

The University of Tokyo
Graduate School of Frontier Sciences
Socio-cultural and Socio-physical Environmental Studies

2005
Master Thesis

**PROBABILITY OF FAILURE OF CONCRETE
RETAINING WALLS DUE TO EARTHQUAKES
IN KANTO AREA**

(関東地方におけるコンクリート擁壁の
地震による破壊確率に関する研究)

August 1, 2005
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Acknowledgments

Despite of my previous background, I became interested in use of reliability and hazard topics for my research, which are enlightened at Kanda-Takada Laboratory. I wish to express my sincere and heartfelt gratitude to my supervisor, Professor Jun Kanda, Institute of Environmental Studies, Graduate School of Frontier Sciences, The University of Tokyo, for his constructive guidance, and precious advices. It would not have been possible to bring out this dissertation in the present form without his constant encouragement and unconditional support throughout this study. I am grateful to my sub-supervisor Associate Professor Huang Guangwei for his great deal of suggestions, and supervision.

I am also thankful to Professor Tsuyoshi Takada for his fruitful suggestions and constructive criticism during each episodes of this study. My gratefulness goes to former Associate Professor Hang Choi who initially supervised me and provided valuable guidance and encouragement. I would like to thank to Dr. Ryoji Iwasaki, Research Associate at Kanda-Takada Laboratory, who provided important information and research equipment. I would also like to thank Mrs Takahashi, lab secretary, for her kind cooperation.

I would also like to thank Associate Prof Taro Uchimura, Department of Civil Engineering, The University of Tokyo, Dr Mikio Futaki, Center for Better Living and Mr Seiichi Onodera, Senior Researcher, Public Works Research Institute for their valuable information and helps.

I am thankful to my tutor Mr Hiroki Kanno for his helps in many ways throughout his stay in the university. Similarly, I cordially thank to labmates- Ms Shoko Okamura, Ms Naomi Hata, Mr Daisuke Soga, Mr Tatuya Ohbuchi, Mr Yong Chul Kim and to all members of Kanda-Takada Laboratory for their instantaneous support and cooperation.

I am also indebted for the help of government officials in Ota-ward and Seibu/Chubu Construction Management Offices in Yokohama for providing important data and information about retaining wall in the respective areas.

I would like to acknowledge ADB-Japan Scholarship Program for the financial assistance to carry out my study in The University of Tokyo.

Finally, I would like to thank to my wife Jyoti and family for their instant inspiration and support.

Tej Prasad Gautam

August 1, 2005

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Chapter 1

Introduction

1.1 General Background

Retaining structures are constructed to protect a slope surface when banking or cutting cannot be conducted in accordance with the standard cross section. They are common in highway and railway embankments, large constructions, individual houses and housing lots. The stability of any construction is dependent on various factors like topography, geology, structural arrangement of construction and work conditions. Now-a-days, additional safety of structure from rare natural and human induced disasters is more widely considered. Such rare disasters include earthquake, wind or typhoon, heavy precipitation, flooding and chemical or some environmental problems. Recently, about 90 % new constructions of retaining wall are of concrete. But most of the old constructions are made up of masonry, stone pavement, or of blocks. All types of retaining walls can be damaged due to earthquakes but scale of damage may be different according to their type, arrangement and some other factors. Damage cases are higher for masonry and dry stone walls in past earthquakes. Similar results were observed in Niigata-Chuestu earthquake by field visit. However, higher number of constructions of concrete retaining wall indicates need of their study for safety. Much attention and researches have been carried out about retaining wall failure concentrated for highways and railways. But many individual houses and housing lots have retaining walls for supporting their buildings and structures which are also important parts to be concerned. If the retaining wall failure of individual houses, housing lots and small scale streets occurs due to any disasters, it may cause up to large scale loss of property and life.

When the safety is concerned in our society, the structures built in our society are to be addressed. These structures are also associated with the safety of our social environment. Moreover, safety of our social environment and built infrastructures include integrated safety of our social and physical environment. When we make efforts to ensure this safety, the safety of slope becomes quite important to our life. Retaining walls are significantly contributing to the safety of such slopes and ultimately to our social environment. The building safety is a common topic to be discussed in many engineering fields from many perspectives. Consequently, the

evaluation of safety is also widely studied for buildings. Unfortunately, safety of such slopes and retaining walls is beyond the focus of research and evaluation which equally contribute to the safety of our physical and social environment. On these backgrounds, I became interested to carry out my research on the evaluation of safety of retaining walls in Kanto area from probabilistic approach. There is unavailability of such research on probabilistic assessment of existing concrete retaining walls although many researches have been done for the designing purpose.

Parts of Yokohama Municipality and Tokyo Metropolitan are chosen for the field survey and data collection about retaining walls. These areas consist of little sloppy and fragile ground condition which may be affected by the problems of retaining wall failures. The strength, age and properties of the concrete retaining walls were assessed by the field survey and they were analyzed from probabilistic approach.

Without reinforcement, a stable slope can be constructed with an inclination angle less than or equal to the internal friction angle of the soil. The friction between the soil and the confined reinforcement keeps the reinforcement from moving during and after construction. For static and dynamic loading conditions, excessive deformations of a reinforced slope can occur when the reinforcement stretches, yields, breaks, or pulls out of the soil. Numerous methods have been developed to design reinforced structures for static loading conditions, but considerably fewer procedures for seismic design are available. Similarly, many researches on designing of retaining walls have been made, but very few on the safety evaluation of existing retaining walls.

1.2. Location of the Study Area

The study area is located in the parts of Tokyo Metropolitan and Yokohama Municipality. It represents mainly southern part of Kanto area, namely Ota ward in Tokyo Metropolitan and other two wards: Hodogaya ward and Naka ward of Yokohama Municipality area (Fig 1.1). Hodogaya Ward has 86,340 households and population total of 204,191 whereas Naka Ward has 72,412 households and 138,977 population. The physiographic feature is characterized by many hillocks

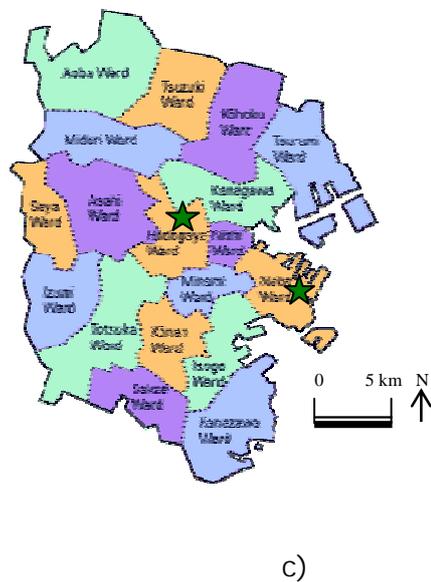
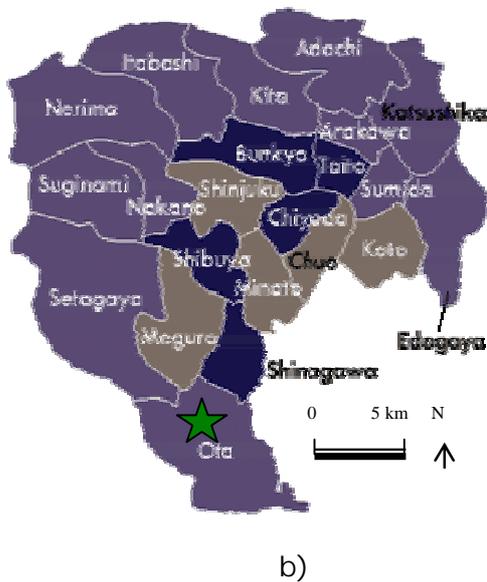
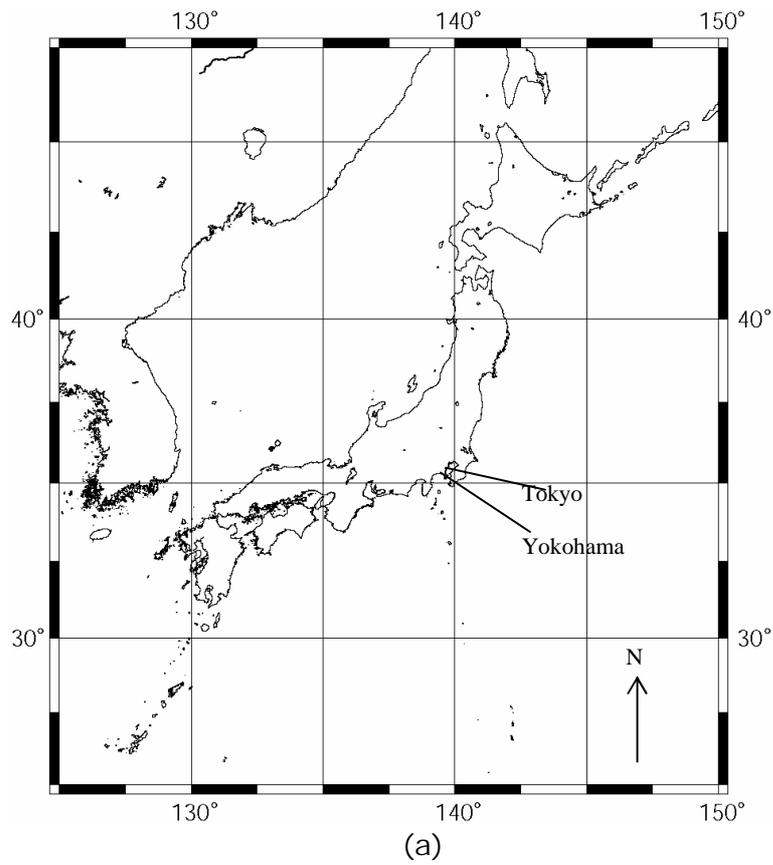


Figure 1.1: a) Location of the Study area in Japan b) Location of Ota Ward in Tokyo Metropolitan c) Location of Hodogaya Ward and Naka Ward in Yokohama Municipality

and valleys. The retaining walls of those areas were observed and studied during the data collection. There are many sloppy areas and hilly terrains where retaining structures are common to construct to hold back the structures on uphill side and to ensure the safety of those structures like individual houses.

1.3. Objectives

The probability of hazard and risk of retaining structures draw attention to make the structures safe and reliable. So, owner of the structure should recognize the condition of probabilistic failure of their structures.

My objectives of this study are as following:

1. Conduct a field survey to find out the strength, age and properties of concrete retaining walls in Kanto area, and find out the relationship between present strength and age of concrete.
2. Estimate the seismic hazard curve of the field site based on the probability of occurrence of earthquakes and find out the probability of failure of concrete retaining walls due to earthquakes.

