------|
| Backfill Soil           | KOEQ  | P1   | 1.5m             | LS-CH-SM-GM    | P-P                |
|                         | CHIEQ | P2   | 1.5m             | LS-CH-SM-GM    | P-P                |
|                         | KEQ   | P3   | 1.5m             | LS-CH-SM-GM    | P-P                |
|                         | IVEQ  | P4   | 1.5m             | LS-CH-SM-GM    | P-P                |
|                         | LPEQ  | P5   | 1.5m             | LS-CH-SM-GM    | P-P                |
|                         | FEQ   | P6   | 1.5m             | LS-CH-SM-GM    | P-P                |
|                         | HEQ   | P7   | 1.5m             | LS-CH-SM-GM    | P-P                |
|                         | LEQ   | P8   | 1.5m             | LS-CH-SM-GM    | P-P                |
|                         | NEQ   | P9   | 1.5m             | LS-CH-SM-GM    | P-P                |
|                         | TEQ   | P10  | 1.5m             | LS-CH-SM-GM    | P-P                |
| Burial Depth            | KOEQ  | P11  | 1.0m, 1.5m, 2.0m | LS             | P-P                |
|                         | CHIEQ | P12  | 1.0m, 1.5m, 2.0m | LS             | P-P                |
|                         | KEQ   | P13  | 1.0m, 1.5m, 2.0m | LS             | P-P                |
|                         | IVEQ  | P14  | 1.0m, 1.5m, 2.0m | LS             | P-P                |
|                         | LPEQ  | P15  | 1.0m, 1.5m, 2.0m | LS             | P-P                |
|                         | FEQ   | P16  | 1.0m, 1.5m, 2.0m | LS             | P-P                |
|                         | HEQ   | P17  | 1.0m, 1.5m, 2.0m | LS             | P-P                |
|                         | LEQ   | P18  | 1.0m, 1.5m, 2.0m | LS             | P-P                |
|                         | NEQ   | P19  | 1.0m, 1.5m, 2.0m | LS             | P-P                |
|                         | TEQ   | P20  | 1.0m, 1.5m, 2.0m | LS             | P-P                |
| Boundary End Conditions | KOEQ  | P21  | 1.5m             | LS             | P-P, P-F, F-F      |
|                         | CHIEQ | P22  | 1.5m             | LS             | P-P, P-F, F-F      |
|                         | KEQ   | P23  | 1.5m             | LS             | P-P, P-F, F-F      |
|                         | IVEQ  | P24  | 1.5m             | LS             | P-P, P-F, F-F      |
|                         | LPEQ  | P25  | 1.5m             | LS             | P-P, P-F, F-F      |
|                         | FEQ   | P26  | 1.5m             | LS             | P-P, P-F, F-F      |
|                         | HEQ   | P27  | 1.5m             | LS             | P-P, P-F, F-F      |
|                         | LEQ   | P28  | 1.5m             | LS             | P-P, P-F, F-F      |
|                         | NEQ   | P29  | 1.5m             | LS             | P-P, P-F, F-F      |
|                         | TEQ   | P30  | 1.5m             | LS             | P-P, P-F, F-F      |

## 7.4. Threshold Limit Values

The failure aspects in BCGPs are usually considered by taking into consideration the longitudinal axial strain as damage parameter. The failure associated due to flexural strain can be neglected as ground curvatures insert a very negligible value of flexural strain in the pipes. The threshold limit values for maximum longitudinal axial strain presented in different standards are given in Table 7.6.

Table 7.6: Threshold limit values presented in different standards.

| Standard/ Code Name     | ALA 2005 | JSCE 2000 | API 1104 | ASCE 1984 |
|-------------------------|----------|-----------|----------|-----------|
| Max. Axial Strain Limit | 0.03     | 0.03      | 0.04     | 0.03-0.05 |

Additionally, damage limit states categorized as damage levels are also presented by various researchers and in published reports for maximum allowable axial strain of BCGPs and are summarized in Table 7.7.

Table 7.7: Maximum allowable tensile strain values by various researchers.

| Damage Level              | Limit | Description                                                                                         | Report /<br>Researcher          |
|---------------------------|-------|-----------------------------------------------------------------------------------------------------|---------------------------------|
| Operational level<br>(OL) | 0.01  | May be a little plastic deformation, however the pipe will continue to operate in normal condition. | JG(G)-206-03,<br>EN1998-4       |
| Pressurized level<br>(PL) | 0.015 | Plastic deformation can occur, however no leakage from gas pipe                                     | CEN, Bai, and<br>Bai            |
| Leakage Level<br>(LL)     | 0.02  | Leakage occur that can be controlled                                                                | Honegger et al.,<br>Bai and Bai |
| Collapsed Level<br>(CL)   | 0.03  | Complete collapsed of the buried gas pipe                                                           | Zhang, Ahmed<br>et al.          |

## 7.5. Output for Seismic Analysis by ABAQUS/CAE

The created model is analyzed for three different parameters such as change in soil types, boundary conditions and burial depths. For this, the selected nonlinear acceleration time histories presented in Table 7.4. are scaled from 0.1g to 1.5g with an interval of 0.5g to examine the response of each ground motion intensity more precisely. The deformation shapes obtained after analysis indicated that maximum strain in soil is produced at the upper layers and values decrease as the layers are trend to downward sides. Similarly, the maximum strain in buried steel pipe is observed at the center rather on the end or peripheral sides and it is due to the application of boundary conditions and interface used between pipe-soil.

### 7.5.1. Effect of Backfill Soil

As described in earlier section, the pipe is buried at the depth of 1.5m from the top surface of soil. The burial depth of pipe is kept constant and the maximum tensile strain values are computed by changing the surrounding soil types of first 4m (first layer) such as loose soil, high plastic clay, medium sand, and sandstone, respectively. From Figures 7.15. to 7.24., It can be noticed that as the gas pipe is buried in loose soil, the computed maximum axial strain values are more than the other corresponding

soil types. Therefore, the fill soil for buried pipes play an important role to ensure the safety of BCGPs against seismic events.

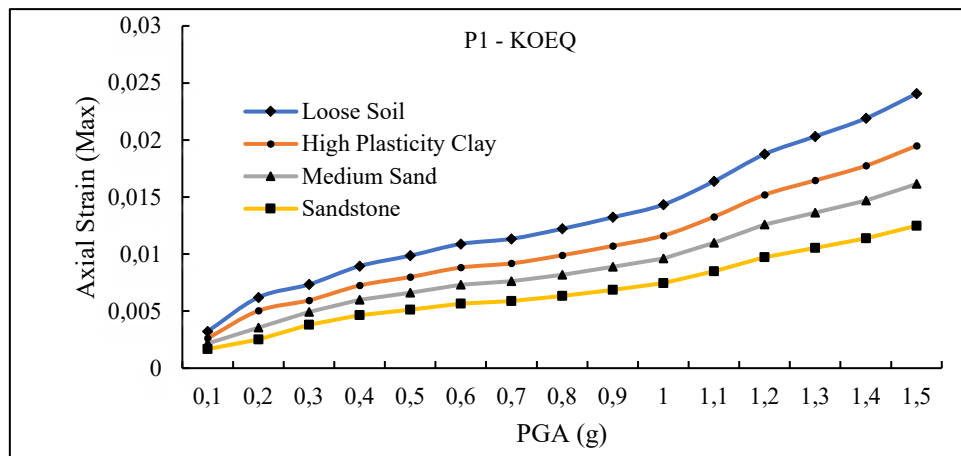


Figure 7.15: Maximum axial strain response of BCGPs for P1 model.

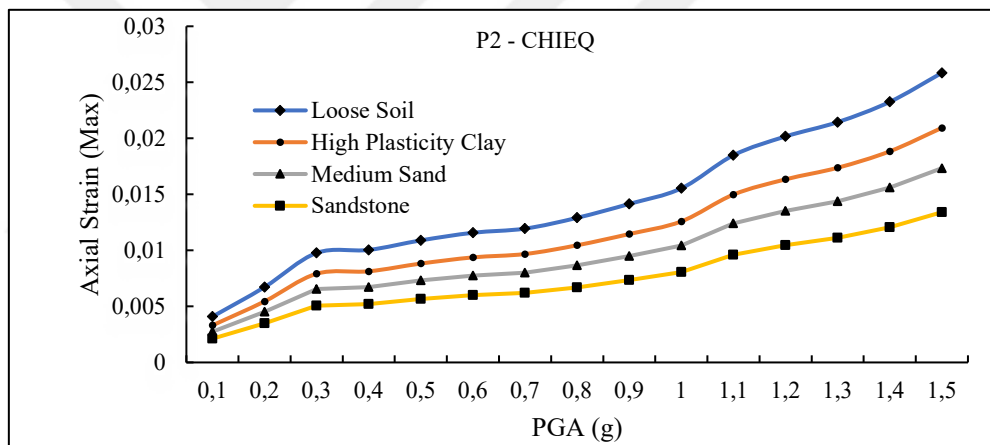


Figure 7.16: Maximum axial strain response of BCGPs for P2 model.

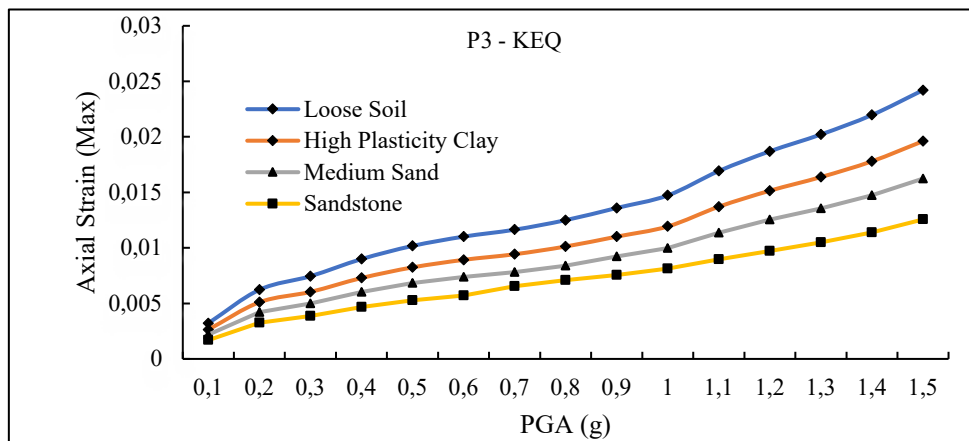


Figure 7.17: Maximum axial strain response of BCGPs for P3 model.

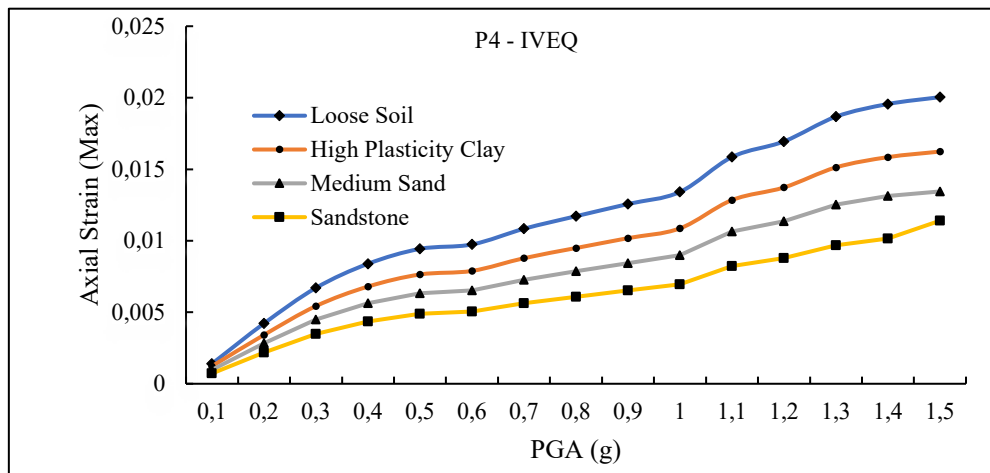


Figure 7.18: Maximum axial strain response of BCGPs for P4 model.

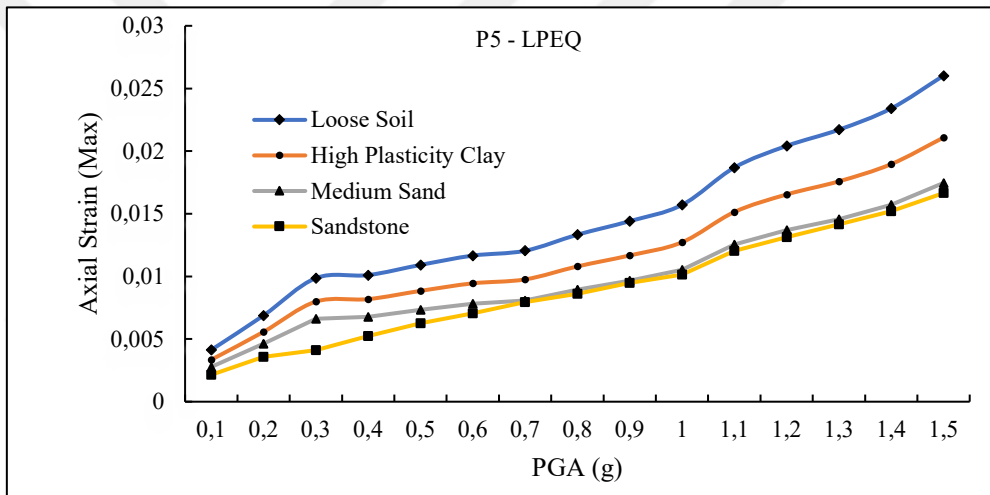


Figure 7.19: Maximum axial strain response of BCGPs for P5 model.

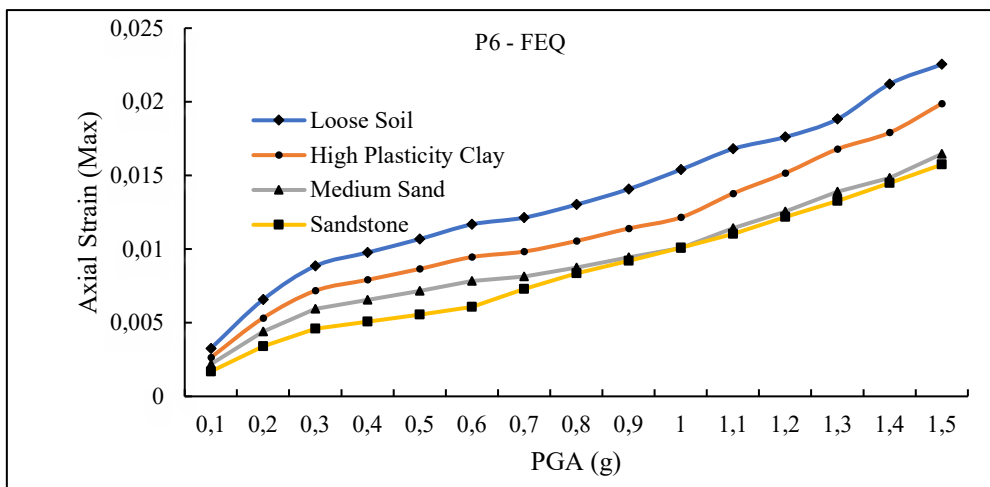


Figure 7.20: Maximum axial strain response of BCGPs for P6 model.

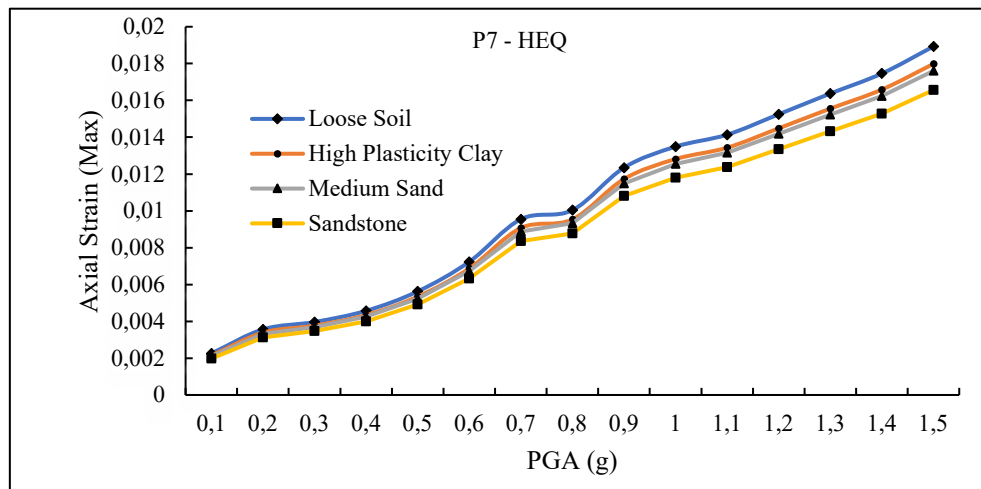


Figure 7.21: Maximum axial strain response of BCGPs for P7 model.

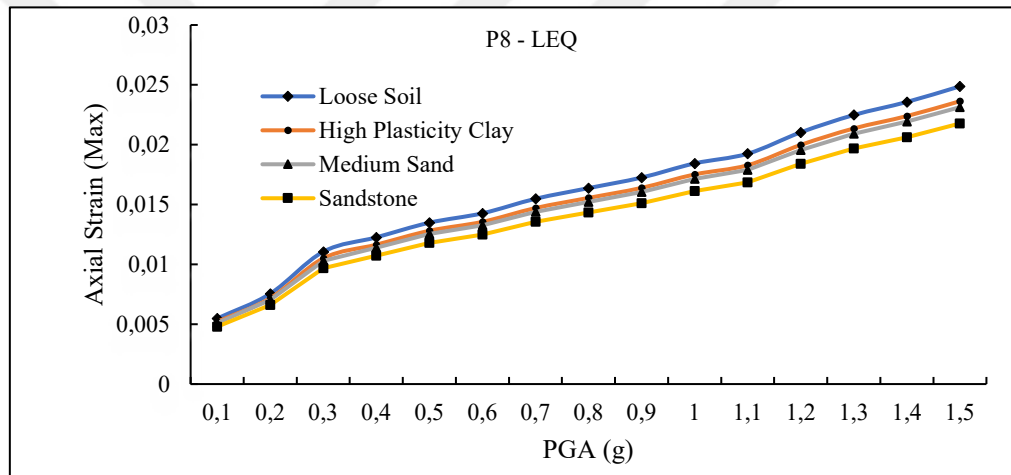


Figure 7.22: Maximum axial strain response of BCGPs for P8 model.

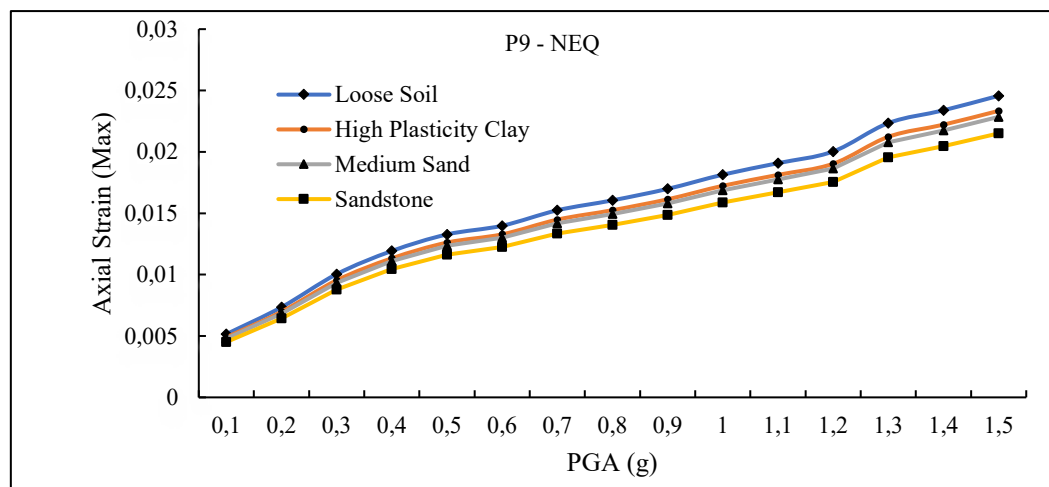


Figure 7.23: Maximum axial strain response of BCGPs for P9 model.

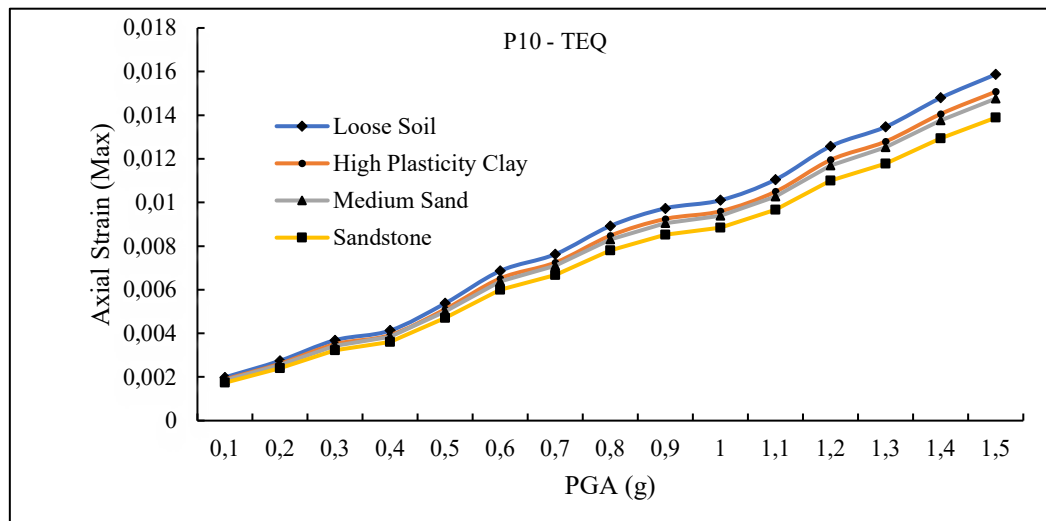


Figure 7.24: Maximum axial strain response of BCGPs for P10 model.

The graphical results of P.1 to P.4 models are well matched at lower PGA values, however the maximum axial strain tends to increase at higher PGA levels. Similarly, for models P.5 to P.10, the curves are very close to each other and there is a slight difference in maximum axial strain values. Comparing the accumulated values with the threshold limit values with Table 7.6 and Table 7.7, it is noticed that the selected pipe can operate safely under assumed conditions without facing collapsed level (CL). The damage level based on maximum axial strain observed in each model against various PGA (g) values is summarized in Table 7.8.

Table 7.8: Damage levels observed in models P.1-P10.

| Model | Damage States for LS Type Soil |            |           |           |
|-------|--------------------------------|------------|-----------|-----------|
|       | OL (0.01)                      | PL (0.015) | LL (0.02) | CL (0.03) |
| P1    | 0.35g                          | 1.05g      | 1.34g     | -         |
| P2    | 0.25g                          | 0.95g      | 1.18g     | -         |
| P3    | 0.34g                          | 1.04g      | 1.25g     | -         |
| P4    | 0.55g                          | 1.15g      | 1.35g     | -         |
| P5    | 0.24g                          | 1.05g      | 1.25g     | -         |
| P6    | 0.36g                          | 1.06g      | 1.38g     | -         |
| P7    | 0.78g                          | 1.15g      | -         | -         |
| P8    | 0.20g                          | 0.68g      | 1.25g     | -         |
| P9    | 0.26g                          | 0.66g      | 1.34g     | -         |
| P10   | 1.0g                           | 1.48g      | -         | -         |

From Table 7.8 for LS type, OL state is observed at approx. 0.20g – 0.35g in P.1 - P.2 - P.3 - P.5 - P.6 - P.8 and P.9 models, and in P.4 - P.7 - P.10 models at approximately 0.55g – 1.0g respectively. Similarly, PL damage state is noticed at approx. 0.65g – 1.05g in P.1 - P.2 - P.3 - P.5 - P.6 - P.8 and P.9 models, and in P.4 - P.7 at approximately 1.15g, in P.10 model at 1.48g respectively. LL damage state is seen between 1.18g – 1.38g in P.1 - P.2 - P.3 - P.4 - P.5 - P.6 - P.8 and P.9, however no LL level is available for P.7 and P.10 models. Furthermore, CL level is not observed in all 10 models of this study.

### **7.5.2. Effect of Changing Pipe Burial Depth**

The pipes are usually buried in soil due to many reasons such as aesthetic, safety and economical aspects. The knowledge from past seismic assessments shows that superstructures are affected more than the buried structures. However, buried pipes placed at shallow depths are considered as damage first due to ground shaking. Therefore, burial depth is an important parameter to study the seismic behaviour of BCGPs. In this study, 10 models (P.11- P.20) from Table 7.5 are considered to check the burial depth effect on the subject pipe. The pipe is buried at 1.0m, 1.5m and 2.0 m by keeping the soil as constant LS type with pipe boundary end conditions as pinned-pinned. The maximum axial strain effect for considered burial depths is presented in Figure 7.25 - Figure 7.34. From the results, it is concluded that pipe buried at shallow depth such as 1.0m depth in current study is prone to higher damage due to exceedance of axial strain in the pipe than the maximum limit value of 0.03.

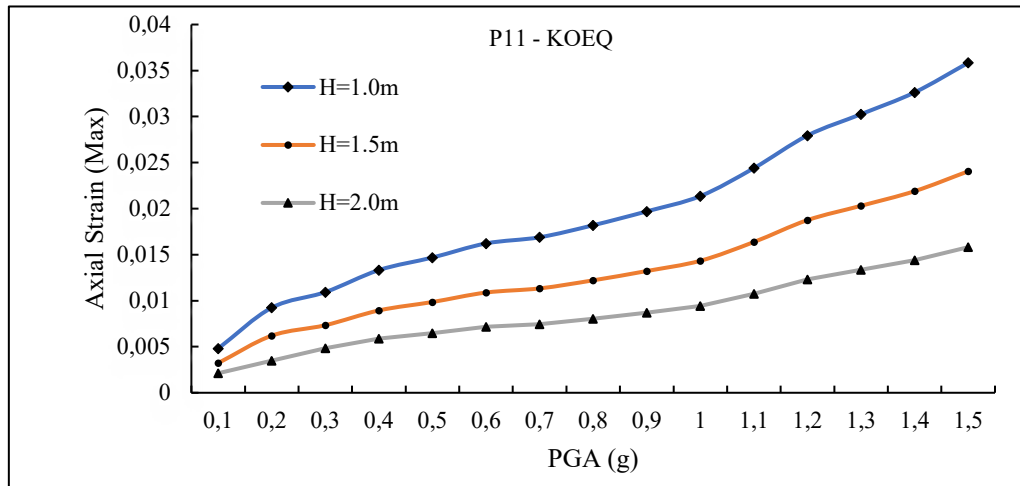


Figure 7.25: Maximum axial strain response of BCGPs for P11 model.

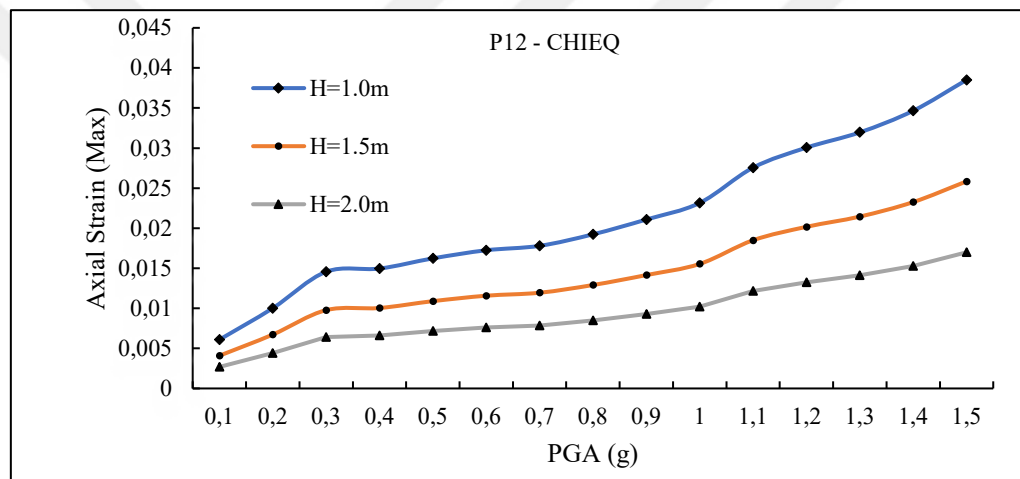


Figure 7.26: Maximum axial strain response of BCGPs for P12 model.

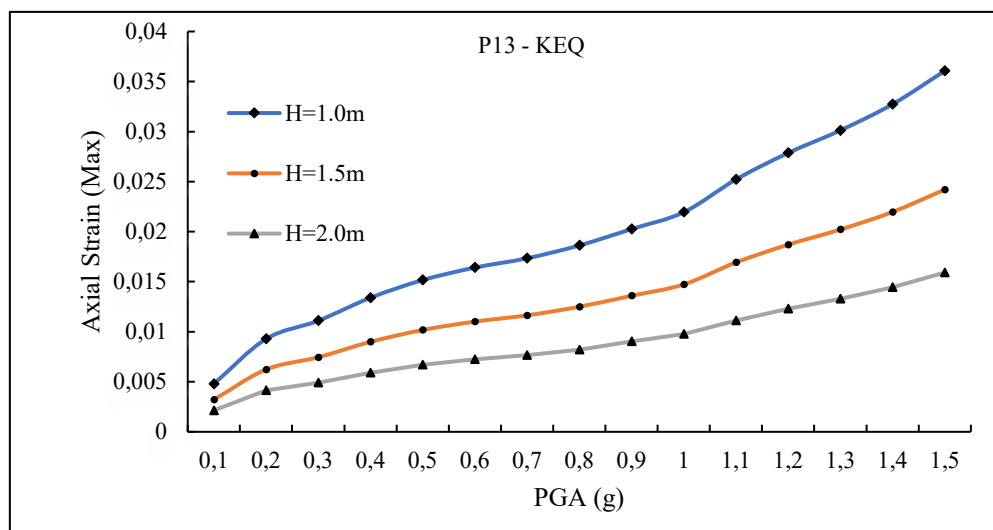


Figure 7.27: Maximum axial strain response of BCGPs for P13 model.