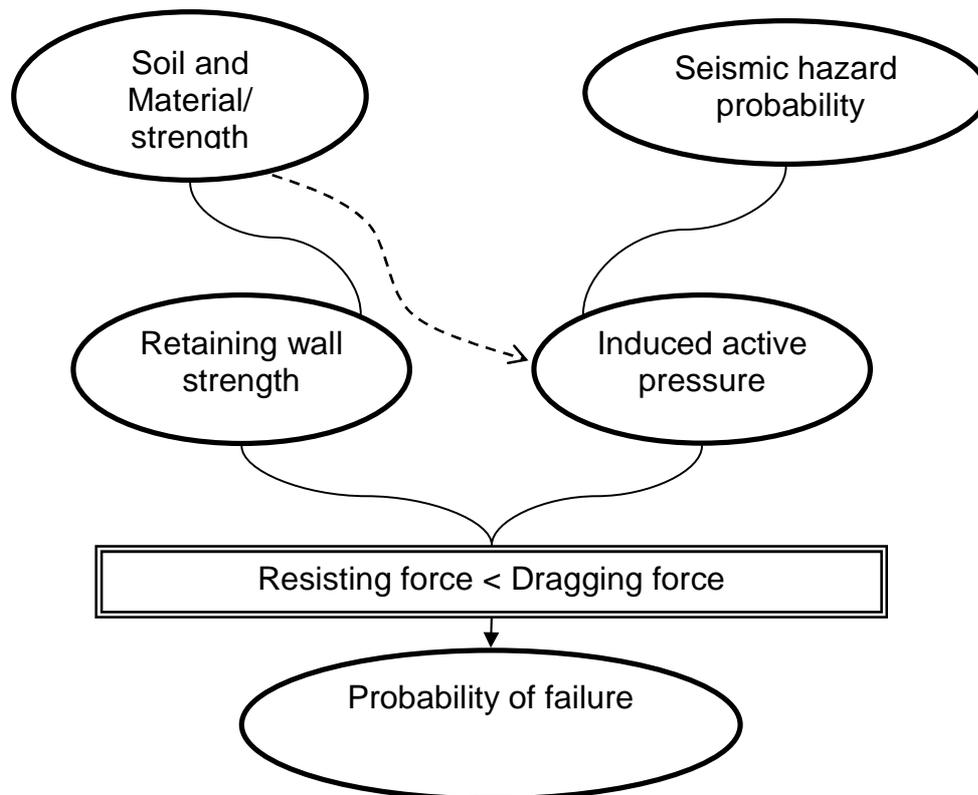


Figure 1.2: Conceptual Flow Chart

1.4. Organization of the Thesis

The synthesis of two years research work is presented in this thesis. The field work of for the survey was conducted for about 3 months to collect the retaining wall properties and measurement of their strength and age. There are altogether 8 chapters.

This, first chapter, introduces the study background, location of study area, objectives and organization of the thesis.

Chapter two is aimed to deal the present state of art in the reliability concepts in retaining wall design with the failure modes and seismic design consideration. The theoretical background and computation methodology that is used in the analysis of retaining wall failure is explained in chapter three. Fourth chapter is focused on the case studies about the damage of retaining walls during some major earthquakes occurred in Japan. These typical examples verify the possibility of

damages of the retaining wall due to future earthquakes and put the significance of such studies. Chapter five describes strength measurement of wall using Schmidt hammer, its applicability, and consistency. Further obtained strength values were analyzed to show the strength reduction pattern of concrete retaining walls with increasing age. In chapter six, seismic hazard is obtained for each field sites exhibiting possibility of occurrence of earthquakes. I have endeavored to describe the numerical process and results for the different mode of failure for each retaining wall in chapter seven. The modes of failure by overturning, sliding, bending or shear were analyzed with potential seismic hazard to deal with the failure probability of retaining walls. Finally, I have made conclusions based on my own study and further recommendations were presented in last chapter eight.

Chapter 2

Literature Review

2.1. Introduction

This chapter deals with the earth pressure and failure modes of retaining walls with present state of the art in the reliability concepts for slope stability analysis. The seismic pressure given by earthquakes is computed and analyzed for the design process more widely. The different types of uncertainty prevailing on the system enlighten the need of their analysis from the approach of reliability basis.

2.2. Earth Pressure in Retaining Wall

In the general discussions of retaining walls, they are visualized as a two-dimensional structure. The backfill behind the retaining wall produces a lateral pressure (thrust) on the wall. The thrust can be denoted by P , which is characterized by

- (a) Magnitude,
- (b) Direction, and
- (c) Point of application ("center of pressure").

That part of the thrust which is caused by dry cohesionless soil will be designated with P_a , is called active earth pressure. The determination of thrust is the chief problem in the analysis and design of a retaining wall. The value of thrust acting on a retaining wall in the real field site has some uncertainty and depends mainly on the properties of the backfill. The designer should provide some specific value for active thrust in designing of retaining walls. The soil properties and backfill have variability in parameters and their extent is complex to understand (Krynine 1941). In designing the walls, following sources of information are used:

- a. Theoretical formulas
- b. Empirical knowledge from tests and observations on large models and full-sized structures
- c. Engineering experience and judgment of the designer himself

Designer is required to design the safe and reliable structure. Now-a-days, the external loads from rare disasters should be considered. Earthquake, wind, and

environmental disasters are common problems that have been considered in designing. In case of existing structures, the variability and time dependent changes may cause to loosen the design load value of the structures. At the same time, the unexpected and rare disasters may overcome the design limits of the structures. Earthquakes can be taken as examples of such disasters. In this condition, the retaining walls may have probability of damage or failure.

2.3. Failure Mechanisms and Stability for Retaining Wall

Under static conditions, retaining walls are acted upon by body forces related to the mass of the wall, by soil pressure, and by external forces. A well designed retaining wall achieves equilibrium of these forces without inducing soil stresses up to shear strength of the soil. During an earthquake, internal forces and changes in soil strength make changes the equilibrium and cause the permanent deformation of the wall. So, the retaining walls design requires keeping the wall safe from different types of instability. There may be main three types of instability: overturning, sliding and gross failure (Kramer 1996). There are also some other types description of failures such as flexural failure and failures by bending effects.

Cantilever retaining walls may face any of overturning, sliding, gross failure and sometimes flexural failure. Gravity wall may have similar mechanism to cantilever except flexural. Damages of retaining wall failures during past earthquakes are described in chapter 4.

2.3.1. Overturning failure

Overturning failure occurs when the sum of moments tending to cause overturn will be greater than the sum of the moments to make resisting. It is common to require the safety factor at least 1.5 of resisting and driving moment for the design of retaining walls for static earth pressure (Day 2002). But it has to be investigated further more about procedures and influencing factors for design as well as for existing retaining wall case. Overturning may often happen by the bearing failures at the base of the wall which usually results from underestimating the driving force.

In the fig. 2.1, the overturning about a point may happen in exceeding the equilibrium safety condition.

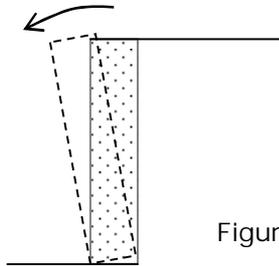


Figure 2.1: Overturning failure mechanism

2.3.2 Sliding failure

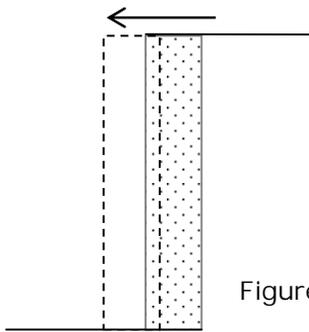


Figure 2.2: Mechanism of Sliding Failure

Sliding occurs when the lateral pressures on the back of the wall produce a thrust that exceeds the available sliding resistance on the base of the wall (Fig. 2.2). Generally, this failure occurs when either driving force is underestimated or the resisting force is overestimated. The underestimation comes from i) neglecting surcharge forces from other walls, ii) designing for length backfill when the backfill is in fact sloped.

2.3.3 Gross failure

This kind of failure is treated as slope stability failures that encompass the wall. Gross failure generally happens to the gravity type retaining wall and cantilever retaining wall (Fig. 2.3). The soil condition beneath the wall may response similar failure as gross type in some other wall types such as crib walls, bin walls and

mechanically stabilized walls. A number of internal mechanisms exist which may involve shearing, pullout, or tensile failure of various wall elements. In sloppy and

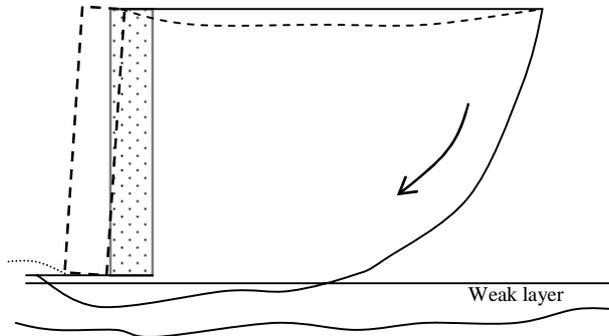


Figure 2.3: Mechanism of Gross Failure

hilly areas, excessive pore water pressure and consequently the liquefaction effect causes gross failure of walls and foundations. This failure is also known as global failure.

Earthquake resistant design of retaining structures like retaining walls, earth dams and foundations are very important problems to minimize the devastating effect of earthquake hazards. So, the inertia force originally from the self weight of a retaining wall and the earth pressure of the backfilling soil during earthquake must be considered as the effects at the time of earthquake.

2.4. Uncertainty and Reliability Concept

Most of the theoretical models and natural phenomenon consists of uncertainties due to their parameters and its behaviors. We can only reduce such uncertainty up to some level but we can't remove them absolutely.

2.4.1. Types of uncertainty

The retaining walls including any structure may have following three types of uncertainty (Christensen and Baker1982, Morgan and Henrion 1990).

Physical uncertainty: Whether or not a structure or structural element fails when loaded depends in part on the actual values of the relevant material properties that

govern its strength. The reliability analyst must therefore be concerned with the nature of the actual variability of physical quantities, such as loads, material properties and dimensions.

Statistical uncertainty: Data may be collected for the purposes of building a probabilistic model of the physical variability of some quantity which are involved parameters. For a given set of data, the distribution parameters may themselves be considered to be random variables. This uncertainty is termed as statistical uncertainty and, unlike physical variability, it arises solely as a result of lack of information.

Model uncertainty: Structural design and analysis make use of mathematical models relating desired output quantities (e.g. the deflection at the centre of a reinforced concrete beam) to the values of a set of input quantities or basic variables (e.g. load intensities, modulus of elasticity, duration of loading, etc.). The response of typical structures and structural elements contains a component of uncertainty in addition to those components arising from uncertainties in the values of the basic loading and strength variables. This can be described as model uncertainty because we assume continuous probability distribution in many assumptions.

These uncertainties can be described into two categories: Aleatory and Epistemic.

Aleatory uncertainty: Some natural phenomena have randomness in their occurrence or behavior. In such phenomena, we cannot reduce the uncertainty even if having large set of data. Aleatory literary means "pertaining to luck" which is said to exploit the principle of randomness. The randomness of earthquake can be taken as an example of aleatory type of uncertainty.

Epistemic uncertainty: The uncertainties that are based on expert knowledge or available data set, is known as epistemic uncertainty. In this case, we can reduce uncertainty in having larger data set.

Physical uncertainties about variability of soil profile may be aleatory when ground condition at a particular site is epistemic uncertainty. Statistical and model uncertainties may be epistemic.