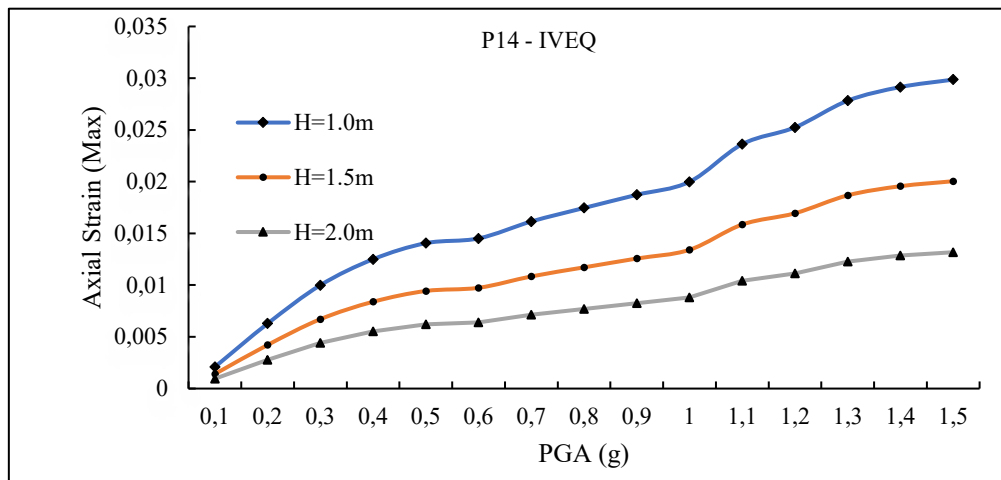


Figure 7.28: Maximum axial strain response of BCGPs for P14 model.

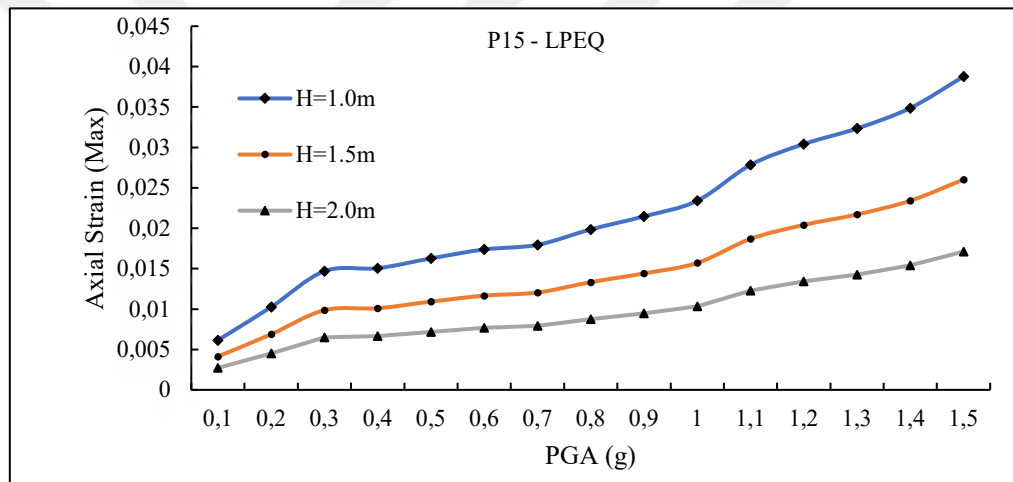


Figure 7.29: Maximum axial strain response of BCGPs for P15 model.

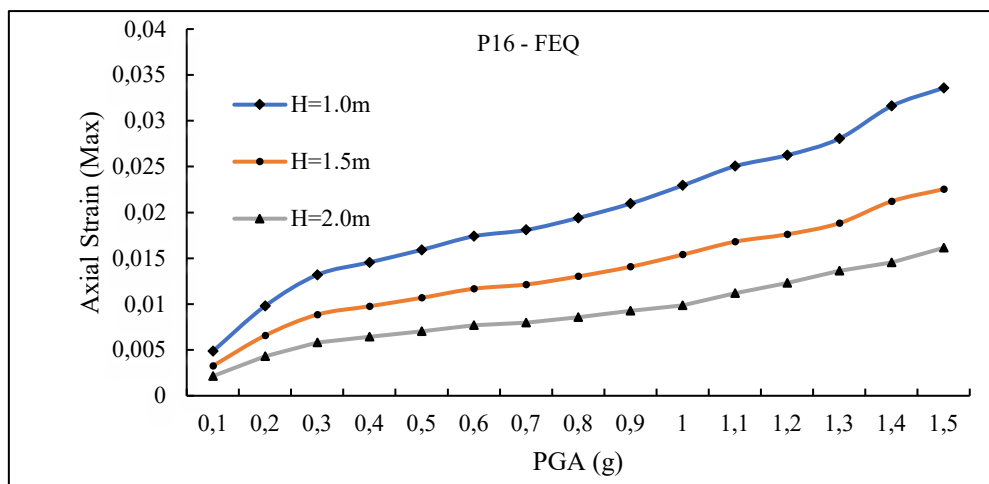


Figure 7.30: Maximum axial strain response of BCGPs for P16 model.

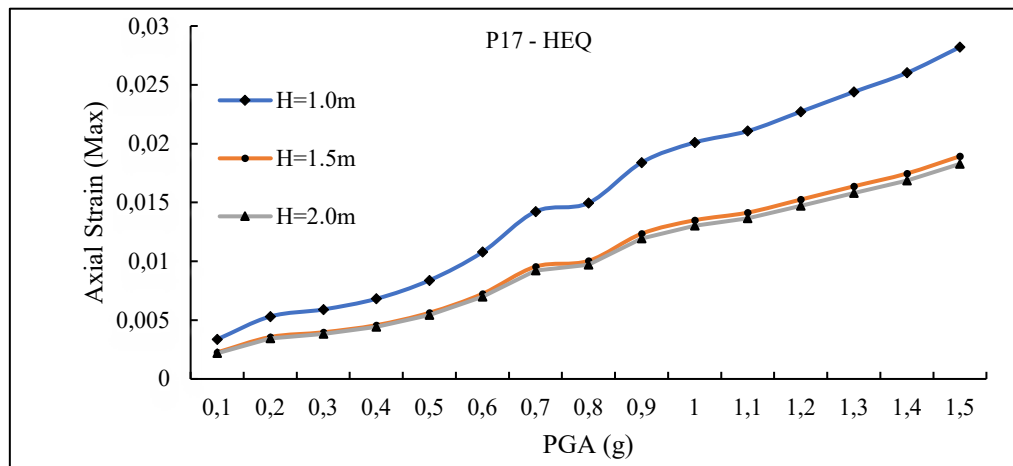


Figure 7.31: Maximum axial strain response of BCGPs for P17 model.

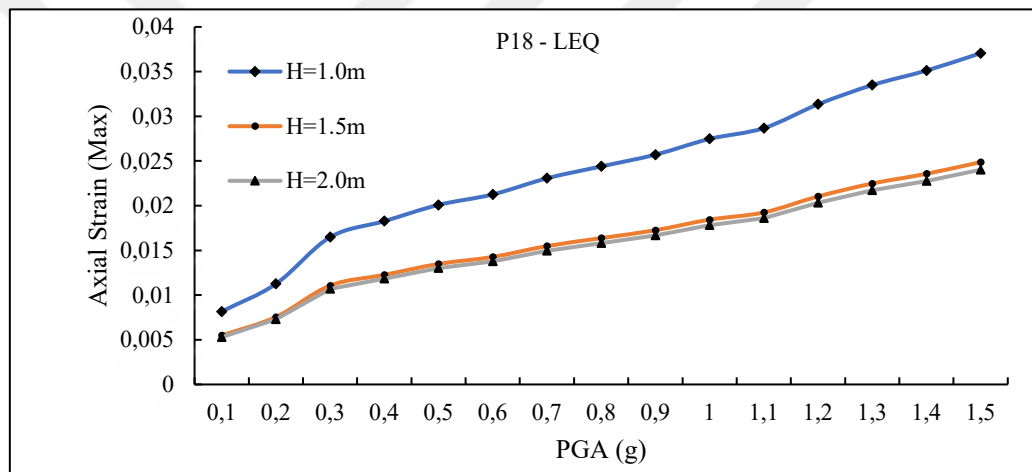


Figure 7.32: Maximum axial strain response of BCGPs for P18 model.

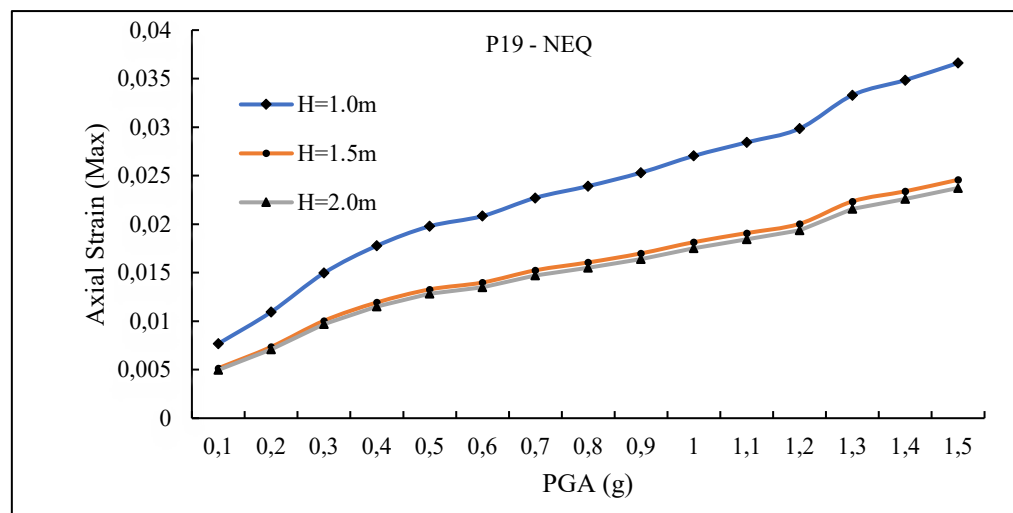


Figure 7.33: Maximum axial strain response of BCGPs for P19 model

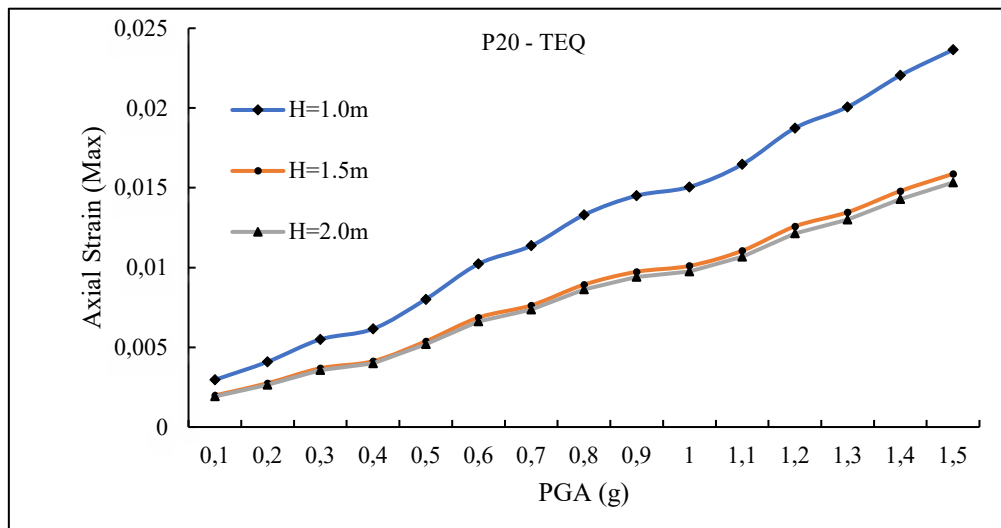


Figure 7.34: Maximum axial strain response of BCGPs for P20 model.

Comparing the accumulated values with the threshold limit values with Table 7.6 and Table 7.7, it is noticed that the selected pipe can face total collapse under assumed conditions at PGA values higher than 1.10g except P20 model. The damage level based on maximum axial strain observed in each model against various PGA (g) values is summarized in Table 7.9.

Table 7.9: Damage levels observed in models P.11-P20.

| Model | Damage States for LS Type Soil |            |           |           |
|-------|--------------------------------|------------|-----------|-----------|
|       | OL (0.01)                      | PL (0.015) | LL (0.02) | CL (0.03) |
| P11   | 0.15g                          | 0.45g      | 0.95g     | 1.25g     |
| P12   | 0.16g                          | 0.25g      | 0.89g     | 1.26g     |
| P13   | 0.14g                          | 0.48g      | 0.97g     | 1.27g     |
| P14   | 0.28g                          | 0.62g      | 1.15g     | 1.42g     |
| P15   | 0.13g                          | 0.29g      | 0.99g     | 1.28g     |
| P16   | 0.18g                          | 0.46g      | 0.96g     | 1.43g     |
| P17   | 0.52g                          | 0.68g      | 1.01g     | 1.49g     |
| P18   | 0.12g                          | 0.22g      | 0.43g     | 1.10g     |
| P19   | 0.11g                          | 0.23g      | 0.47g     | 1.12g     |
| P20   | 0.51g                          | 0.88g      | 1.25g     | -         |

From Table 7.9 for LS type, OL state is observed at approx. 0.11g – 0.18g in P.11 - P.12 - P.13 - P.15 - P.16 - P.18 and P.19 models, in P.14 model at 0.28g and in

P.17 - P.20 models at approximately 0.51g and 0.52g respectively. Similarly, PL damage state is noticed at approx. 0.45g – 0.68g in P.11 - P.13 - P.14 – P.16 and P.17 models, and in P.12 - P.15 – P.18 – P.19 models at approximately 0.22g – 0.29g, in P.20 model at 0.88g respectively. LL damage state is noticed between 0.89g – 1.15g in P.11 - P.12 - P.13 - P.14 - P.15 and P.17 models and at 0.43g-0.47g in P.18 and P.19 models and at 1.25g in P.20 model. Furthermore, CL level is observed in 9 models (P.11-P.19) at 1.10g- 1.49g except P.20 model of this study.