2.4.2. Basic concepts of reliability for slope stability analysis

The uncertainty and risk are inevitable in applying the design procedures and observational methods in many geotechnical problems (Peck 1969). When the structures are constructed, the critical behavior of those structures can not be easily observed to make the required changes. So, the designer should rely on a calculated risk. Now, many theoretical investigations have evolved into methods that can be applied to practical problems (Christain et. al 1994). Selecting the design shear strength of soil raises following problems:

- Soil has real spatial variability within the profile which can not be avoided
- Data scatters due to random testing errors or noise
- A systematic error in the computed mean value of the property due to the limited number of tests performed, leads to statistical uncertainty
- An error in the mean occurs due to bias in measurement

An uncertainty in soil parameters consequently increases the risk level or causes the great variation of uncertainty in the factor of safety. In this case, the reliability index provides a more meaningful measure of stability of slope.

In case of probability of slope failures, the focus is placed on modeling of variability of spatial soil strength, pore-water pressure, external loads and geometrical variable (Cheung and Tang 2005) as quite similar to failure of retaining walls, embankment and foundations. The various factors that play a role in deterioration of slope and its performance are important parameters to be considered for modeling. The degradation of slope surface cover, blockage and damage of surface and subsurface drainage systems, reduction of soil strength due to weathering of slope forming materials and seasonal fluctuation of pore pressure are major factors involving in deterioration of slopes. Statistical variability of such factors is complicated to characterize. In these cases, constant probabilities in each unit of time such as

annual failure probabilities are used. But in real behavior the annual probability of failure is not constant. The variation of annual failure probability can be used as a variable upon time, so age dependent failure probability may be more realistic approach for such problems. This modeling method performed well in case of Hong Kong slope failure (Cheung and Tang 2005). Time dependent failure probability should be based on observation data and then using the probability methods. The advantages of time dependent failure probability is to find the serviceable time for the slope in the range of certain age (Tang and Cheung 2004). These results provided useful inputs to the realistic reliability assessment of a specific slope in Hong Kong.

In general analysis of reliability, the safety of retaining wall against earthquakes is determined by comparing the wall resistance with applied seismic load. The moment equilibrium method is commonly applied for slope stability analysis (Al-homound and Tahtamoni 2002). The failure probability can be given as,

$$P_f = P [M_D > M_R]$$

where, M_D is driving moment and M_R is resisting moment.

As given by Al-homound and Tahtamoni (2002), the failure surface is assumed to be cylindrical in a three dimensional analysis. The location of the sliding mass and its width is assumed to occur at their critical value. Although having high fluctuation and variability it is common to use homogeneity of soil properties, uniform cross-section, and particular slope geometries. When the earthquake occurs, the assumptions so far used are unidirectional ground motion for an earthquake traveling along the axis of slope (x-axis) and the spatial variability of earthquake in y-direction is neglected. Evaluating the mechanism and materials, it will be easy to compute failure probability for many structures.

2.5. Design Consideration for Seismic Stability

There are different design procedures for construction of retaining walls considering seismic stability of structures (Huang and Chen 2004). Since the earthquake is one

of major factor causing movements and/or failure of walls, the design process should fulfill the requirements of seismic stability. To ensure the safety of walls, following two basic approach of design procedure can be applied (Huang and Chen 2004, Choudhury et. al. 2004).

1. One of the popular methods is to fulfill the safety requirements by increasing the input seismic force in design process. This method is known as force based design method.
2. Another process of design is displacement based design to allow the retaining wall to move up to certain allowable displacement.

2.5.1. Force based analysis

Simply, stability of retaining structures is usually considered for static equilibrium condition. So, the static earth pressure analysis is applied for the analysis. The seismic stability of those structures is analyzed by pseudo-static approach in which effects of earthquake forces are expressed by constant horizontal and vertical acceleration attached to the inertia. Mononobe and Matsuo (1929) have obtained the active and passive earth pressure coefficients under seismic conditions. Their method is widely known as Mononobe-Okabe method. It was based on Coulomb's theory on static earth pressure considering the equilibrium of triangular failure wedge. The failure mechanism should be described on the following basis (Huang and Chen 2004).

1. Failure mechanisms are described separately as active and bearing capacity failures. Prior displacements may also occur on the walls which follow bearing capacity failure.

Sliding failure or direct shear failure along the wall base consists of a two wedge failure- wedge F and B behind the wall (Fig. 2.4), and a passive failure of wedge P in front of the wall.

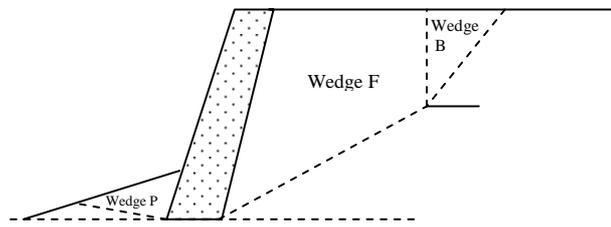


Figure 2.4: Failure wedges for bearing capacity

2. Ultimate bearing capacity of the slope under seismic loading is important to take account the inertia of soil mass confined to the slip line.
3. Tilting of wall caused by local yielding of foundation soil. When the foundation is weaker or slanted, the foundation slope failure becomes dominant.

2.5.2. Displacement based analysis

A retaining structure subjected to earthquake motion will vibrate with the backfill soil and the wall can easily move out from the original position due to earthquakes. The designing methods for the displacement based analysis of retaining structures during seismic conditions are based on the work of Newmark (Choudhury et. al. 2004), which allows some permanent displacements under dynamic condition.

2.5.3. Reliability based design method

It is easy to have uncertainties in the design analysis of the retaining structures during the earthquake motion. There are many parameters to be analyzed which affect the stability of the wall. All of the structures may have uncertainties on their parameters. So, probability of damage or failure can be estimated by probabilistic approach (Chalermyanont and Benson 2004, Hoeg and Murarka 1974)

The probability of failure is calculated as the probability of safety factor being less than unity. The variability in soil properties, wall components and surcharge load are characterized using random variables (Vanmark 1983, El-Ramly et. al. 2002). Frictional angle and unit weight are assumed to be normally distributed variables

(Phoon and Khuldhary 1999). Spatial variability of tensile strength of reinforcement and surcharge load can be assumed as normally distributed (Low and Tang 1997)

Among the slope stability methods, Bishop's simplified method predicts the factor of safety as satisfying all conditions of equilibrium (Wright and Duncan 1991). Firstly all the parameters analyzed and then calculated for factor of safety including randomly generated frictional angle, backfill unit weight and reinforcement tensile strength.

Application of PC programs: Recently there is wide range of application of computer software programs to evaluate the safety conditions of the structures using probabilistic approach (Yucemen and Al-Homond 1990, El-Ramly et. al. 2002) in terms of reliability index. Yuceman and Al-Homond's probabilistic model analyzes the stability of earth slopes under long-term conditions. Long term stability involves the main sources of uncertainty associated with method of analysis, pore pressure distribution, and in-situ values of angle of friction and cohesion. Fluctuation of spatial correlation associated with shear strength parameters increases uncertainty in resisting moment within a soil deposit.

Yuceman and Al-Homond's (1990) computer program PTDSSA has been considered the volumetric spatial averaging over the slope axis and arc of failure. It simplifies the procedure by assuming 1-D fluctuation model for earthquake motion. But the slope geometry is considered in 3-D model for realistic landslides. Pore pressure, angle of friction and cohesion are important for long term stability of slopes. 3-D stability analysis requires the spatial averaging of soil volume. PTDSSA analyses slope failure in 3-D.

The assessment of seismic hazard curve is also widely applied probabilistic approach for during earthquakes (Hwang and Huo 1994). The hazard curve can be assessed as a basic tool to observe various conditions of probability such as failure of structures, cost-benefit analysis, life-cycle analysis, insurance portfolio etc. There is also some web based programming to find out the hazard curve. The seismic hazard

curve was prepared for the needed field sites to analyze the failure probability of retaining walls.

This chapter concludes that the state of art in the earth pressure and seismic design of retaining wall shows the significance of the use of reliability to address the problems of safety. Because these problems have many uncertainties in model, phenomena or in assumptions so probability methods are essential for their interpretation.

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Chapter 3

Theoretical Background

3.1. Introduction

Different types of retaining walls, principal theory on their stability analysis, and basic formulations used in the analysis are described in this chapter. Earth retaining structures are constructed in different forms such as retaining walls, bridge abutments, anchored bulk heads, mechanically stabilized walls to maintain the stabilization of various structures. So, the retaining walls are also considered as the key elements of transport systems, buildings, port or harbors, lifelines and many other constructed facilities.

In this study, the retaining walls associated with the buildings are studied in detail. During the disasters like earthquake, the safety of the building is prime factor for which retaining walls are key elements.

3.2. Types of Retaining Wall

Numerous structural arrangements can be applied for the construction of retaining walls while the most commonly used types are considered as gravity type, cantilever, counterfort, crib walls, blocks. So, they are classified according to their relative mass, flexibility and anchorage conditions.

3.2.1. Gravity wall

They are the oldest and simplest type of retaining walls and commonly constructed of plain concrete although light reinforcements are sometimes used (Fig. 3.1). Gravity walls are thick and stiff enough, so that they don't bend generally. Similarly designed, certain types of composite wall systems with appropriate consideration of internal stability such as crib walls may bend very little. There is no tensile stress in the concrete in gravity walls.

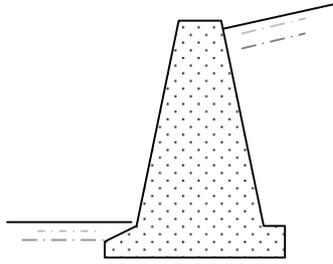


Figure 3.1: Gravity type retaining wall

3.2.2. Cantilever wall

Cantilever walls can bend, translate or rotate and they rely on their flexural strength of material to resist the lateral pressures on the wall. However, larger lateral pressure may also lead to structural damage. Cantilever walls are constructed so that the wall acts as a cantilever beam (Fig.3.2). The actual distribution of lateral earth pressure on a cantilever wall is influenced by the relative stiffness and deformation of both the wall and the soil.

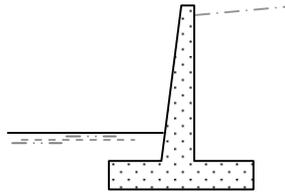


Figure 3.2 : Cantilever type retaining wall

3.2.3. Counterfort retaining wall

Counterfort retaining walls as known as buttressed have thin vertical concrete webs

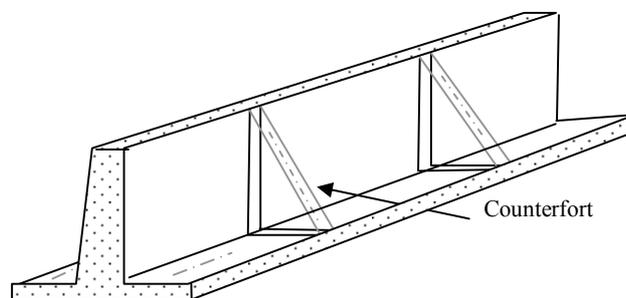


Figure 3.3: Counterfort type retaining wall

which are termed as counterforts. They are constructed in a regular interval along the backside of the wall and almost all configurations are similar to cantilever walls (Fig. 3.3).

The counterforts tie the slab and base together, and the purpose of them is to reduce the shear forces and bending moments imposed on the wall by the soil. A secondary effect is to increase the weight of the wall from the added concrete. They may be precise or formed on site. Counterfort retaining walls are more economical than cantilever walls for heights above 8 m.

3.3. Earth Pressure and Design Condition of Retaining Walls

The earth pressure theories describe about the condition to develop a state of limit equilibrium in the soil. Most of the retaining walls including gravity walls, cantilever walls, counterfort walls and crib walls, are more or less free to move at the top. According the type of wall, the design criteria may have some differences but basic principle remains same.

The seismic behavior of retaining walls depends on the total lateral earth pressure that develops during earthquake shaking. These total pressures include both the static gravitational pressures that exist before an earthquake occurs, and transient dynamic pressures induced by the earthquake.

3.3.1. Static pressures on retaining walls

Static earth pressures on retaining structures are influenced by wall and soil movements. There are two types of static pressures, active earth pressure and passive earth pressure.

When the retaining wall moves away from the soil behind it, the active earth pressure is developed. Minimum active earth pressure act on the wall when the wall movement is sufficient to fully mobilize the strength of the soil behind the wall. When the retaining wall moves towards the soil, passive earth pressure is developed producing compressive lateral strain in the soil. Maximum passive earth pressure acts on the wall when the strength of soil is fully mobilized. Under the static condition, a number of approaches are used to find out the strength of the retaining

wall because it has a complicated soil-structure interaction problem. So, we need estimate much higher value of factor of safety in order to design a safe structure.

The common approaches given by Rankine (1857) and Coulomb (1776) are widely applicable in most of the practices.

3.3.1.1. Rankine's Theory

Rankine (1857) developed the procedure for computing minimum active and maximum passive earth pressure. Minimum active pressure at a point on the back of a retaining wall as:

$$p_A = K_A \sigma'_v - 2c\sqrt{K_A} \quad (3.1)$$

where,

K_A is the coefficient of minimum active earth pressure,

σ'_v is the vertical effective stress at the point of interest, and

c is the cohesive strength of soil

When the principal stress planes are vertical and horizontal (as in the case of a smooth vertical wall retaining a horizontal backfill), the coefficient of minimum active earth pressure is given by

$$K_A = \frac{1 - \sin \phi}{1 + \sin \phi} = \tan^2 \left(45^\circ - \frac{\phi}{2} \right)$$

For case of cohesionless backfill inclined at an angle β with the horizontal, infinite slope solutions can be used to compute K_A as

$$K_A = \cos \beta \frac{\cos \beta - \sqrt{\cos^2 \beta - \cos^2 \phi}}{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \phi}} \quad \text{for } \beta \leq \phi$$

For dry homogenous cohesionless backfill, Rankine's theory predicts a triangular active pressure distribution oriented parallel to the backfill surface (Fig. 3.4). The active earth pressure resultant, P_A , acts at a point located $H/3$ above the base of a wall of height, H , with magnitude

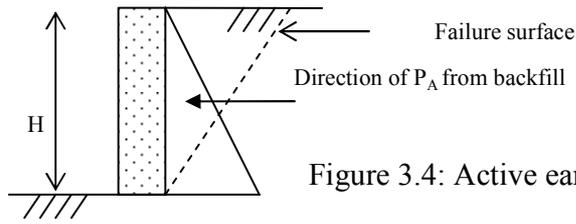


Figure 3.4: Active earth pressure by Rankine

$$P_A = \frac{1}{2} K_A \gamma H^2 \quad (3.2)$$

Under maximum passive conditions, Rankine theory predicts wall pressure given by

$$p_p = K_p \sigma'_v + 2c\sqrt{K_p}$$

where, K_p -the coefficient of maximum passive earth pressure and c -cohesion

For smooth, vertical walls retaining horizontal backfills,

$$K_p = \frac{1 - \sin \phi}{1 + \sin \phi} = \tan^2 \left(45^\circ + \frac{\phi}{2} \right)$$

and

$$K_p = \cos \beta \frac{\cos \beta + \sqrt{\cos^2 \beta - \cos^2 \phi}}{\cos \beta - \sqrt{\cos^2 \beta - \cos^2 \phi}} \text{ for backfills inclined at } \beta \text{ to the}$$

horizontal.

For a dry homogenous backfill, Rankine theory predicts a triangular passive pressure distribution oriented parallel to the backfill surface. The passive earth pressure resultant, or passive thrust, P_p , acts at a point located $H/3$ above the base of a wall of height H with magnitude

$$P_p = \frac{1}{2} K_p \gamma H^2 \quad (3.3)$$

3.3.1.2. Coulomb's Theory

Problem of lateral earth pressure on retaining was studied first by Coulomb (1776). There was assumed number of potential failure surfaces to identify the critical failure surface i.e. the surface that produces the greatest active thrust or the smallest passive thrust.

Under the minimum active earth pressure conditions, the active thrust on a wall with the geometry as shown in Fig 3.5. is obtained from force equilibrium. For the critical failure surface, the active thrust on a wall retaining a cohesionless soil can be expressed as

$$P_A = \frac{1}{2} K_A \gamma H^2 \quad (3.4)$$

where,

$$K_A = \frac{\cos^2(\phi - \theta)}{\cos^2 \theta \cos(\delta + \theta) \left[1 + \sqrt{\frac{\sin(\delta + \phi) \sin(\phi - \beta)}{\cos(\delta + \theta) \cos(\beta - \theta)}} \right]^2}$$

δ is the angle of interface friction between the wall and the soil, and β and θ are as shown in Fig. 3.5. The critical failure surface is inclined at an angle to the horizontal

where,

$$\alpha_A = \phi + \tan^{-1} \left[\frac{\tan(\phi - \beta) + C_1}{C_2} \right]$$

$$C_1 = \sqrt{\tan(\phi - \beta) [\tan(\phi - \beta) + \cot(\phi - \theta)] [1 + \tan(\delta + \theta) \cot(\phi - \theta)]}$$

$$C_2 = 1 + \{ \tan(\delta + \theta) [\tan(\phi - \beta) + \cot(\phi - \theta)] \}$$

P_A acts at a point located $H/3$ above the height of a wall of height H .

Equation (5) was used to find out the maximum active earth pressure applied to the wall in static condition. For maximum passive conditions in cohesionless backfills, Coulomb theory predicts a passive thrust

$$P_P = \frac{1}{2} K_P \gamma H^2 \quad (3.5)$$

where,

3.3.2.1. Mononobe-Okabe method

Based on the Coulomb theory, Mononobe and Matsuo (1929) developed the basis of pseudo-static analysis of seismic earth pressures on retaining structures that has become popularly known as the Mononobe-Okabe (M-O) method. Okabe (1924) also discussed the stability of walls during earthquakes.

For the Active Earth Pressure Conditions:

The forces acting on an active wedge in a dry, cohesionless backfill are shown in Fig. 3.5. There are also additional forces that exist under static conditions. In this case, total active thrust can be expressed in a form similar to that developed for static conditions, that is,

$$P_{AE} = \frac{1}{2} K_{AE} \gamma H^2 (1 - k_v) \quad (3.6)$$

where, the dynamic active earth pressure coefficient, K_{AE} , is given by

$$K_{AE} = \frac{\cos^2(\phi - \theta - \psi)}{\cos \psi \cos^2 \cos(\delta + \theta + \psi) \left[1 + \sqrt{\frac{\sin(\delta + \phi) \sin(\phi - \beta - \psi)}{\cos(\delta + \theta + \psi) \cos(\beta - \theta)}} \right]^2}$$

where, $\phi - \beta \geq \psi$, $\gamma = \gamma_d$, and $\psi = \tan^{-1}[k_h / (1 - k_v)]$, and γ is unit weight; k_h , k_v are horizontal and vertical components of earth pressure respectively

The critical failure surface, which is flatter than the critical failure surface for static conditions, is inclined at an angle

$$\alpha_{AE} = \phi - \psi + \tan^{-1} \left[\frac{-\tan(\phi - \psi - \beta) + C_{1E}}{C_{2E}} \right]$$

where

$$C_{1E} = \sqrt{\tan(\phi - \psi - \beta) [\tan(\phi - \psi - \beta) + \cot(\phi - \psi - \theta)] [1 + \tan(\delta + \psi + \theta) \cot(\phi - \psi - \theta)]}$$

$$C_{2E} = 1 + \{ \tan(\delta + \psi + \theta) [\tan(\phi - \psi - \beta) + \cot(\phi - \psi - \theta)] \}$$

In the calculation of earth pressure using 3.6, the vertical component was neglected as suggested by Seedman and Whitman (1970). Then the equation 3.6 can be written as

$$P_{AE} = \frac{1}{2} K_{AE} \gamma H^2 \quad (3.7)$$

For passive pressure condition:

The maximum passive pressure can also be calculated by Mononobe Okabe method as

$$P_{PE} = \frac{1}{2} K_{PE} \gamma H^2 (1 - k_v) \quad (3.8)$$

where,

dynamic active earth pressure coefficient, K_{AE} , is given by

$$K_{PE} = \frac{\cos^2(\phi + \theta - \psi)}{\cos \psi \cos^2 \theta \cos(\delta - \theta + \psi) \left[1 - \sqrt{\frac{\sin(\delta + \phi) \sin(\phi + \beta - \psi)}{\cos(\delta - \theta + \psi) \cos(\beta - \theta)}} \right]^2}$$

The total active thrust, P_{AE} , can be divided into a static component, P_A and a dynamic component, ΔP_{AE}

$$P_{AE} = P_A + \Delta P_{AE} \quad (3.9)$$

As given by Kramer (1996), a reinforced wall is treated as like a gravity wall for the evaluation of external stability. The earthquake loading is represented pseudostatically by the dynamic thrust, ΔP_{AE} . The external stability can be evaluated by the following procedure for a particular wall design.

Peak ground acceleration, a_c , at the centroid of the reinforced zone can be calculated from the equation

$$a_c = \left(1.45 - \frac{a_{\max}}{g} \right) a_{\max} \quad (3.10)$$

where, a_{\max} is maximum acceleration,
and dynamic thrust from

$$\Delta P_{AE} = 0.375 \frac{a_c \gamma^{(b)} H^2}{g} \quad (3.11)$$

where $\gamma^{(b)}$ is the unit weight of the backfill soil.

The point of act of earth pressure may differ as given for static component.

Seedman and Whitman (1970), has recommended to use as,

$$h = \frac{P_A \frac{H}{3} + \Delta P_{AE} (0.6H)}{P_{AE}} \quad (3.12)$$

Using this height, the overturning moment can be calculated by,

$$M_o = (P_{AE})_h h \quad (3.13)$$

where, $(P_{AE})_h$ = dynamic load at height h and h = height of wall to act dynamic load

The resisting moment can also be calculated using the wall configuration from its backfill soil load with reference to its toe. In this case, backfill unit weight, base width and height of the wall may be influencing parameters.

3.4. Stability of Retaining Wall

When the soil pressure on the back of retaining wall is estimated, the deformation condition imposed on the soil by the retaining wall is considered. The deformation conditions are generally controlled by the type of retaining structure adopted. Most common applicable type of retaining structure can be considered for analysis because there are wide ranges of variation in their structure. The higher extreme of active earth pressure and lower extreme of passive earth pressure is considered to take develop the state of limit equilibrium. Then the deformation can take place when the stress magnitude will be greater than that of a state of limit equilibrium.