### **7.5.3. Effect of Changing Pipe Boundary End Conditions**

During seismic event, buried pipe moves along with the surrounding soil. Therefore, the pipe end conditions play an import role to consider during designing. Three pipe boundary end condition models such as pinned-pinned, pinned-fixed and fixed-fixed are presented in Table 7.5. The concept behind this selection is due to the infinite pipe length assumption as pinned-pinned, and finite length as pinned-fixed and fixed-fixed respectively. Pinned-pinned represent the continuous buried pipe with no restrictions at both ends. Pinned-fixed shows that one side of pipe is restricted with gas station or supply building and other side is continuous. Similarly, fixed-fixed present the restriction on both sides due to the building availability on the end of each side of the pipe. The maximum axial strain effect for considered boundary conditions is presented in Figure 7.35- 7.44. It is noticed that the overall axial strain is maximum in the pinned-pinned and pinned-fixed end conditions of the pipe than fixed-fixed. Therefore, pinned-pinned end conditions of pipe at PGA level intensities more than 1.0g is vulnerable to damage first and special precautions should be employed to reduce this likelihood phenomenon.

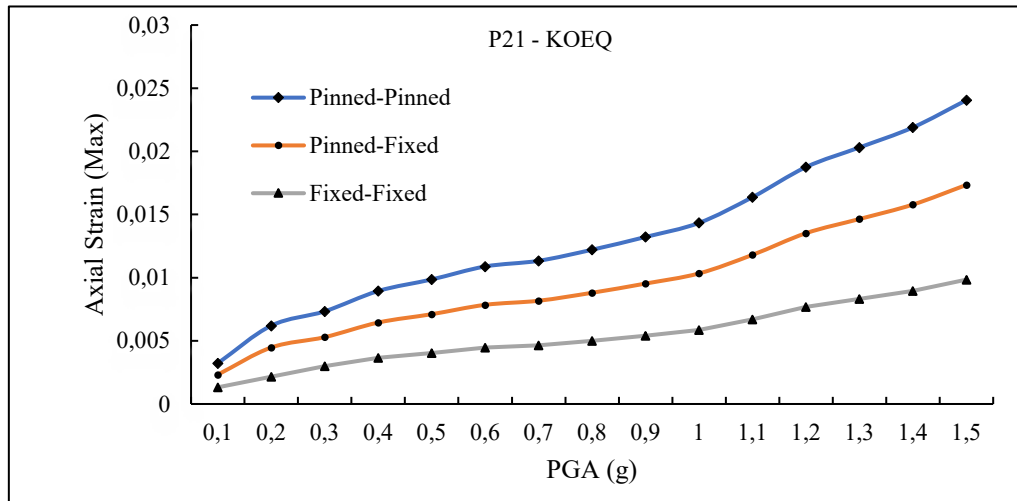


Figure 7.35: Maximum axial strain response of BCGPs for P21 model.

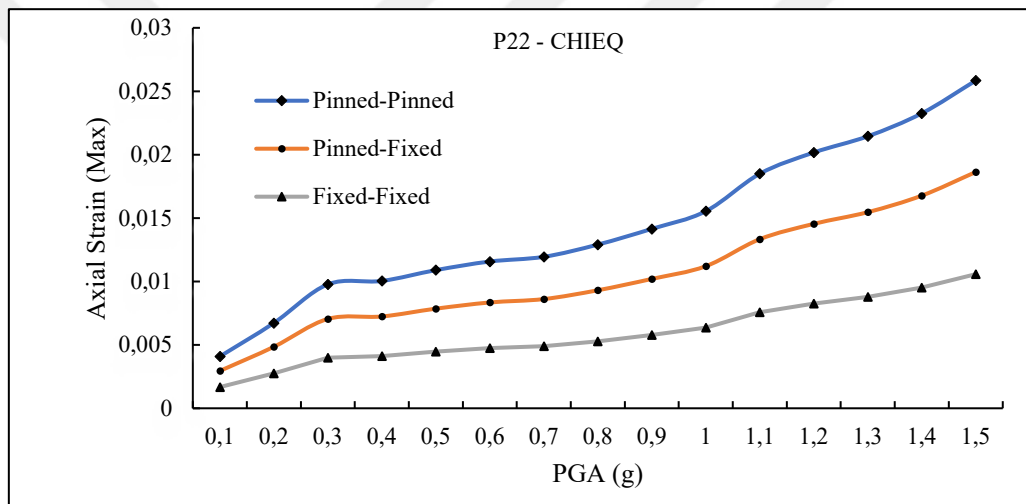


Figure 7.36: Maximum axial strain response of BCGPs for P22 model.

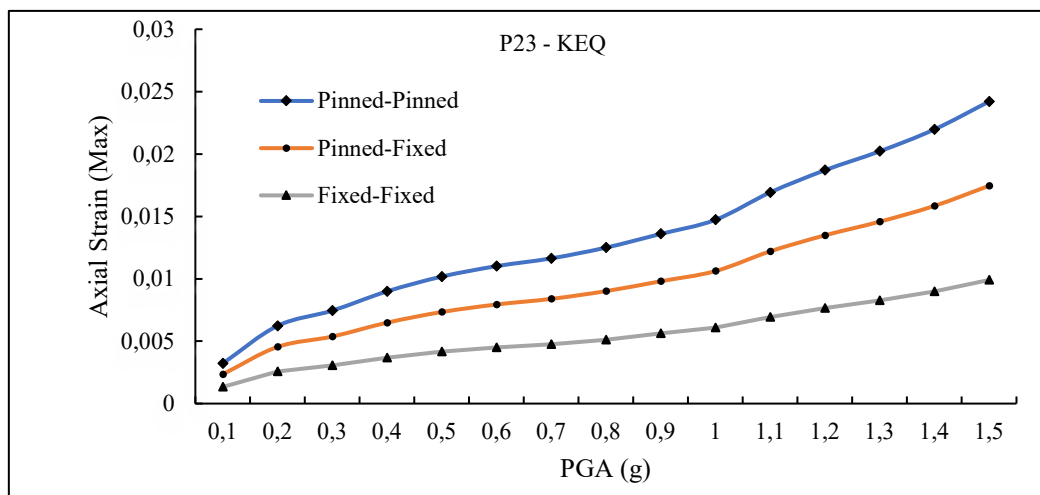


Figure 7.37: Maximum axial strain response of BCGPs for P23 model.

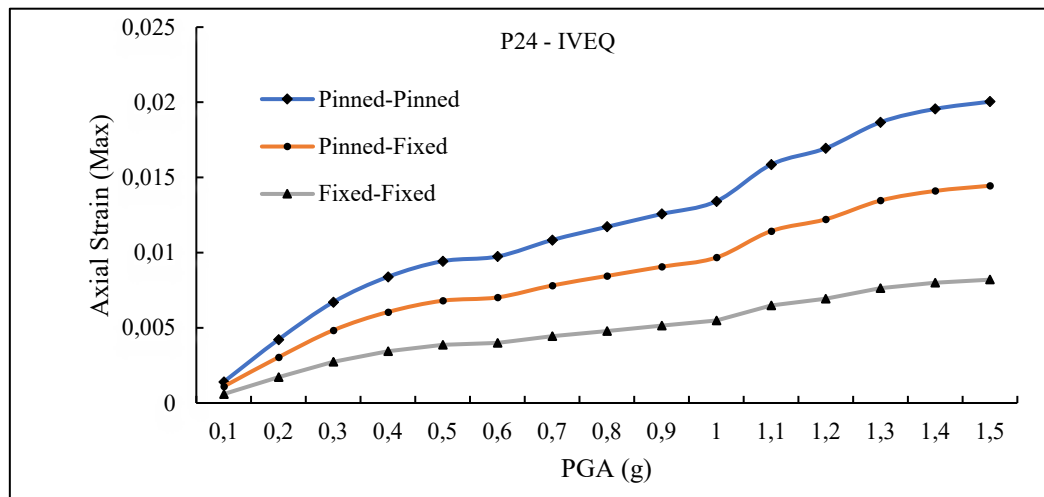


Figure 7.38: Maximum axial strain response of BCGPs for P24 model.

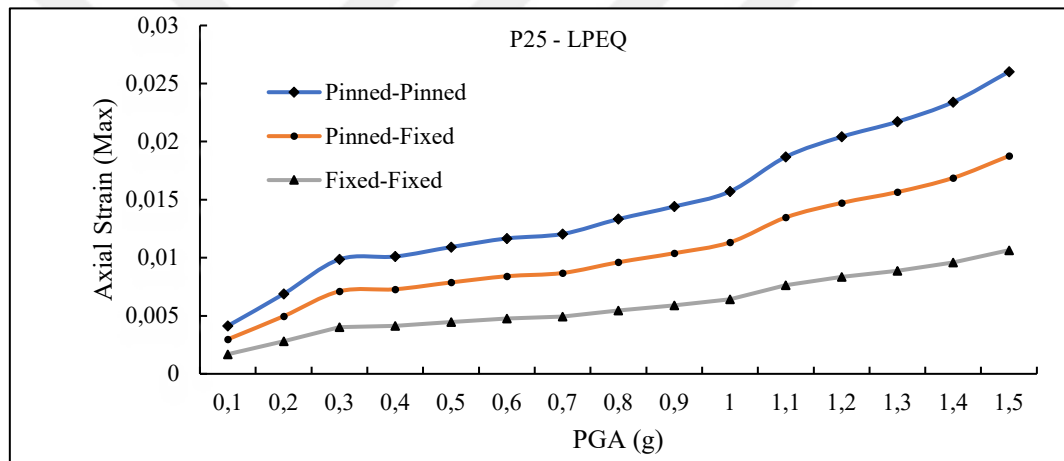


Figure 7.39: Maximum axial strain response of BCGPs for P25 model.

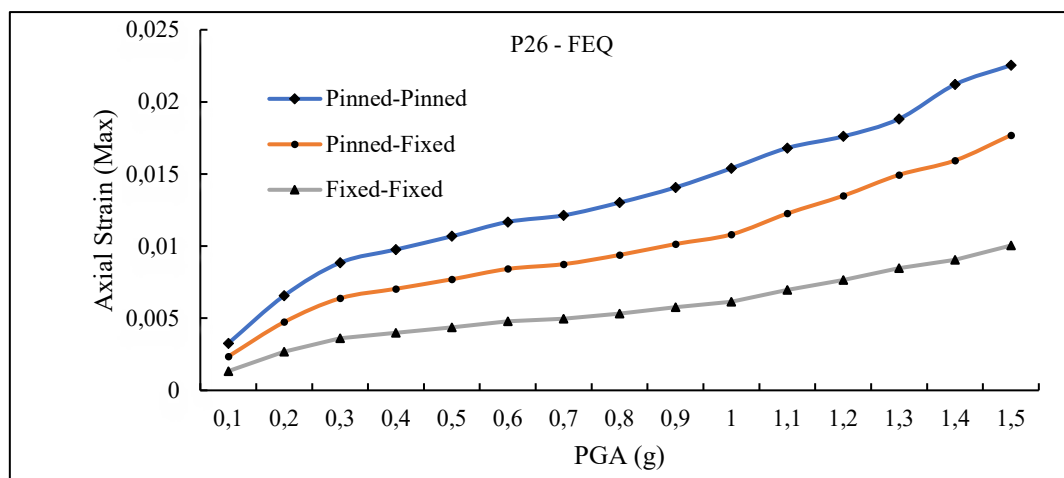


Figure 7.40: Maximum axial strain response of BCGPs for P26 model.

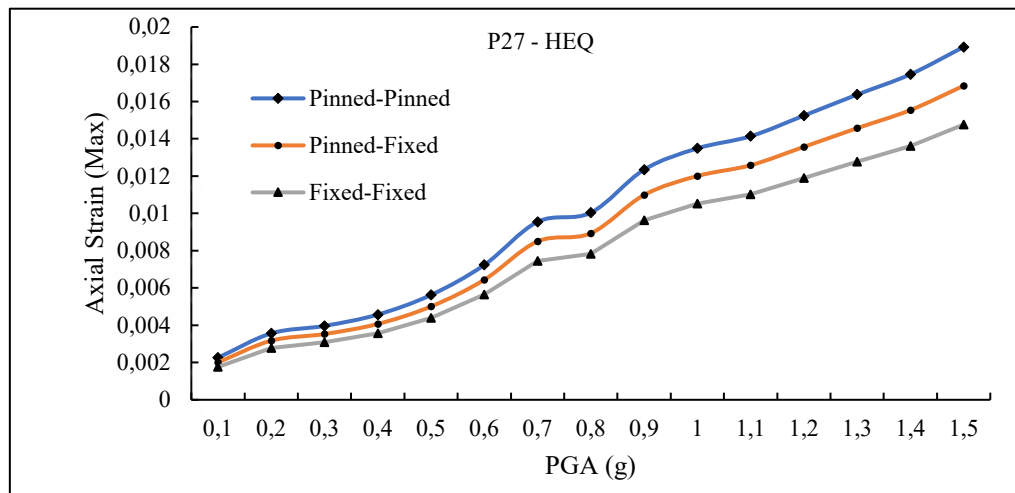


Figure 7.41: Maximum axial strain response of BCGPs for P27 model.

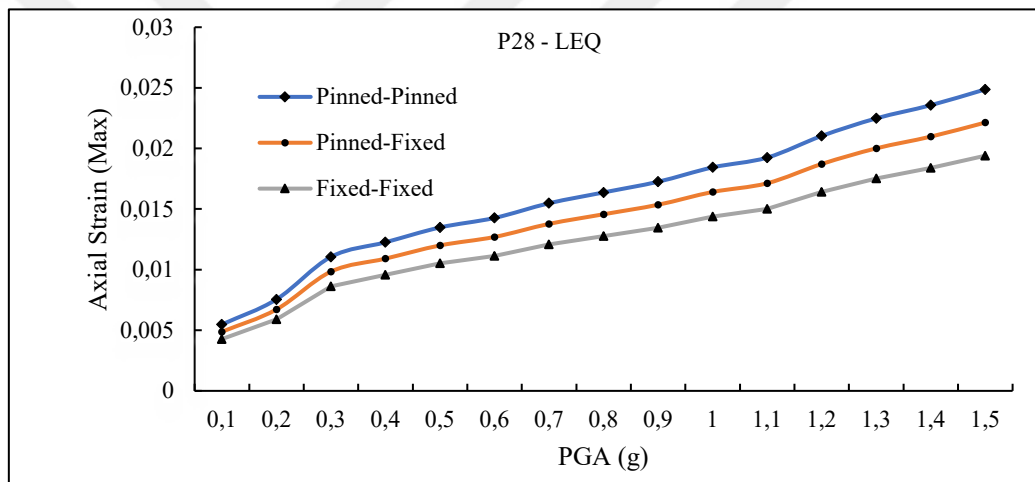


Figure 7.42: Maximum axial strain response of BCGPs for P28 model.