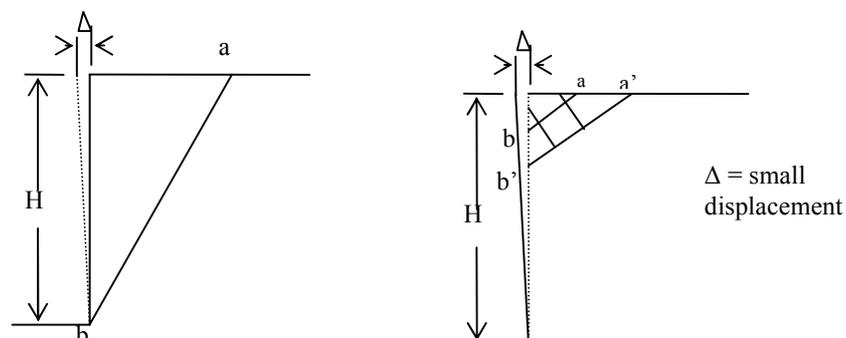


Figure 3.6: Titling of wall forming slip surface

If the wall tilts away from the soil, it forms a slip surface as in fig. 3.6, with a displacement of Δa close to the values required for parallel movement. When the wall rotates against the soil, large displacements are required to develop a state of shear failure in the soil even though shear failure is not common in cantilever walls.

During earthquakes, ground motion increases the earth pressure above the static earth pressure. The retaining wall needs a factor of safety of 1.5 for the static loading (PWRI 2004) which value is only expected to be able to withstand horizontal acceleration up to 0.2 gal. When the acceleration will be higher than 0.2 gal, additional design improvement should be applied in designing the retaining structure to withstand earthquake forces. The determination of safety factor may need more study.

The stability characteristic of wall also depends on the backfill soil types. Generally granular dry backfills are preferred. The shear strength of such granular material is relatively independent of the water content. The change in water content in soil is responsible to cause high fluctuation in shear strength and consequently the earth pressure of cohesive soils (Terzaghi 1943). The retaining wall construction with clay backfill shows unsatisfactory performance according to a survey (Peak et. al. 1948). The backfill soil for the purpose of retaining wall design is classified into following types:

1. *Type A*: Cohesionless soils such as sands and gravels with high permeability and soil type of GW, GP, SW, and SP.
2. *Type B*: Cohesionless soils such as sands and gravels containing some silt, GW-GP, GM-GW, SM-SP, SM-SW. permeability of such soils varies vastly with almost zero pore pressure.
3. *Type C*: Sandy and gravelly soils with considerable percentage of silts and clays, soils GM, GC, SM, and SC which have the properties of residual soils.
4. *Type D*: Silts and clays ML, MH, CL, and CH that are thoroughly broken into small pieces when placed in the backfill and these clays have high plasticity which is undesirable.
5. *Type E*: Clays that are placed in the form of large chunks which is filled with soft material. The strength depends upon the property of infilling, usually not preferable.

Mononobe-Okabe analysis assumes that the point of application of the total seismic earth force is at $H/3$ from the base of the wall. This is common assumption even some experimental results have shown that seismic force acts above $H/3$ from the base of wall in active case (Choudhary et.al. 2004).

3.4.1. Stability against overturning

Load due to the self weight of retaining wall, surcharge earth pressure will act to the bottom of base of retaining wall. The ground reaction below the bottom will vary depending upon the location of point to which the resultant of this loads works. The overturning can be checked by comparing the resisting moment and driving moment. They are represented by,

ΣM_r : moments of resistance at toe of base of retaining wall (kN·m/m)

ΣM_o : overturning moment at toe of base of retaining wall (kN·m/m)

Simple resisting moment can be calculated by evaluation of following three types of load parameters:

a). load due to the concrete retaining wall

$$M_c = \gamma * t * h$$

where, γ = unit weight of concrete (kN/m)

t = thickness of wall (m)

h = height of wall (m) (above the base)

b). load due to the base of the retaining wall

$$M_B = \gamma * t * h$$

where, γ = unit weight of concrete (kN/m)

t = thickness of wall (m)

l = length of wall base (mostly $1/3$ of h)

c). load due to the soil above the wall base

$$M_s = \gamma * b_s * h$$

where, γ = unit weight of soil above base of wall base (kN/m)

b_s = distance between point of action (of moment) and midpoint of concrete base below soil

h = height of wall (above the wall base)

3.4.2. Stability against sliding

The force, which tends to slide the retaining wall along the plan below the base, is the horizontal component of the earth pressure and is resisted by the shear resisting force created between the foundation ground and base. The factor of safety against sliding should satisfy the following formula.

$$F_s = \frac{\text{Resisting force against sliding}}{\text{Sliding force}} = \frac{(\Sigma V \cdot \mu + c_B \cdot B)}{\Sigma H} \quad (3.14)$$

where,

ΣV : Total vertical load on the bottom on the base

ΣH : Total horizontal load on the bottom on the base

μ : Coefficient of friction between base and foundation ground

c_B : Adhesion between base and foundation ground

B : Width of base of retaining wall (m)

The safety factor shall not be less than 1.5 at the ordinary condition and 1.2 at time of earthquake. Width of the base should be increased for stabilization if the factor of safety F_s is not satisfied as required during design of the wall. There will be some other factors such as topographic condition, the depth of embedment which will also affect the stability of the wall. Equation (3.14) gives the factor of safety for design purpose. For the analysis of stability checking during earthquake, resisting and sliding forces are compared.

3.4.3. Entire stability of retaining wall

In the case of retaining wall construction on ground incorporating a soft layer, the load of the embankment at the back of the wall may cause failure inside the ground, consolidation settlement by the embankment and /or lateral flow of the ground. The stability against such failure should be confirmed by analyzing the possibility of circular sliding and other relevant matters.

Above methods in this chapter are bases of theoretical bases for calculation of earth pressure which gives stability of retaining walls. Numerical results from these calculations can be used for appropriate probability distribution to find out the failure probability.

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Chapter 4

Damages of Retaining Walls due to Earthquakes

4.1. Introduction

Damage of retaining walls during the earthquake can be seen in many past earthquakes. This chapter is aimed to describe some of large earthquakes and their damages to retaining wall. This will show typical examples of case study and signifies the importance of such study in housing lots and individual houses. This damage scenario can be taken as verification of this research results.

4.2. Types of Damages

The damages to the retaining walls are caused by the various effects of earthquakes such as direct wave motion, seismic force applied to wall of structure, liquefaction in supporting ground, gross failure of slopes with structures. Different triggering factor have different response to the earthquake effects. This effect is based on geographical and geological condition of the area, seismic intensity and safety condition of the structures (Tateyama et.al.1995). Failure mechanism also differs according to intensity of triggering factors.

The retaining wall damages can be divided into following two types of damages.

1. *Material damage*: the retaining wall itself is a concrete material. When the wall itself goes to cracking down, breaking or distortion, then this can be described as material damage. The tensile or shearing stress to the wall causes material damage or its failure.
2. *Structural damage*: When the framework or structure of wall is damaged, it can be said to be structural damage. The displacement, collapse or distortion of the structure either partially or completely causes the structural damage. When the entire wall is overturned or slid without any cracks or damages to the wall components, it can be a typical example of structural failure.

In most of the cases, both of the above damage types occur at the same time. The masonry walls may tend to be easily breakdown under larger earth pressure. The small cracks and displacements are not considered in evaluation of safety for rigid retaining walls (Prakash and Wu 1996, Nadim and Whitman 1983) because some allowable displacements are permitted during their designing. The displacements at the base of the wall structure are important thing to consider for rigid retaining

walls like cantilever and some other reinforced types. Sliding and rotation at the base are commonly observed for such walls. The seismic design also provides allowable displacements based on performance of wall types (Ostadan and White 1998).

The mechanism of failure of retaining walls are sliding, overturning, and gross failure that may lead to collapse of rigid walls (Kramer 1996). These failure mechanisms are also described in chapter 2.3.

The estimation of damage due to earthquake is usually developed with some assumptions on earthquake probability and extent of damage (Grossi 2000) for the mitigation purposes. In the same background of geographical and geological condition, one type of retaining wall may have more damages than other type, due to the difference of performance of each type of walls. Therefore, my study focuses on concrete retaining walls.

4.3. Damages of Retaining Walls in Different Earthquakes

Many earthquakes have caused large scale of damages of lives and property from past. The type of damages by very recent and past earthquakes greatly varies because of the change in infrastructure development, construction technology and response of people to disasters and social environment. The scale of damage is very much reduced if we see from Kanto earthquake to Kobe earthquake even we need to draw much attention to reduce such effects. There are frequent earthquakes in Japan such as Fukuoka-ken Earthquake 2005, Niigata Chuetsu Earthquake 2004, Tokachi-Oki Earthquake 2003, Tottori-ken Seibu Earthquake 2000, Hyogoken Nanbu (Kobe) Earthquake 1995. Some of them are discussed below.

Hyogoken Nambu (Kobe) Earthquake

Niigata Chuetsu Earthquake

Fukuoka ken Earthquake

4.3.1. Hyogoken Nambu (Kobe) Earthquake

Hyogoken Nambu (Kobe) Earthquake with JMA (Japan Meteorological Agency) intensity of 7.2 occurred in January 17, 1995. Some of the investigations have been done on this earthquake focusing the retaining wall damages (JSCE 1998, Prakash

and Wu 1996, Tateyama et. al. 1995, Tatsuoka et.al. 1995). During the earthquake, many of conventional masonry and un-reinforced concrete gravity-type retaining walls were totally collapsed. Damage of reinforced cantilever concrete retaining wall was around 8% among all types of walls. High damage for the walls in residential area was occurred at the base of Rokko-san mountain and Suma ward. The damage suffered area includes Tsurumi ward, Kita ward, Nishinomiya city and Kawanishi city. Yamada and Kitagawa (2005) estimated the vulnerability function on building damage data for Kobe city with respect to structural type, construction date and the predominant period of ground.

JSCE (1998) study was done for total houses of 5100. The totals of 7000 were needed to be repaired from 2400 houses. According to PWRI (1997), the factors examined in the design of a retaining wall are the sliding and overturning failure of the retaining wall, the bearing capacity of the foundation ground, the stability against sliding of the entire structure including the foundation, and also the stress generated in the body.

Damages due to soil condition

There are different kinds of ground conditions comprising of Osaka Layer Group (Osaka Basin Clay of volcanic ashes and marine clay deposits). The deformation in soil is observed at the base of structures and foundations (JSCE 1998). The cases of retaining wall damages and deformation in ground condition have similar type of distribution (Fig. 4.1).

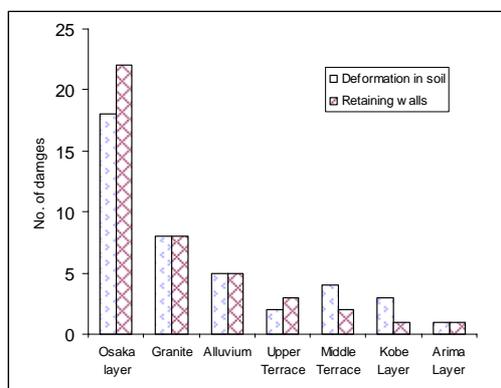


Figure 4.1: Damages of retaining walls and soil deformation
(Source: JSCE 1998)

As reported by Tateyama et. al. (1995), high seismic forces applied to the wall structure and the backfill was the main triggering factors to cause the failures for many highway embankments. However, their observation shows that high damage rate was not found in the grounds of thick clays deposits for those highway embankments. In contrary to this, the deformation on the ground soil was well compared with the damages of retaining walls on the housing lots and individual houses (JSCE 1998) in the same area. In later case, most of the damages were found on Osaka Layer.

Damages due to retaining wall types

Tateyama et. al (1995) have explained the damages to retaining walls based on the type of walls. Masonry walls, the oldest type walls, were not designed according to seismic design. These types of walls were seriously damaged among all other walls. Most of these walls were constructed more than 70 years ago. These types of walls are not allowed for railway embankments now. Leaning type of un-reinforced concrete walls with no seismic design was mainly constructed more than about 60 years ago which suffered largely by the overturning and then collapses.

Masonry, leaning type and gravity type un-reinforced walls were assumed to resist lateral earth pressure but couldn't perform so much during the evaluation of earthquakes damages. Masonry (concrete block) retaining walls consist of stone or concrete blocks bonded and piled. Most of the damage road – retaining walls is designed on the basis of empirically obtained standard cross- section, without stress calculation JSCE (1998).

However, during the earthquake, since a horizontal inertia force that is proportional to the wall's own weight came into effect, gravity type retaining walls constructed on sides other than roads were destroyed, or slid (JSCE 1998). The cantilever type retaining walls were also aseismically designed (Tatsuoka et. al. 1995) nearly about 30 years ago. These walls were less damaged in comparison with other types, however cracking and tilting was occurred in many cases. The geo-grid reinforced soil retaining walls were performed well but the cases of these types wall were very few.

Cantilever types of walls were affected mainly due to lateral seismic earth pressure. Cantilever retaining walls could have been damaged in higher intensity of ground motion. The reinforced concrete retaining walls which are constructed by applying seismic standard for design are considered safe against normal intensity of earthquake around 200 gal. Beyond this standard those walls may also have chances to fail. So, higher intensity of earthquake should be considered for safety analysis. The reasons for this are considered to be that special attention is given to earthquake stability in the design. Since, the earth pressure is a large factor among the external force acting on the retaining wall, the safety margin including in the calculation of the earth pressure is very influential. However, some retaining wall constructed on relatively weak foundation, such as those constructed at the approaches to bridges over, were seriously damaged. The damages for different types of wall are also shown in fig 4.2 (source JSCE 1998).

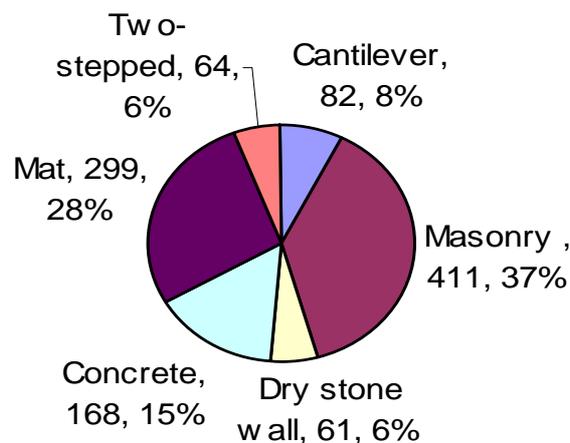


Figure 4.2: Distribution of damages for different types of retaining wall types

Damage of retaining wall and age effect

According to the age of the retaining wall, the distribution of damage is shown below (Fig. 4.3) (JSCE 1998). The land development made from 1956 to 1989 was studied using aerial and topographic maps of that period. They have classified four age ranges and each no of cases were represented by more than one hector housing lots area. The age range is older than 1935, 1936-1961, 1962-1969 and 1970 or later. The highest damage cases, 75% were occurred in the age range of

1936-1961. It is believed that the caused of damage to this range was due to the old Building Standard Law because it didn't required high strength of retaining wall and not need of compaction of land surface. The regulation including the housing land development was only enforced in 1969. After that period small number of damages was observed.

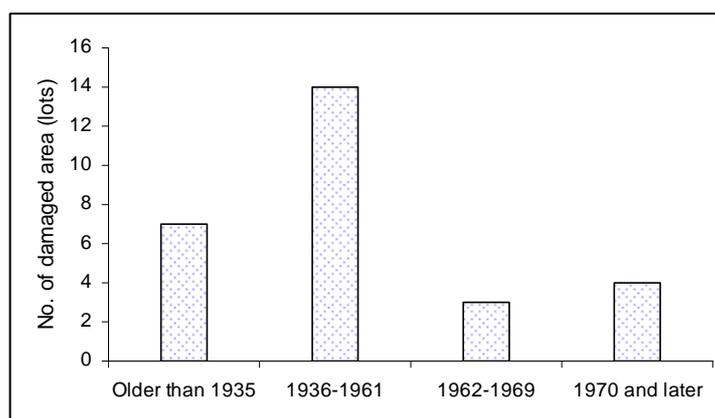


Figure 4.3: Damages and age of retaining walls

4.3.2. Niigata Chuetsu Earthquake

Niigata Chuetsu earthquake occurred in October 23, 2004 with the JMA intensity of 6.8. Highly damaged area includes Kawaguchi, Tokamachi, Ojiya, Horiuchi.

The large scale damages consist of many slope failures and consequently damages on building and retaining structures. The collapses of railway and highway embankments were affected widely by underneath failure of the foundation of such structures.

The soft ground consisting silty clay soil was dominant in Nagaoka area. Earlier Niigata Earthquake of June 1964 hit the city having less damage to the buildings supported by reinforced concrete walls (AIJ 1970). Buildings which followed the Standard Design Manual were fairly safe. The damages studies by Tamura et. al (2005) has explained different types of damages and ground conditions which are given below.

Damages due to ground condition

The damages based on the ground condition, shows that the filled ground soil have caused high collapses. The figure 4.4 and table 4.1 shows that the collapse, displacement of foundation and cracks were high in filled ground. The cut and flat ground has fewer damages of displacement and cracking with no complete collapse of structures.

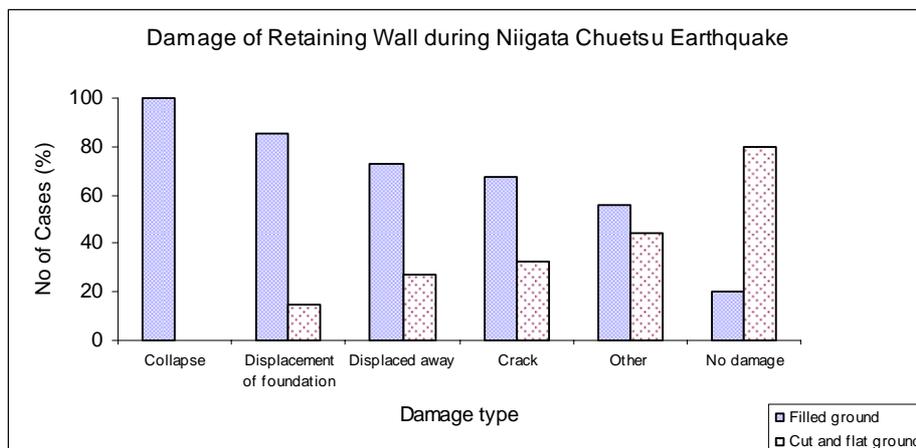


Figure 4.4: Damage of retaining wall and ground relation (Tamura et. al. 2005)

The totals of 548 cases were seen in this case. The damage on ground condition shows that the occurrence of landslide is also high as 75 cases in filled ground. Then the fractures and cracks with unequal settlement on ground surface were found for 70 cases (Table 4.1 and 4.2).

Table 4.1: Damage on ground surface

Damage type	Damage level (%)	Damage for type of ground condition		Total cases of retaining walls
		Filled ground (valley filling)	Cut and flat ground	
Landslide	9.1	89.3	10.7	75
Unequal settlement, fractured	8.5	88.6	11.4	70
Simple settlement	4.5	78.4	21.6	37
Cracks	10.2	73.8	26.2	84
Liquefaction	0.9	57.1	42.9	7
No damage	66.7	21	79	548
Total	100			

During the field visit to the Nagaoka area, the ground condition and the damage cases were observed (Fig. 4.5). The filled ground was found to be highly damaged than other types of natural ground.



Figure 4.5: Damage on the filled ground surface of the building foundation (photograph taken at Nagaoka Technical College)

Damages due to retaining wall types

Damage due to retaining wall types is represented by figure 4.6.

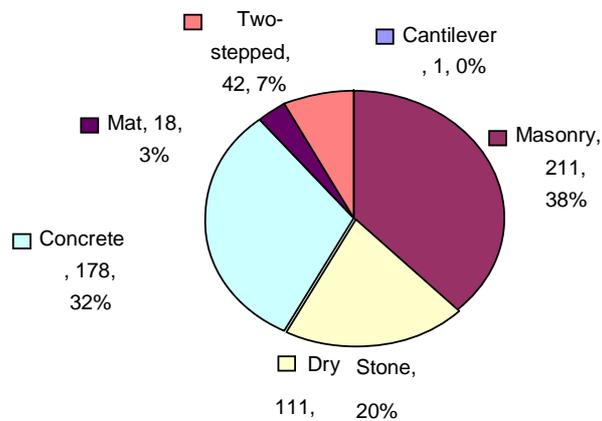


Figure 4.6: Damages of different types of retaining wall

Most of the damages have occurred in masonry and two stepped walls. Cantilever walls have least damage in this case.

Types of damages

The retaining wall collapse was highest on the filled ground even small damages were highest in flat ground. The filled ground was mainly valley filling. The types of damage can be seen in table 4.2 and figure 4.7.

Table 4.2: Retaining wall damages based on types of damages

Type of damage of retaining walls	Damage level (%)	Filled ground (valley filling) %	Cut and flat ground%	Total cases
Collapse	3.3	100	0	27
Displacement of foundation	5	85.4	14.6	41
Displaced away	8.4	72.5	27.5	69
Crack	22.8	67.4	32.6	187
Other	1.1	55.6	44.4	9
No damage	59.4	19.9	80.1	488
Total	100			

The most of the damages have occurred in masonry and dry stone walls.

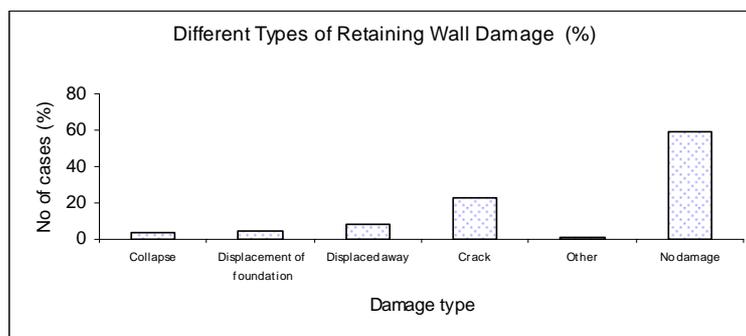


Figure 4.7: Different types of retaining wall damage

The figure 4.8 shows the damage to the wall composed of block and concrete. This is located in Tokamachi, one of the severely suffered villages.



Figure 4.8: The small retaining structure was sloped down in the photograph in Tokamachi village.

In the figure 4.9, the wall is severely damaged due to its structural arrangement. The upper part was simple concrete construction and the lower part was tile. Tiles couldn't hold the load given during the earth pressure and seems to be collapsed. Such failure cases have been commonly happened in different affected areas.