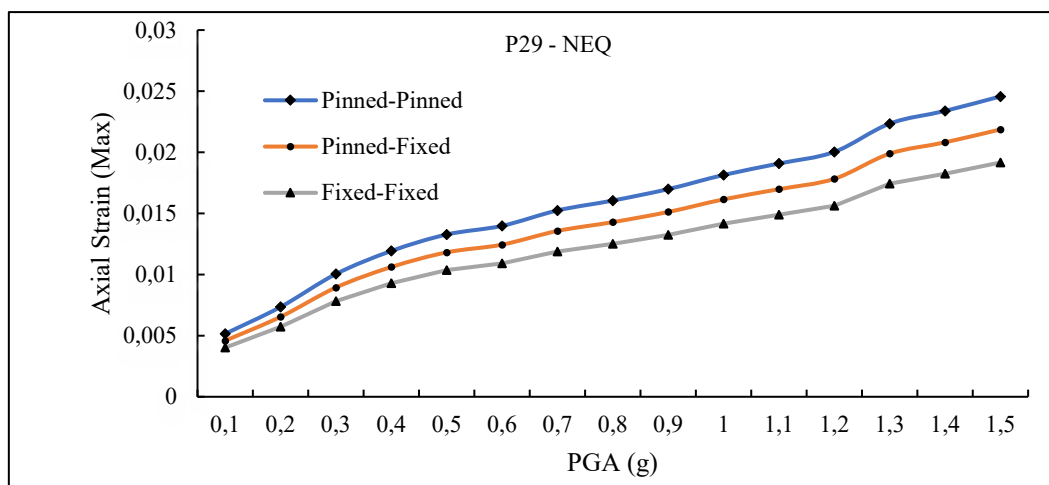


Figure 7.43: Maximum axial strain response of BCGPs for P29 model.

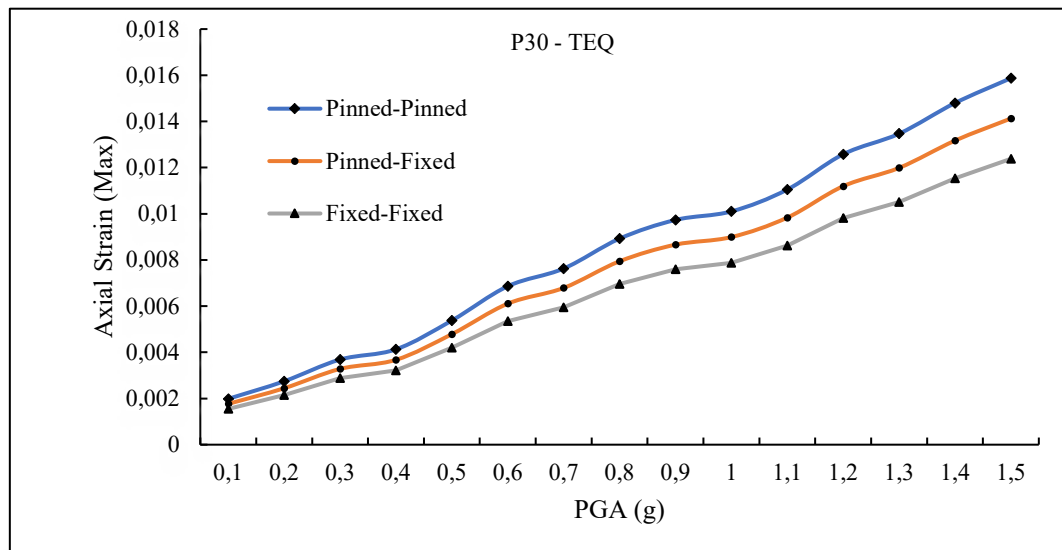


Figure 7.44: Maximum axial strain response of BCGPs for P30 model.

Comparing the accumulated values with the threshold limit values with Table 7.6 and Table 7.7, it is noticed that the selected pipe can face total collapse under assumed conditions. The damage level based on maximum axial strain observed in each model against various PGA (g) values is summarized in Table 7.10.

Table 7.10: Damage levels observed in models P.11-P20.

| Model | Damage States for LS Type Soil |            |           |           |
|-------|--------------------------------|------------|-----------|-----------|
|       | OL (0.01)                      | PL (0.015) | LL (0.02) | CL (0.03) |
| P21   | 0.34g                          | 1.05g      | 1.35g     | -         |
| P22   | 0.25g                          | 0.95g      | 1.50g     | -         |
| P23   | 0.45g                          | 0.97g      | 1.28g     | -         |
| P24   | 0.48g                          | 1.08g      | 1.36g     | -         |
| P25   | 0.23g                          | 0.76g      | 1.22g     | -         |
| P26   | 0.33g                          | 1.09g      | 1.45g     | -         |
| P27   | 0.79g                          | 1.36g      | 1.49g     | -         |
| P28   | 0.28g                          | 0.78g      | 1.27g     | -         |
| P29   | 0.29g                          | 0.80g      | 1.15g     | -         |
| P30   | 0.91g                          | 1.43g      | -         | -         |

From Table 7.10 for LS type, OL state is observed at approx. 0.23g – 0.48g in P.21 - P.22 - P.23 - P.24 - P.25 - P.26 - P.28 and P.29 models, and in P.27 - P.30 models at approximately 0.79g and 0.91g respectively. Similarly, PL damage state is

noticed at approx. 0.76g – 1.08g in P.21 - P.22 - P.23 - P.24 - P.25 – P.26 - P.28 and P.29 models, and in P.27 models at approximately 1.36g, and in P.30 model at 1.43g respectively. LL damage state is noticed at  $> 1.15g$  in 9 models such as P.21 - P.22 - P.23 - P.24 - P.25 – P.26 – P.27 – P.28 and P.29 models and no LL damage in P.30 model. Furthermore, CL level is not observed in all 10 models (P.21-P.30) of this study.

## **7.6. Output Comparison with ZeusNL Program**

The Zeus-NL program is developed for 2D and 3D nonlinear analysis of superstructures and substructures. In current study, buried pipe is modelled as line element. The soil-pipe interaction is modelled using nonlinear Winkler foundation model, in which soil springs are used to denote 3D joint elements. The soil spring properties are computed using ALA's equations (ALA, 2001) for axial, vertical and transverse directions and the procedure is explained in Chapter-4 of this thesis. Figure 7.45 represent the graphical view of buried pipe model with Winkler foundation aspects.

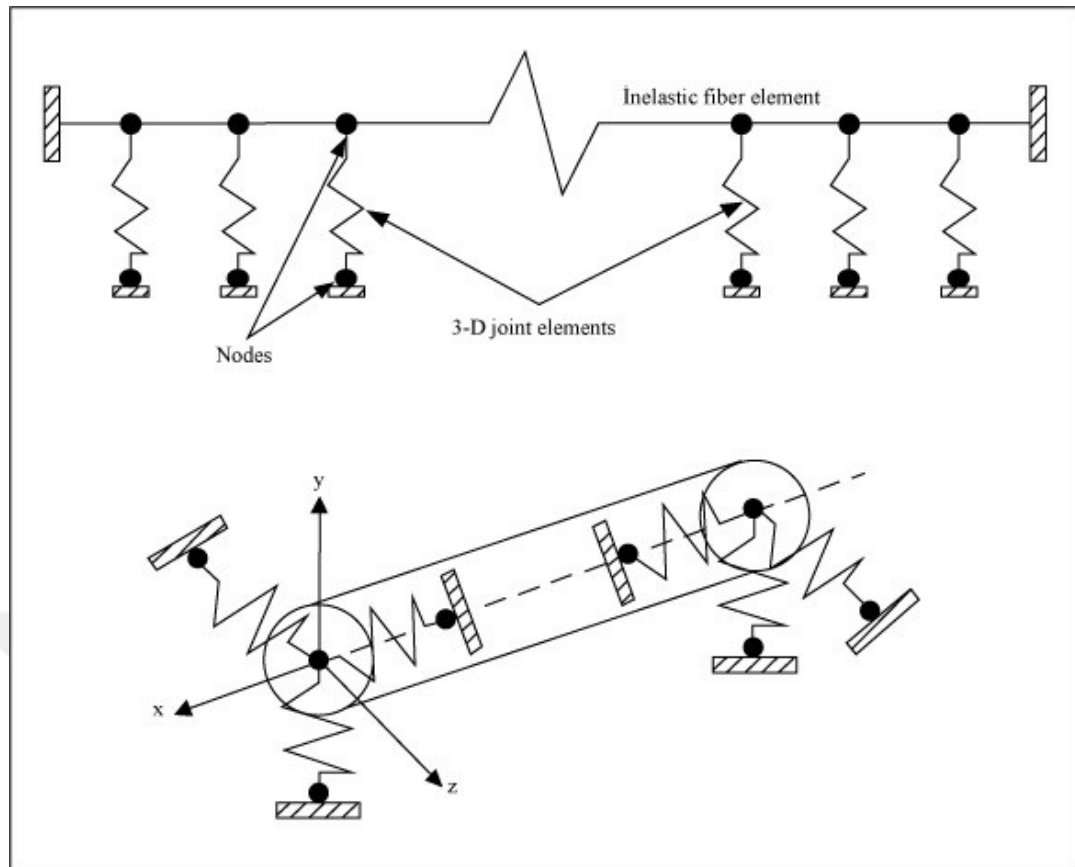


Figure 7.45: The graphical view of buried pipe as beam on nonlinear Winkler foundation model.

### 7.6.1. Comparison for Change of Backfill Soil

Compatibility of 3D FE program Abaqus results are checked with Zeus-NL and it presents the accuracy and similarity of results in both analysis programs. The output in the form of maximum axial strain obtained by Zeus-NL is slightly lower values than the Abaqus and this can be due to the fact of differences in methods of boundary condition application.

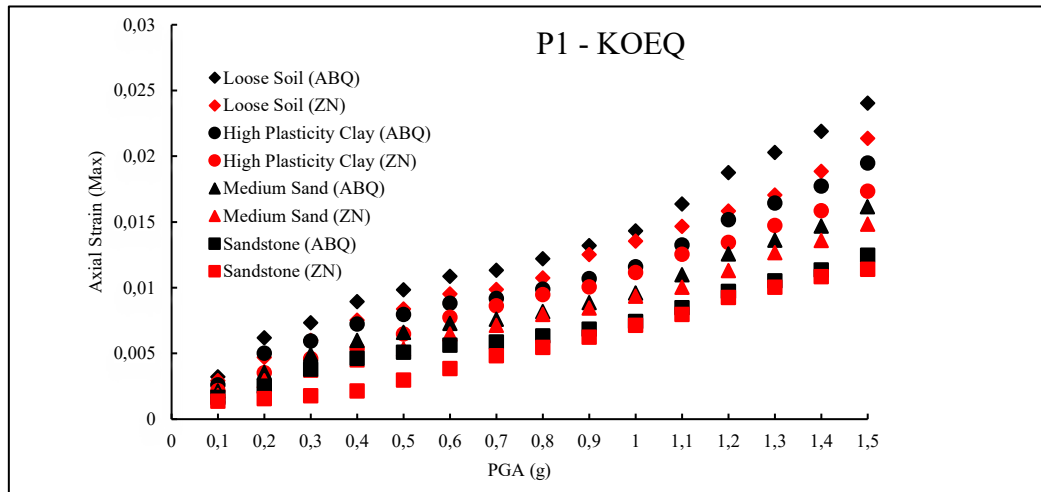


Figure 7.46: Comparative distributions of max. axial strain for KOEQ.

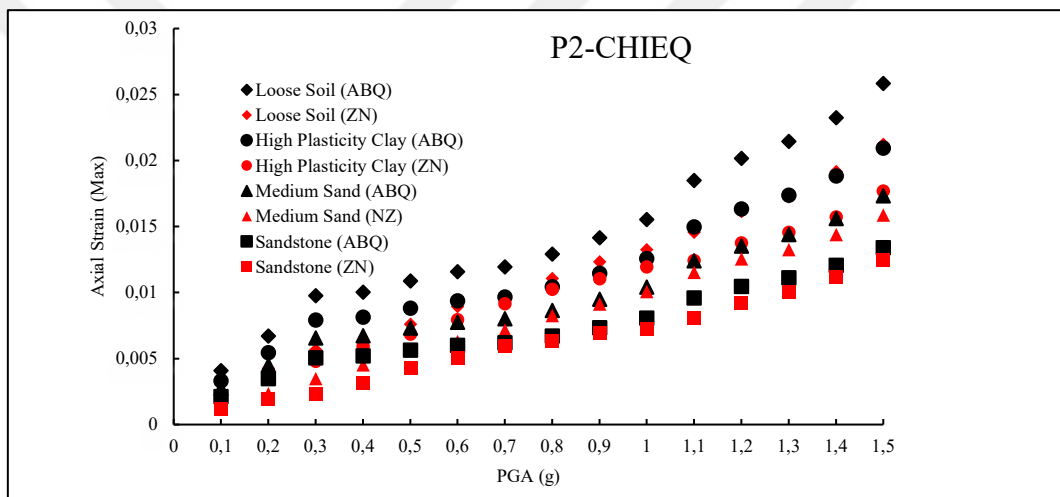


Figure 7.47: Comparative distributions of max. axial strain for CHIEQ.

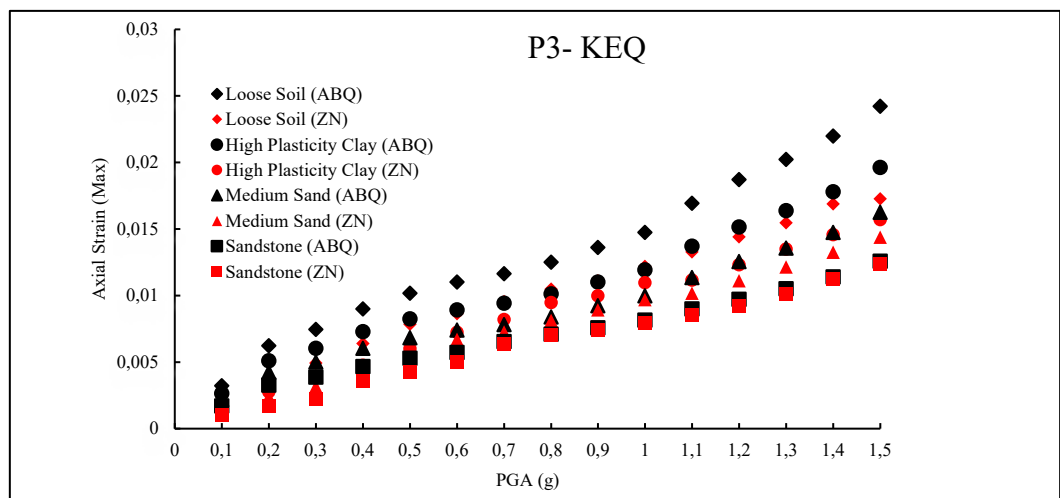


Figure 7.48: Maximum axial strain response of BCGPs for P30 model.

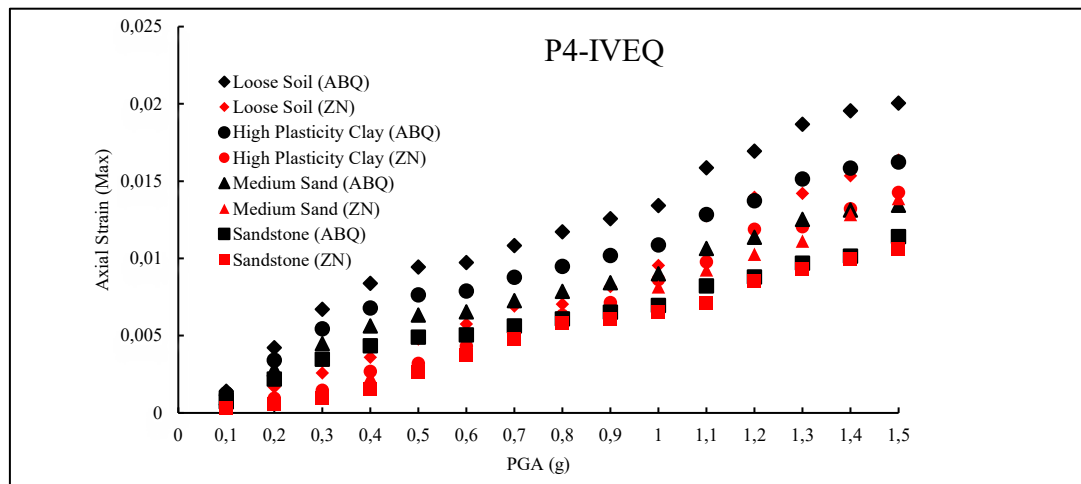


Figure 7.49: Comparative distributions of max. axial strain for IVEQ.

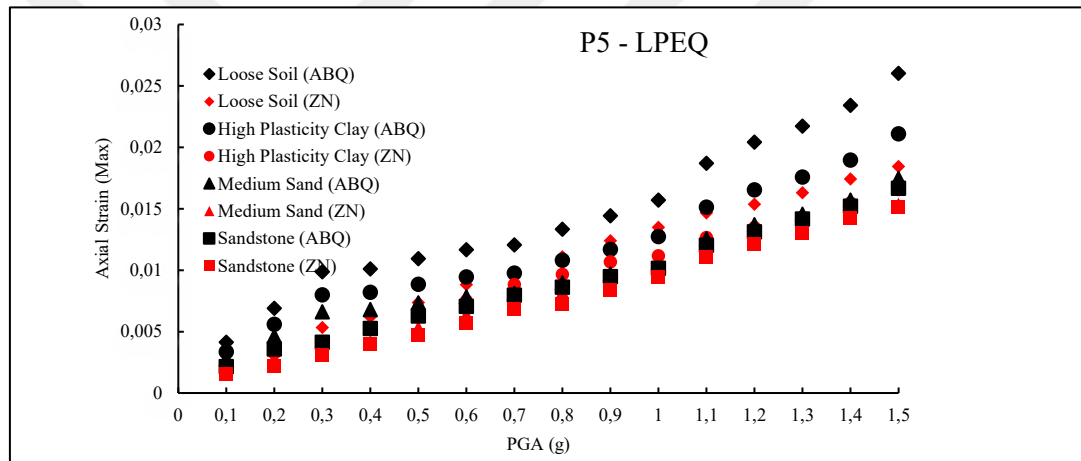


Figure 7.50: Comparative distributions of max. axial strain for LPEQ.

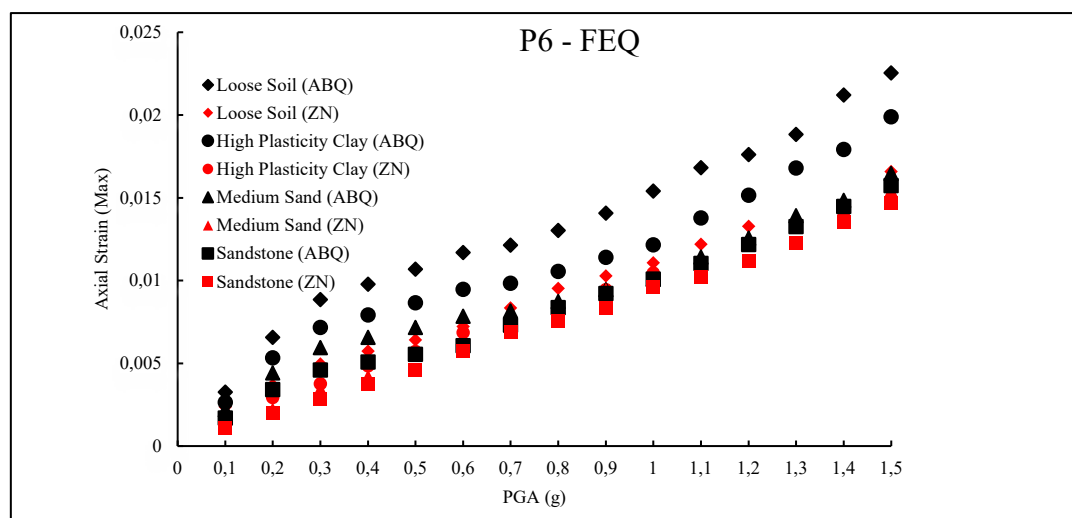


Figure 7.51: Comparative distributions of max. axial strain for FEQ.

From Figures 7.48 – 7.53 (P.1-P.6 models), It is noticed that except for loose soil, the computed maximum axial strain values are close to each other and it verifies the compatibility of both methods for seismic analysis of BCGPs. For loose soil, results obtained by Abaqus analysis are on upward trend and this can be since in Abaqus 4 layers are assumed for the analysis, however in Zeus-NL, the soil is uniform for each analysis.

### 7.6.2. Comparison for Change of Pipe Burial Depth

The comparison of FE analysis methods and fiber element analysis method for change in burial depth of BCGP is presented in Figures 7.52 – 7.57 for the models of P.11-P.16.

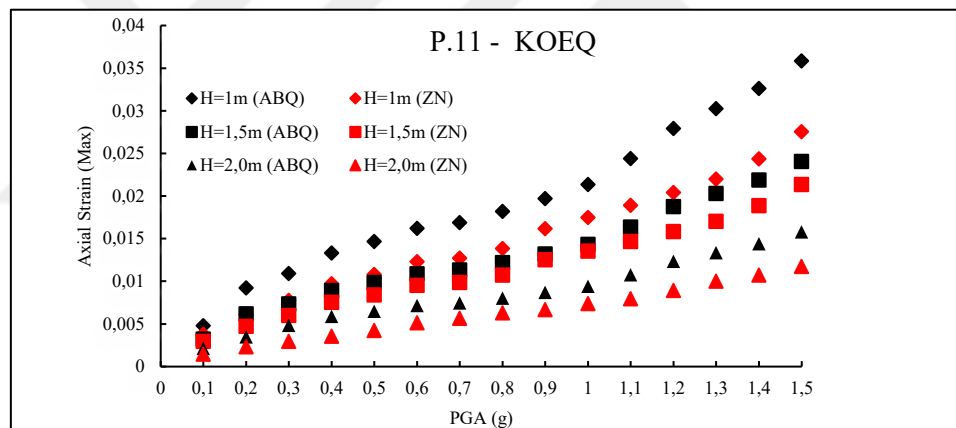


Figure 7.52: Comparative distributions of max. axial strain for LPEQ.

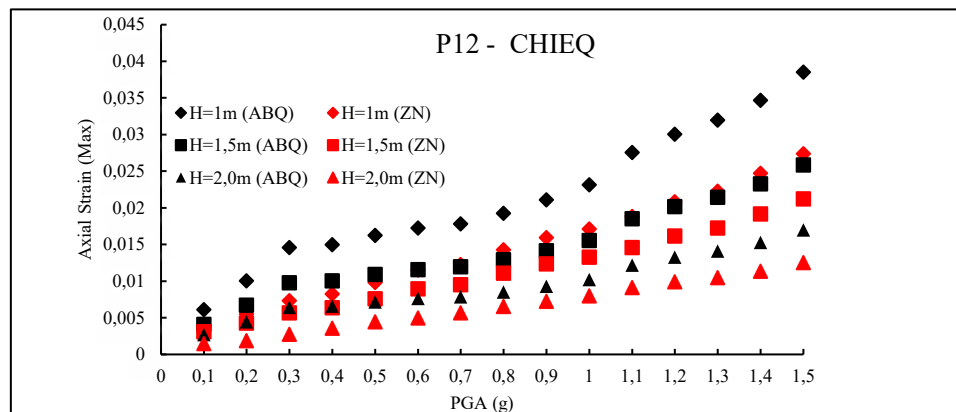


Figure 7.53: Comparative distributions of max. axial strain for CHIEQ.

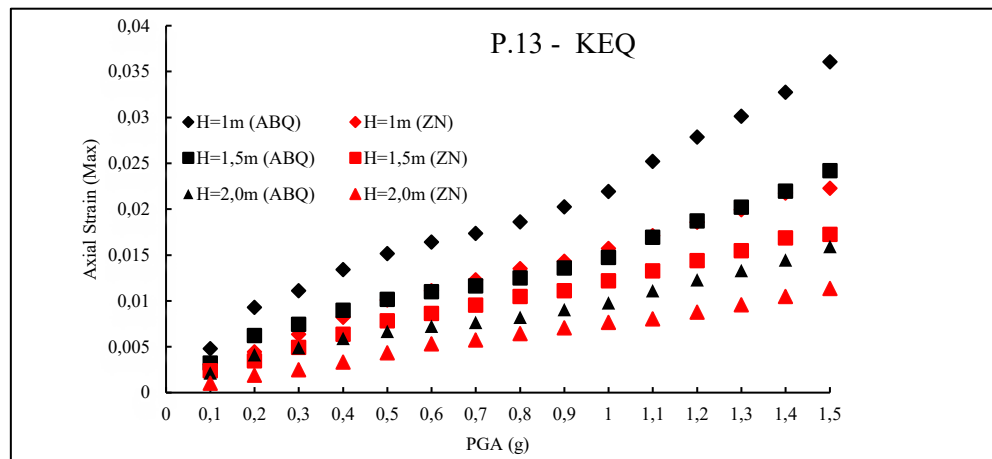


Figure 7.54: Comparative distributions of max. axial strain for KEQ.

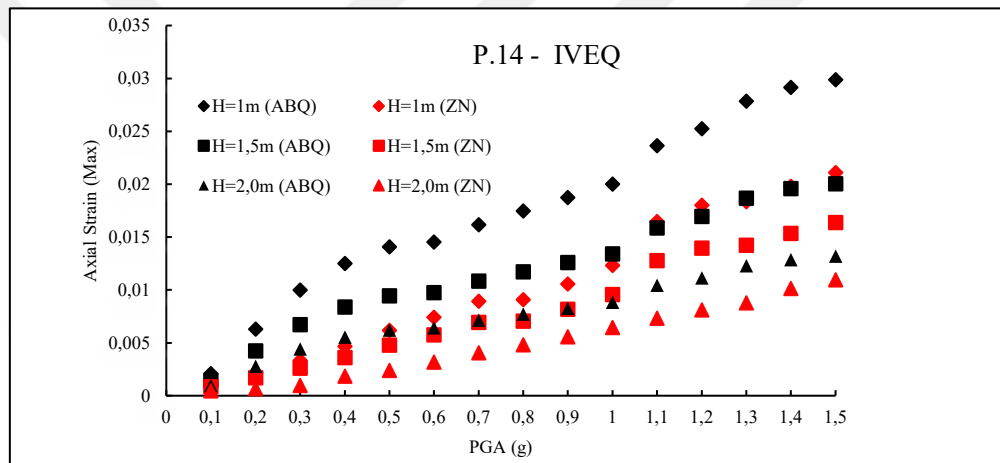


Figure 7.55: Comparative distributions of max. axial strain for IVEQ.

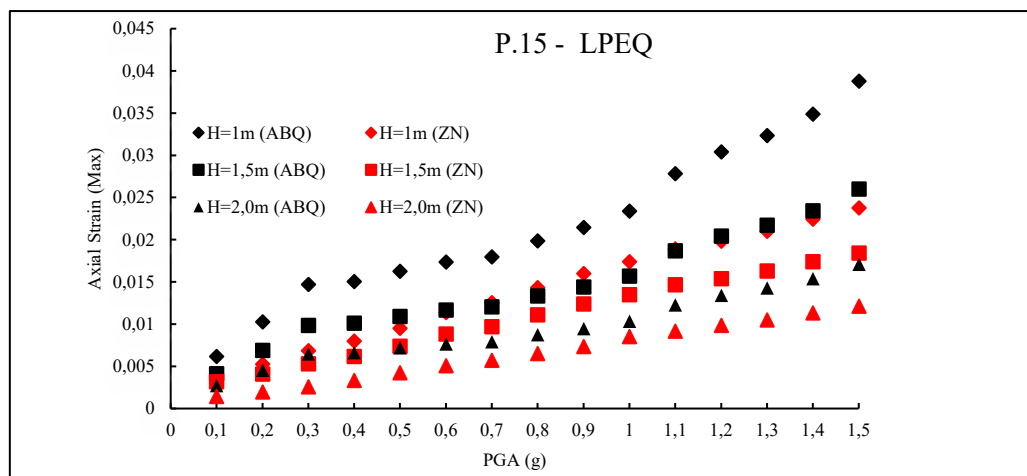


Figure 7.56: Comparative distributions of max. axial strain for LPEQ.

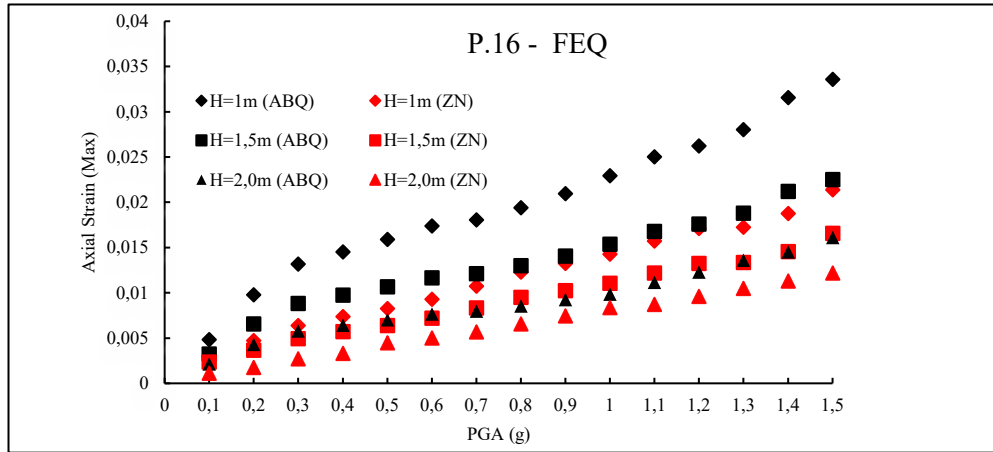


Figure 7.57: Comparative distributions of max. axial strain for LPEQ.