Figure 4.9: Damage to the retaining wall of weak structural arrangement

4.3.3. Fukuoka-ken Earthquake

The Fukuoka-ken earthquake was occurred on March 20, 2005 with the intensity of JMA scale 7. A small Genkai Island was highly affected due to this earthquake.

The damages of Fukuoka earthquake has been well explained in a report of Japanese Geotechnical Society (2005). The contour map of the earthquake intensity and the damage distribution was found well correlated (JGS 2005) and such

approach of using the contour map would be useful in many other earthquakes to find the hazard.

Damages based on types of retaining walls

The earth retaining walls of masonry and dry stone walls were highly damaged in comparison to other types. The types of walls were categorized into following types (JGS 2005).

- a. Masonry retaining walls
- b. Dry stone walls
- c. Concrete walls
- d. Gravitational type retaining walls
- e. Others
- f. Damage to the surface of wall protection

The distribution of the damage of these walls is depicted in the figure 4.9.

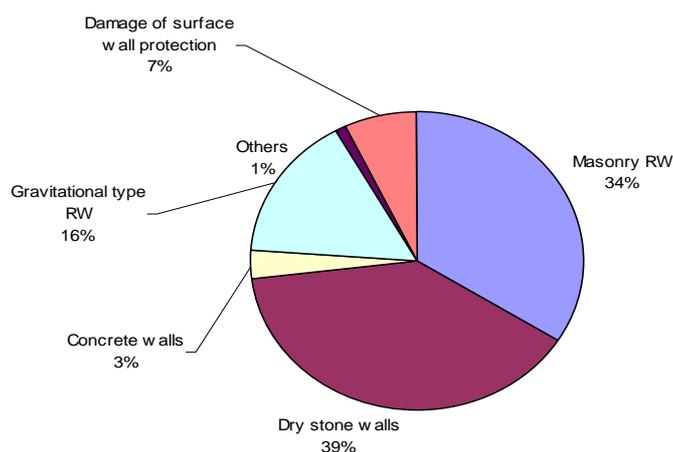


Figure 4.10: Different types wall damages (Source: JGS 2005)

Some minor cracks and spalls were found in concrete retaining walls (JGS 2005). The typical example of an elementary school in the island shows the significant cracks in the filled ground whereas the same ground has minor cracks in the natural ground supported by concrete retaining walls.

Based on the age

The age of the construction was divided into 5 categories (JGS 2005). The age range was 1995-1965, 1965-1975, 1975-1985, 1985-1995 and after 1995. The damage to the retaining walls constructed from 1965 to 1975 was higher than others.

According to the damage statistics of Great Kanto Earthquake for reinforced concrete buildings in the Old Tokyo City, the damage rate was higher in uptown area even in good ground condition. This damage rate includes partial damage to the structures too. In case of serious damage, the damage rate is higher in downtown where the ground was soft. Some examples given by Prakash and Wu (1996) consists damages to the rigid retaining walls during Northridge Earthquake and Hokkaido-Nansi-Okai Earthquake. In that case the allowed displacement was overcome by many rigid retaining walls which were supposed to bear such loads. The damages of earthquake to the buildings are also dependent on the foundation and supporting retaining structures (Bruneau 2002, and Faccioli et. al. 1999).

From the above case studies of major earthquake damages, large scale damages of retaining wall are associated with ground condition and slope failures. The relation between deformation on ground soil and retaining wall damages is well established in case of those walls constructed in housing lots or individual houses rather than those of highway or railway embankments. It makes significance that separate consideration of those retaining walls should be the new research approach to find out their safety assessment. Sometimes, rather safe structures may be collapsed because of lack of concern of ground behavior or having uncertainty in some design parameters. Preparation of contour map for the intensity of earthquake will be an appropriate method to predict hazard and it is also useful to estimate possible damages.

Damage of reinforced concrete retaining wall can be seen in above earthquakes (Fig. 4.2, 4.6 and 4.10). Large numbers of retaining walls are being constructed in recent years. So, their study for estimation of safety is important. Higher intensity of ground motion than in design standard should be considered for these type of retaining walls.

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Chapter 5

Application of Schmidt Hammer and its Results

5.1. Introduction

The use of Schmidt hammer to measure strength of retaining walls, its obtained results and age are dealt in this chapter. The strength of the retaining wall will go decreasing by age but its rate may depend on various factors. Since, the use of Schmidt hammer is commonly accepted for the preliminary assessment of concrete strength as the non destructive test, this will provide useful estimation about the retaining walls and their strength parameters.

5.2 Instrumentation

The Schmidt Hammer was originally designed by E. Schmidt in 1948 as a nondestructive method of testing the strength of in situ concrete (Schmidt 1951). After this its use has been extended widely in finding the strength of concrete structures as well as rock outcrops (Cargill and Shankar 1990, Amaral et. al. 1999 and Qasrawi 2000). The amount of rebound of the hammer is measured and correlated with the manufacturer's data to estimate the strength of the concrete (FEMA 306, 1999). There are different types of Schmidt Hammer as N and L types. The PROCEQ N type of hammer was used to carry out the strength of the concrete from the field. It is recommended to use for the non-destructive testing of the uniformity of concrete and for estimating the compressive strength. This N type of hammer (Fig. 5.1) has the following specification as provided by PROCEQ Company, Switzerland.



Figure 5.1: PROCEQ Schmidt rebound hammer

Impact energy = 2.207 Nm

Measuring range = 10-70 N/mm² Compressive strength

It has standards ASTM C805 / BS 1881, Part 202 / DIN 1048, Part 2 / UNE 83.307 / ISO / DIS 8045. The application of Schmidt Hammer to the concrete testing is widely accepted following the procedures given by ASTM C805 (ASTM 1995).

5.3. Operation Principle of Schmidt Hammer

Basu and Aydin (2004) have described operation principle of the Schmidt hammer. It consists of a spring-loaded piston of a steel mass. When the hammer is pressed orthogonally against a surface, the piston is automatically released onto the plunger (Fig. 5.1) and the rebound height of the piston is considered to be an index of surface hardness. A small sliding pointer indicates the rebound of the hammer on the graduated scale.

5.4. Consistency of Schmidt Rebound Values

Destructive methods of evaluation are inherently limited because specimen removal may be aesthetically and structurally damaging. Further, because of the potentially, structurally destructive nature of these methods can be relatively expensive and aesthetically unpleasant. The number of specimens taken may be limited to a small number for such evaluation method. Thus, potentially, the quantity and quality of the resulting data may be poor and/or inconsistent. Schmidt hammer may have some variability, so some destructive tests can be done for further reliability of rebound values.

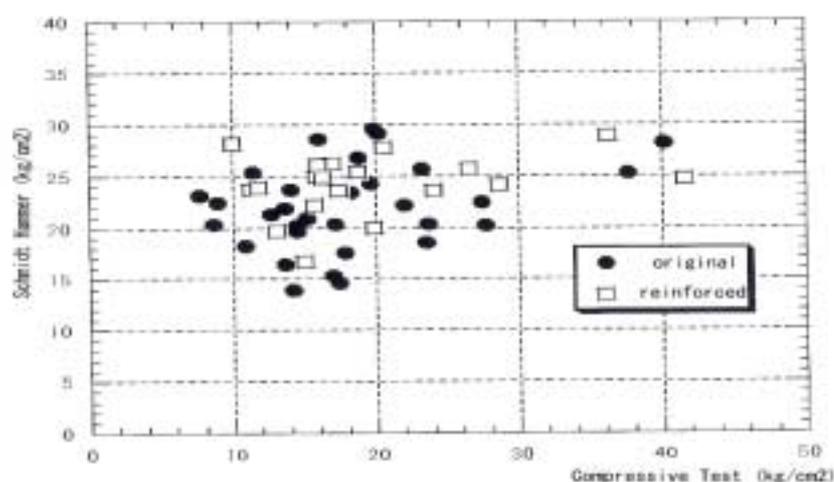


Figure 5.2: Relationship between Schmidt hammer value and compressive strength

Source: Mitsubishi Est. Corp. Ltd. 2002

The consistency of the use of Schmidt hammer can be observed in Fig. 5.2. It is the building of Industry Club of Japan near Tokyo Station. There is some variation in the Schmidt Hammer and Compressive strength values because it was used in different parts of the building and also the repaired reinforcement part and non repaired original part of that single building.

Some of the researches have shown that the Schmidt hammer rebound values are consistent even having small variation due to spatial variability of wall face (Poole and Farmer 1980). The empirical correlations between rebound readings and compressive strength can be derived from some standard tests (Katz et. al 2000, Kahraman 2001, Qasrawi 2000, Yilmaz 2002). The materials having higher density would tend to have a higher compressive strength (Cargill and Shankoor 1990). Now the digital hammers are also available in which inbuilt normalized values can be read. Sometimes, Schmidt hammers are used to find the variability of the compressive strength on a wall surface.

5.5. Advantages of using Schmidt hammers

Using the Schmidt hammers has following advantages:

1. A small amount of structure damage occurs in testing, usually negligible.
2. It makes possibility of testing concrete strength in structures where cores cannot be drilled. For example thin walls, densely reinforced walls etc.)
3. It has an application of less expensive testing equipment.
4. It doesn't need high consumption of labor.

5.6. Application and Assessment of Schmidt Rebound Values

In the field site, the Schmidt hammer was used for all concrete retaining walls that were observed during the survey. To reduce the error the following procedures were applied.

1. The wall surface was tentatively divided into grids of one square meter.
2. Rebound value was obtained from each of the grid area
3. The 3-5 reading of rebound numbers was measured and the average of them was found. Extremely higher and lower values were ignored. The consistency in the rebound value was expected from several tests.

4. The strike of the hammer was introduced from horizontal direction to obtain compressive strength easily.
5. In some cases, additional readings were taken if initial rebound values were not consistent.
6. The rebound value were converted into cylinder compressive strength value (kg/cm²) using the provided conversion chart (Fig. 5.3).
7. The obtained values were reduced to using the time factor provided by the company (Table 5.1).

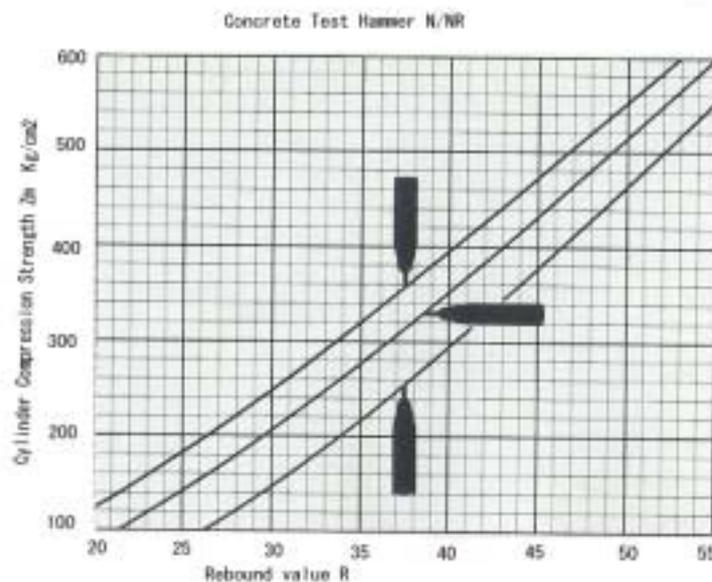


Figure 5.3: Conversion chart for rebound value and compressive strength

This time factor is influenced by the carbonation effect on the concrete wall. It has two major impacts on concrete.