From Figures 7.52 – 7.57 for the models of P.11-P.16, It is noticed that except for 1m burial depth, the computed maximum axial strain values are close to each other and it verifies the compatibility of both methods for seismic analysis of BCGPs. For 1m burial depth, the results obtained by Abaqus analysis are on upward trend and this can be since in Abaqus 4 layers of soil are taken for the analysis and upper layer is assumed as backfill soil layer of loose soil properties, however in Zeus-NL, the soil is uniform for each analysis.

### 7.6.3. Comparison for Change of Pipe Boundary End Condition

The comparison of FE analysis methods and fiber element analysis method for change in pipe boundary end conditions is presented in Figures 7.58 – 7.63 for the models of P.21-P.26.

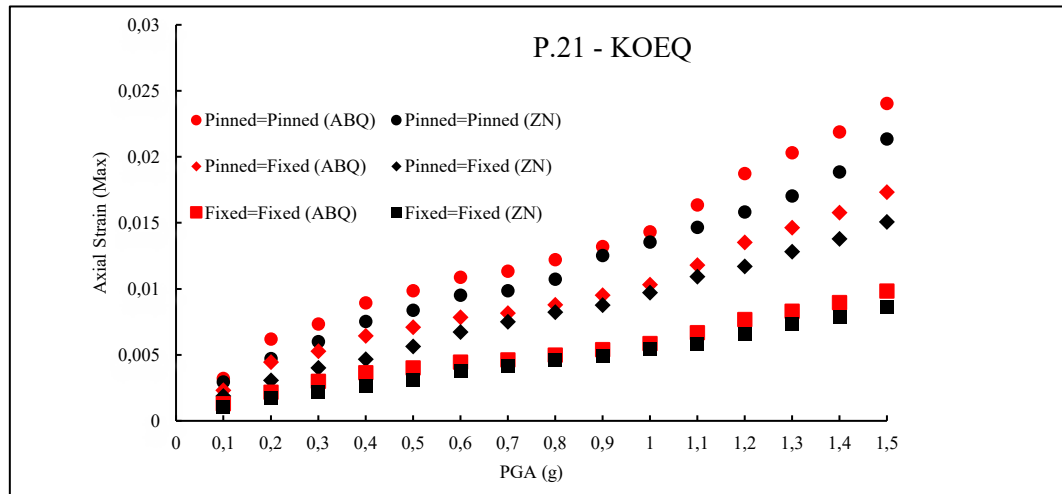


Figure 7.58: Comparative distributions of max. axial strain for KOEQ.

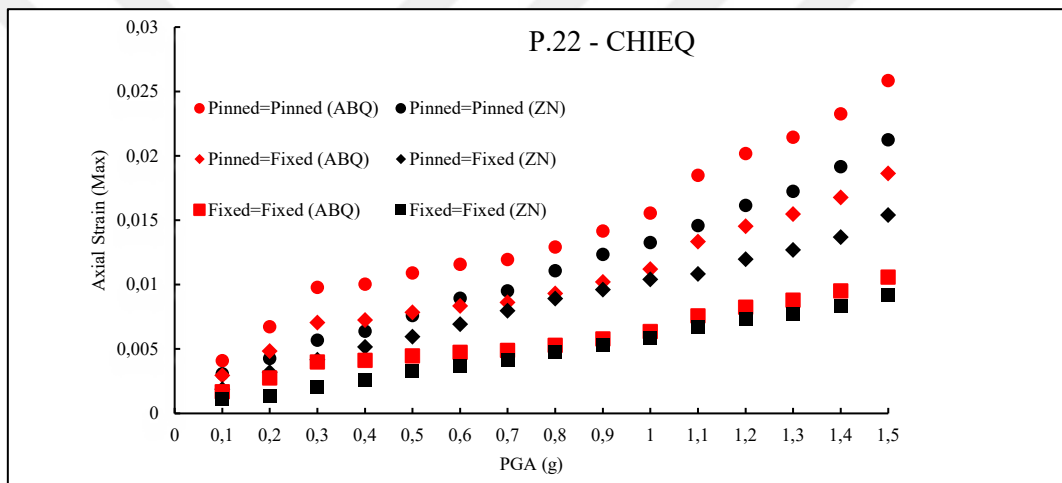


Figure 7.59: Comparative distributions of max. axial strain for CHIEQ.

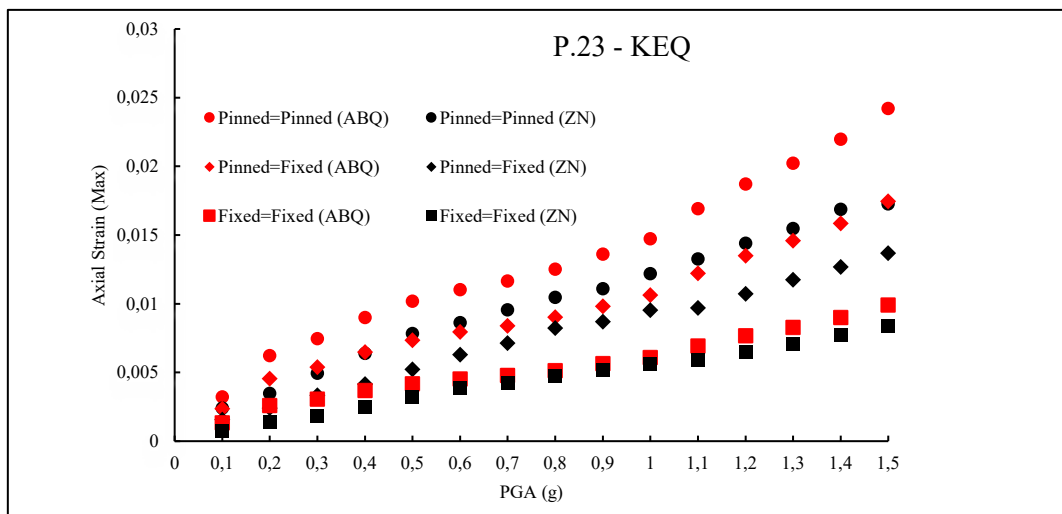


Figure 7.60: Comparative distributions of max. axial strain for KEQ.

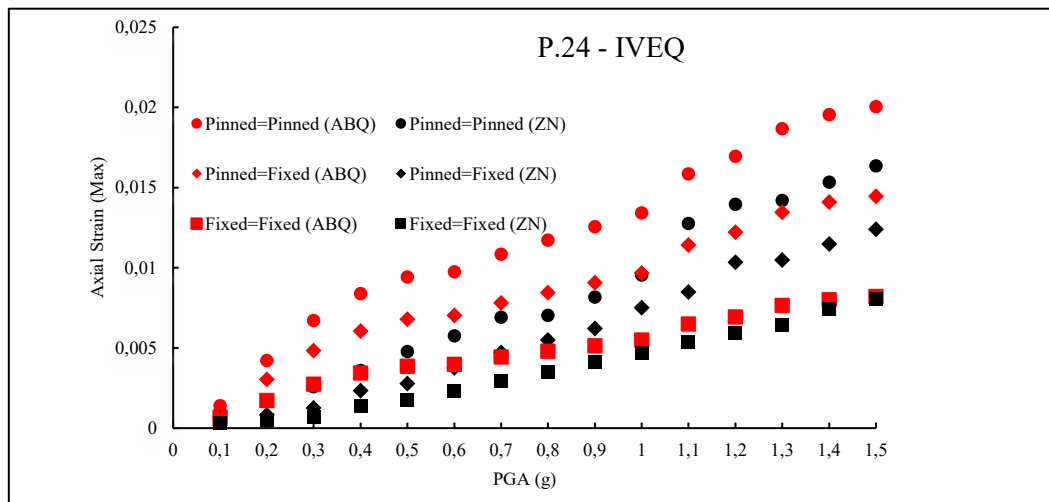


Figure 7.61: Comparative distributions of max. axial strain for IVEQ.

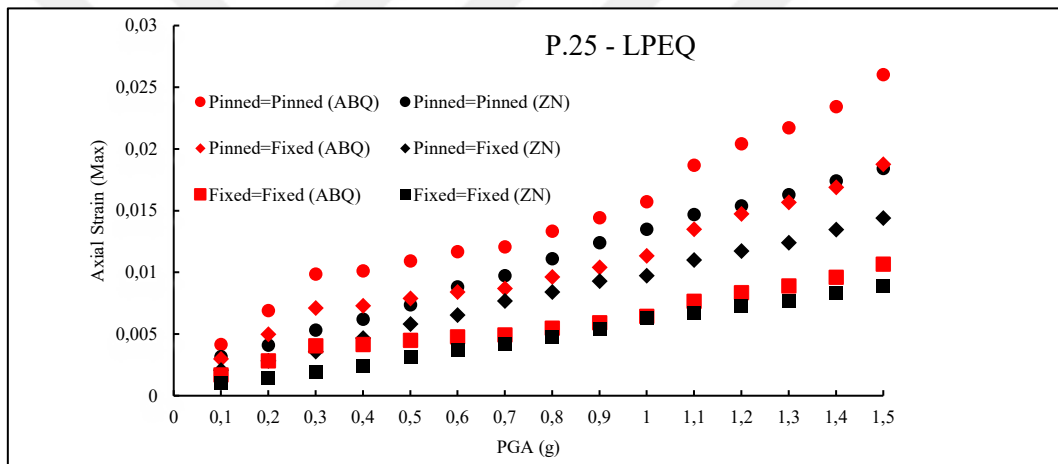


Figure 7.62: Comparative distributions of max. axial strain for LPEQ.

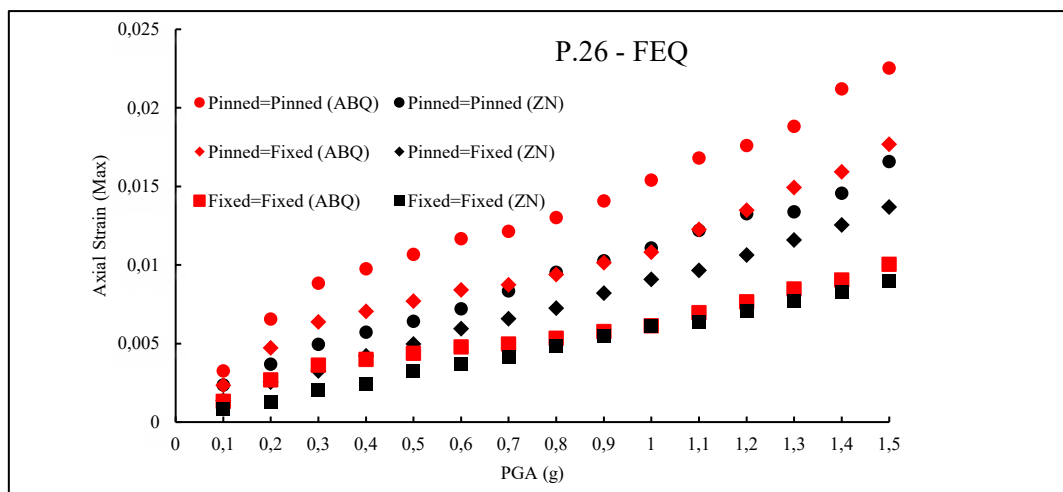


Figure 7.63: Comparative distributions of max. axial strain for LPEQ.

From Figures 7.58 – 7.63 for the models of P.21-P.26, It is noticed that except for pinned-pinned boundary condition, the computed maximum axial strain values are close to each other and it verifies the compatibility of both methods for seismic analysis of BCGPs. For pinned-pinned boundary condition, results obtained by Abaqus analysis are on upward trend and this can be since in Abaqus, the boundary conditions are programmatic designed constant value for the analysis, however in Zeus-NL, the nonlinear Winkler foundation model is adopted.



## **8. HEALTH MONITORING TECHNIQUES FOR BCGPs**

Health monitoring (HM) of a system or its part is recently developed method to ensure the safety of real-life structures [Ali Yeşilyurt et al., 2019]. Generally, two main conditions are kept under considerations after an extreme event like earthquake such as, consistent maintenance and quick repairing. According to Barbian and Beller [Barbian and Beller, 2012], HM is an important factor for BCGPs that should be keep on checking as regular basis. From past studies, various kinds of techniques are provided related to HM of buried pipes such as periodic check, real time-based sensors control and modern developed robotics and smart materials etc., discussed in the following sections in details.

### **8.1. Periodic Inspection Method**

The most widely used method for monitoring of buried pipes is the periodic sensing method. In this method, pipeline inspection gauges (pigs) are installed on the pipes [Liu, 2003] that include sensors, recorders, and data transmitters [Glisic, 2014]. Pigs provide the investigators an easy way to compute the values of different parameters such as strain accumulation, crack pattern control and corrosion of BCGPs as shown in Figure 8a. Various in-line inspection sensors are available commercially around the world [Bradshaw et al., 2009]. For evaluation of parameters like metal loss and crack pattern, the most widely used sensors are the ultrasound-based sensors [Kobayashi et al., 1999], [Rose, 1999], [Towfighi et al., 2002]. Other sensors used for corrosion inspection of pipe wall are the magnetic flux leakage (MFL) and remote field eddy current (RFEC) sensors [Najafi, 2004]. The usage of MFL sensor has been well explained in Figure 8.1 [Barbian and Beller, 2012].

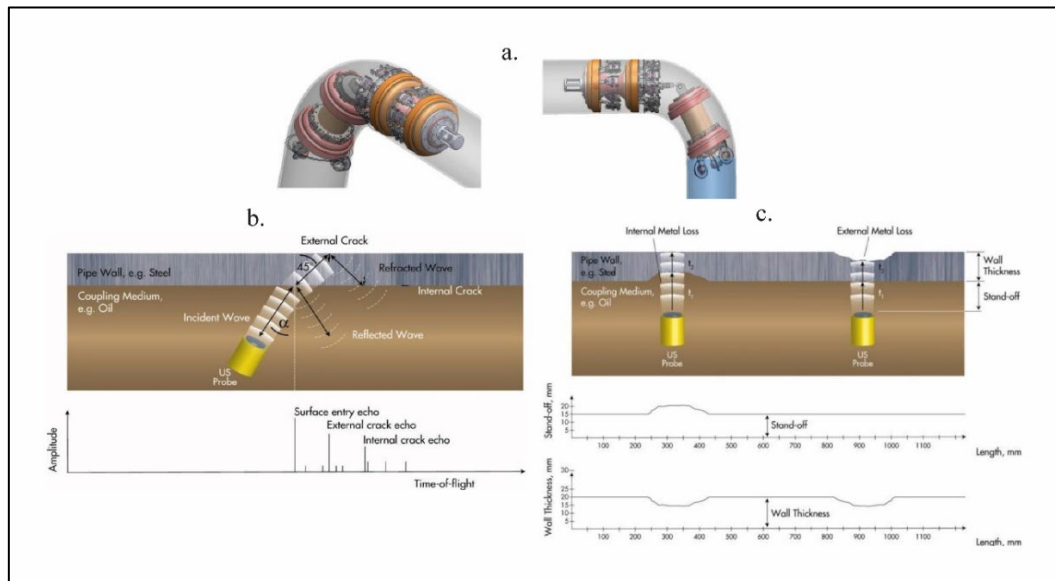


Figure 8.1: Periodic sensing a). isometric and lateral view of sensor with pigs b). ultrasound-based sensor c). magnetic flux leakage (MFL).

## 8.2. Real Time Sensing Methods

Three types of real-time based methods are currently available [Glisic, 2014] that provide a permanent solution to the key challenges of pipelines leakage detections and are discussed in following sections.

### 8.2.1. Wireless Technology (WT) Methods

To avoid the excessive cost of wires for observation of leakage in large structures, WT method has been developed [Straser and Kiremidjian, 1998], [Lynch, 2002], [Spencer et al., 2004], [Lynch et al., 2006]. Two types of problems were reported for assessment of damage using wireless technology a. on-site power capacity and b. attenuation of signals through various ground conditions. On-site power demand can be fulfilled by on-site power production [Glisic, 2014]. Similarly, Kim et al., [Kim et al., 2010] introduced the radio frequency that minimized the attenuation signal problem. Further, Nwalozie and Azubogu [Nwalozie and Azubogu, 2014] presented micro electro-mechanical system (MEMS) based sensors to evaluate acceleration on pipe surface.

### 8.2.2. Fiber Optic Sensing (FOS) Method

In this method, light signals are propagated through fibre strands of buried pipes to measure the various parameters like acceleration, pressure, strain, and temperature [Gastineau et al., 2009]. This is an easy method to use as it requires only a simple cable connection that delivers data to the receiving unit and consumes no extra work of installation. Technologies used in FOS are enlisted in Table 8.1.

Table 8.1: FOS techniques and transducers.

| Fiber Optic Sensing Techniques |               |                    |                      |
|--------------------------------|---------------|--------------------|----------------------|
| Measuring Techniques           | Point sensors | Long gauge sensors | Distributed sensors  |
| Transducers                    | EFPI          | IL-C SOFO          | Raleigh Scattering   |
|                                | FBG           |                    | Raman Scattering     |
|                                | MFC           |                    | Brillouin Scattering |

### 8.2.3. Large Area Networks

This method is developed to monitor strain, corrosion, temperature, pressure, and crack pattern in pipelines. Various electronic devices are installed on large area plastic sheets [Someya et al., 2004], [Someya et al., 2008], [Graz et al., 2009], [Arias et al., 2010].

## 8.3. Modern innovative solutions

Loh et al., [Loh et al., 2007], [Loh et al., 2009] introduced 2D strain monitoring technique which is based on skin type chip mounted on carbon monotubes to evaluate damage pattern in buried pipes. Similarly, innovative materials are also commonly used for assessment of crack pattern and strain measurement in buried pipes [Chung, 2003], [Lynch and Hou, 2005]. In addition, there are many modern robotic systems in which gauges are installed on the robots and then it is processed through the pipelines to measure the required parameters. The modern honeybee robotic monitoring device is shown in Figure 8.2.

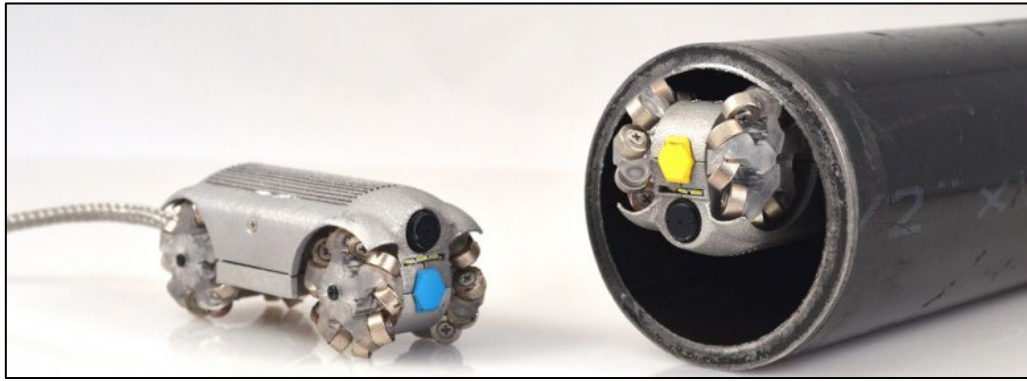


Figure 8.2: Honeybee robotics.

## 8.4. Development of Early Warning System

The procedure adopted and explained before is converted to a monitoring and early warning system. The system is represented in Figure 8.3. It has four phases. First phase is observation point. In this stage, strain accumulated in BGPs due to seismic loading, internal gas pressure and temperature variations are observed. Further, nature of fault movement (NFM) and strength of earthquake based on strong motion (SGM) records are examined. The second phase is the data acquisition and processing unit that transfers the data to wireless communication system. Beidou satellite and 4G technology can be preferred to transfer the unreadable data to remote control receiving unit for display. The third phase is the analysis/display and management system. Display screen are used for exhibition of earthquake early warning (EW), BGPs strain induced, leakage and location required for inspection. The output data can be shared by text messages, email, or smart watches. The fourth and last phase is the alarm system. If any parameter exceeds the threshold limit value (in case of tensile strain limiting value is taken as 0.003), alarm and light color will demonstrate the emergency.

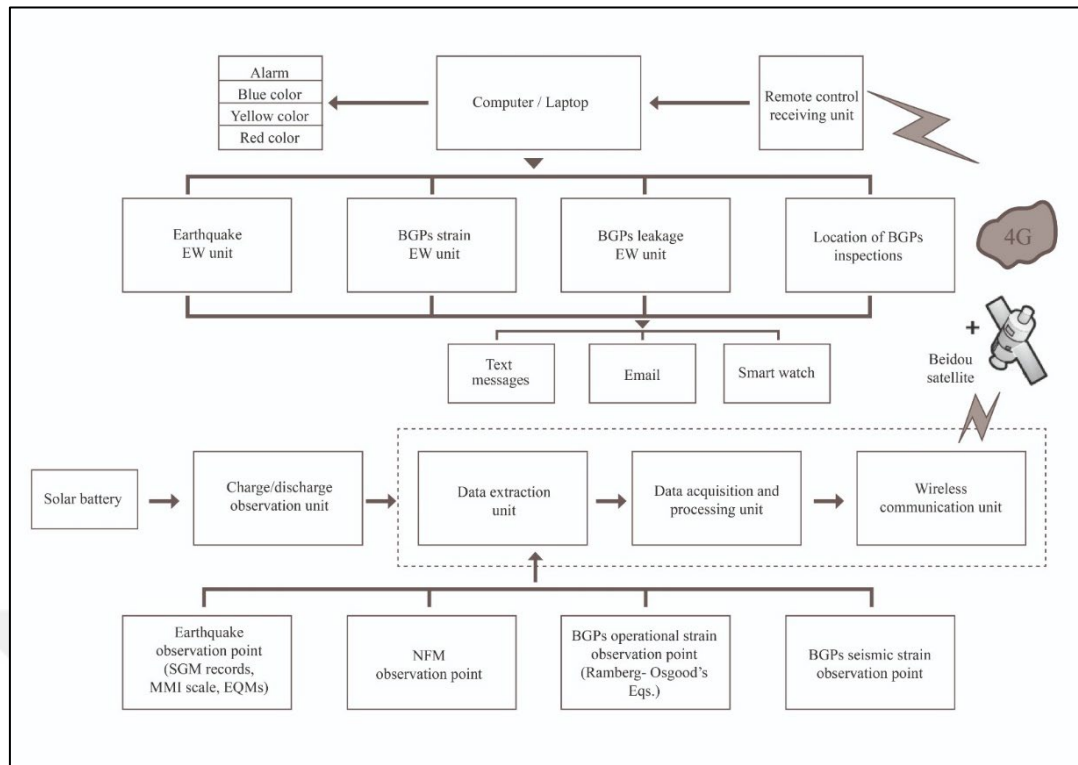


Figure 8.3: Development of EWS for urban risk safety of BCGPs.

## 9. RESULTS AND RECOMMENDATIONS

The current thesis presented a methodology to assess the risk of BCGPs at an urban scale in the aftermath of an earthquake. Following conclusions are derived from the subject thesis.

In seismic hazard analysis, values of IMs made with the SHARE data and the TBDY2018 are quite compatible with each other. However, by comparing EMM data and the TBDY 2018, it is seen that values for Düzce region are larger than TBDY 2018 values. The reason behind this difference can be due to large source values of the regions, different GMPEs application and differentiation in earthquake catalogues. Under the assumed conditions, the BCGPs for current case study is safe for TWP analysis as the computed strain is lower than its threshold limit values. It is noted that the values of seismic strain due to S-wave are less than R-wave as S-waves have larger ground strain coefficient. Pipe class important factor ( $I_p$ ) is directly proportional to the failure mechanism. The lower the  $I_p$  values means the less damage to the buried pipe. It is examined that as the gas pressure is increased to higher levels (0 MPa to 10 MPa), it causes instability in buried continuous pipelines. It is observed that seismic strain induced in the BCGPs is less for very dense soil (C-class) and increases with soil class levels (D-E Classes).

Comparison of different failure modes indicated that local buckling type failure mode is the prominent one in previous major earthquakes. Under the assumed conditions, the BCGPs operated safely against lateral spreading PGD hazard. Length of PGD zones is directly proportional to the failure of BCGPs. Similarly, width of PGD zone is inversely proportional to the accumulation of seismic strains. Under the assumed conditions, the BCGPs operated safely against liquefaction PGD hazard adopted as case study in the thesis. Under the assumed conditions, BCGPs for current study showed instability during analysis for fault crossing (strike-slip fault crossing). It is observed that as the magnitude of earthquake is increased the strain value is directly increased accordingly. Further, a peak increase is observed as the magnitude is increased from 6.5 to 7.0. However, there is slight increment between magnitudes of 6.0 to 6.5 in comparison to the higher levels. The relationship of angle between pipe and fault plane are examined in this study. Results revealed that the value of angle between these two parameters should be kept low to avoid the damage during the

earthquakes. The effect of dip angle is also analyzed to check the contribution in seismic designing of buried pipelines. It is observed that there is no significant change in values of dip angle for different pipe segments. However, pipe segments nearby to the fault crossing are prone to damage higher than the longer distances.

The risk methodology is comprised of a Monte Carlo simulation-based practice; two modern, sequence based, 3-phase geotechnical approach for (i) susceptibility, (ii) triggering, and (iii) PGD. The appliance to the current case study verified the consistency and computational proficiency of the approach. It is observed that geometric, physical, and mechanical properties of both pipelines and soil can influence their vulnerability. Cast iron pipes of small diameters with cement- based connection (A2) perform poorly under corrosive soil condition than all other combinations. Similarly, welded steel pipes of small diameters with screwed and riveted connection (A10-A11) also perform poorly in all soil conditions. However, welded steel pipes of large diameter with lap arc welded connection (A8) perform very well among all other combination.

In 3D FEA method, the results reveal that buried pipes are more prone to damage first in loose soil than high plastic clayey, medium sand and sandstone, respectively. It is also noticed that pinned-pinned boundary conditions (BCs) for pipe ends gain higher axial strain than the pinned-fixed, and fixed-fixed BCs. It is also observed that pipes buried at shallow depth are at higher risks to damage first than the deep ones.

The procedure adopted in current thesis provide a simple and preliminary design path to the professionals and the concerned authorities for quick investigation. A structural health monitoring and decision-making mechanism is developed at urban scale. The developed mechanism can be implemented in the scenario and / or real-time. The development of this project will increase the reliability status of this country to a higher level.

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## **BIOGRAPHY**

Muhammad Rizwan Akram completed his undergraduate studies in Civil Engineering at University of Engineering and Technology Taxila, Pakistan in 2012. He started his master's studies in Civil Engineering (Specialization in Structural Engineering) at University of Engineering and Technology Taxila, Pakistan in 2012 and completed it in 2014. In the same year, he started his Ph.D. studies at Civil Engineering of Graduate School of Natural and Applied Sciences at Gebze Technical University. During his Ph.D. studies, he got thorough experience in Urban Earthquake Risk Assessment of Buried Gas Pipelines.



## APPENDICES

### Appendix A: Publications Within the Scope of Thesis Study

Rizwan M., Yesilyurt A., Zulfikar A., (2021), “Seismic Strain Based Design of Buried Gas Pipelines”, 6<sup>th</sup> International Conference on Earthquake Engineering and Seismology, GTÜ Gebze, Turkey, 13-15 October.

Rizwan M., Zulfikar A., (2020), “Identification of Factors Influencing Sustainability of Buried Continuous Pipelines”, Sustainability, 12 (3), 960.

Rizwan M., Yesilyurt A., Zulfikar A., Göktepe F., (2020), “Failure Analysis of Buried Gas Pipelines Crossing Seismic Faults”, Academic Perspective Procedia, Vol.3 Issue.2. 781-790.

Rizwan M., Yesilyurt A., Zulfikar A., Göktepe F., (2020), “Seismic Failure Analysis of Buried Gas Pipelines Crossing Seismic Faults”, 4<sup>th</sup> International Symposium on Natural Hazards and Disaster Management, Bursa Turkey, 29<sup>th</sup> October.

Rizwan M., Yesilyurt A., Zulfikar A., (2019), “Simplified Procedure for Seismic Design Safety of Buried Gas Pipelines: Buried Gas Pipelines Crossing from Duzce to Istanbul City”, 5th International Student Congress at Trakya University, Edirne, Turkey, 6-8 December.

Rizwan M., Yesilyurt A., Zulfikar A., (2019), “Seismic Safety Evaluation of Buried Gas Pipelines Subjected to Longitudinal Permanent Ground Deformation (PGD)”, 5<sup>th</sup> International Conference on Earthquake Engineering and Seismology, METU Ankara, Turkey, 8-11 October.

Rizwan M., Yesilyurt A., Zulfikar A., (2018), “An overview of Buried Gas Pipelines in Turkey and Their Future Safety Against Earthquake Disaster”, Graduate Research Symposium, Gebze Technical University, Turkey, 14-16 May.