- a) The basic environment of concrete is converted into an acidic one. Hence, the chemical protection of the reinforcement bars is lost. Then reinforcement bars start to corrode. Therefore, it is very important to know how fast the carbonation is penetrating the concrete and what is the cover depth of the reinforcement bars.

- b) Concrete strength becomes harder close to the surface.

Rebound measurement depends on the hardness of surface of the concrete. An increase in surface hardness increases the rebound values. The increase of this surface hardness, however, has no influence on the compressive strength of the

sample or structural concrete. Therefore, the rebound values measured on a carbonated surface must be reduced by a certain time factor (Table 5.1).

Over the range 14-56 days, surface hardness and compressive strength increase proportionally. With older concrete, surface hardness increases faster than compressive strength. The SCHMIDT hammer accordingly erroneously measure excessively high strength values (e.g. at 100 days \approx 6%) the reverse occurs in the range up to 14 days, i.e. the test hammer measures compressive strength values which are too low (e.g. at 7 days = 0-20%, according to compressive strength).

This time factor can be expressed as

$$\text{Time factor} = \frac{\text{strength afterwards}}{\text{strength before}}$$

Table 5.1: Correction factors for strength obtained from Schmidt hammer

Age t in days up to	7	50	100	200	400	800
Time factor at	1-1.12	1.0	0.94	0.87	0.79	0.7

Rebound values may be reduced by up to 40 % by using the time factor curve.

Total of 125 retaining walls were investigated in the initial phase but only 54 walls could be accessed for all information. During the investigation, the age of all retaining wall was acquired from individual house owner or respective government offices. The postal card was used to acquire the data in case of owner were not available during the investigation time.

The investigated sites are shown in figure 5.4, 5.5, and 5.6.

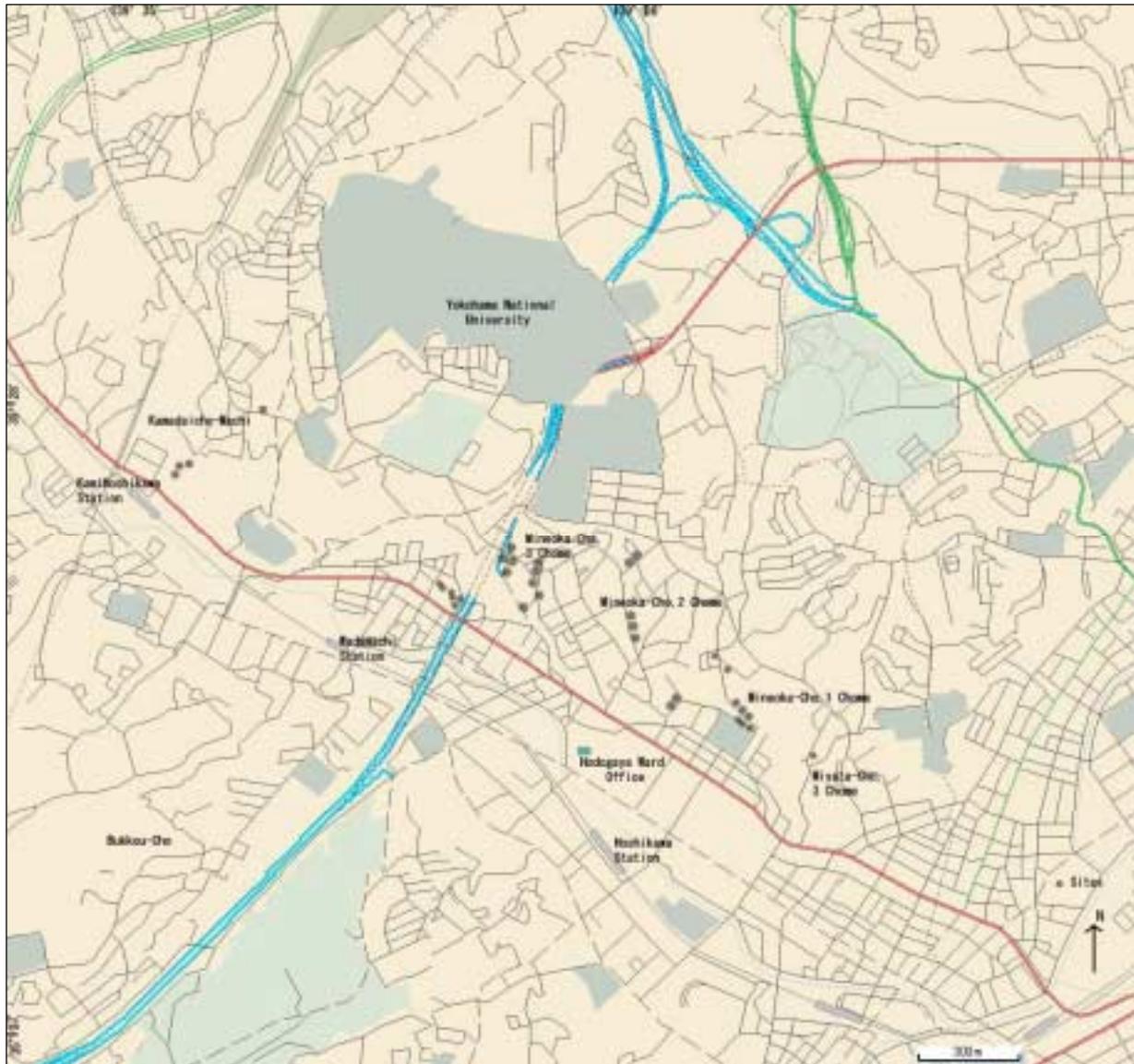


Figure 5.4: Site locations in Hodogaya ward area

The obtained results are given in table 5.2.

Table 5.2a: Wall Strength obtained from Schmidt hammer tests and its properties

Schmidt hammer data assessment and wall properties									
ID No.	Address	House No.	Const. (year)	Age	Height (m)	Length (m)	Thickness (m)	R value	Strength (kg/cm ²)
Y11	Mineoka 3 Chome		1976	29	2.75	1.93	0.20	40.4	216
Y14	Mineoka 3 Chome	306	1994	11	2.25	1.49	0.21	43.5	246
Y15	Mineoka 3 Chome	399	1994	11	2.00	1.32	0.24	37.2	186
Y16	Mineoka 3 Chome	398	1993	12	1.50	0.99	0.18	41.1	222
Y17	Mineoka 3 Chome	398-8	1995	10	1.25	0.83	0.18	40.0	222
Y18	Mineoka 3 Chome	401	1979	26	1.50	0.99	0.22	32.4	144
Y19	Mineoka 3 Chome	401-7	1979	26	2.00	1.32	0.20	41.4	225
Y20	Mineoka 3 Chome	401-6	1980	25	2.75	1.93	0.20	37.8	192
YA1	Mineoka 3 Chome	401	1979	26	1.75	1.23	0.20	37.3	189
Y22	Mineoka 3 Chome		1977	28	3.50	2.45	0.30	41.5	225
Y23	Mineoka 3 Chome		2000	5	3.60	2.70	0.25	44.7	258
Y24	Mineoka 2 Chome	219-7	1999	6	2.75	1.82	0.20	46.0	270
Y25	Mineoka		2000	5	2.30	1.52	0.21	43.3	244.2
Y26	Mineoka		2000	5	3.00	2.25	0.21	42.5	234
Y112	Mineoka 2 Chome	142-10	1995	10	1.50	0.99	0.20	43.6	246
Y113	Mineoka 2 Chome		1995	10	2.75	1.82	0.20	44.1	250.8
Y29	Mineoka 2 Chome	188-4	1998	7	1.60	1.06	0.22	42.2	232.8
YA2	Mineoka 3 Chome	438-6	2000	5	3.25	2.44	0.20	41.6	225
Y33	Mineoka 3 Chome	414	1999	6	2.50	1.65	0.20	45.1	261
Y34	Mineoka 3 Chome	413-2	1993	12	2.50	1.65	0.22	40.0	211.2
YA3	Mineoka 3 Chome		2000	5	2.25	1.49	0.21	43.2	242.4
YA4	Mineoka 3 Chome		2001	4	2.25	1.49	0.21	42.2	232.8
YA5	Mineoka 3 Chome	408	2004	1	2.35	1.55	0.21	38.9	201
Y50	Kamadaicho	8-4	2001	4	2.20	1.45	0.18	30.9	130.8
Y51	Kamadaicho	8-7	2001	4	1.75	1.16	0.20	39.5	207.6
Y52	Kamadaicho	8-10	2001	4	2.00	1.32	0.20	41.3	223.2
Y58	Kamadaicho	39-22	2002	3	1.80	1.19	0.20	46.4	273.6
Y61	Mineoka 1 Chome	1-89	1965	40	2.75	1.82	0.22	29.6	120
Y62	Mineoka 1 Chome	1-94	2001	4	2.25	1.49	0.22	32.1	141.6
Y63	Mineoka 1 Chome	94	2000	5	2.00	1.40	0.20	41.9	234
Y64	Mineoka 1 Chome		1997	8	3.50	2.45	0.20	45.2	262.8
Y65	Mineoka 1 Chome		1997	8	3.50	2.45	0.20	48.8	297
Y66	Mineoka 1 Chome	9-1	2001	4	1.00	0.66	0.20	37.9	192
Y67	Mineoka 1 Chome	98-15	1996	9	1.50	0.99	0.20	43.4	244.8
Y68	Mineoka 1 Chome	98-1	1998	7	2.00	1.32	0.20	37.1	184.8
Y70	Mineoka 1 Chome	100	1964	41	2.50	1.65	0.30	31.9	140.4
Y73	Miyatacho 3 chome	307	1983	22	4.25	3.19	0.20	41.3	211.2
Y86	Nishinoya-cho	114-33	1990	15	2.85	1.88	0.28	44.9	260.4
Y87	Mameguchi-dai	98	1989	16	2.75	1.82	0.20	43.9	253.2

Table 5.2b: Wall Strength obtained from Schmidt hammer tests and its properties

Schmidt hammer data assessment and wall properties									
ID No.	Address	House no.	Const. (yr)	Age	Height (m)	Length (m)	Thickness (m)	R value	Strength (kg/cm ²)
C11	Ikegami 1 Chome	17-19	2002	3	1.25	0.83	0.18	49.0	300
C12	Ikegami 1 Chome	17-17	2002	3	2.10	1.39	0.22	46.9	278.4
C13	Ikegami 1 Chome	17-15	2004	1	3.50	2.45	0.22	38.8	201
C15	Ikegami 1 Chome		2004	1	1.65	1.09	0.20	42.8	238.8
C16	Ikegami 1 Chome	20-26	2004	1	1.75	1.16	0.20	42.0	236.4
C17	Ikegami 1 Chome	20-26	2004	1	1.75	1.16	0.20	40.0	211.2
C18	Sanno 4 Chome	32-11	1960	45	4.00	2.80	0.22	33.4	152.4
C21	Sanno 4 Chome	32-12	1972	33	2.50	1.65	0.22	35.3	175.2
C23	Sanno 4 Chome	32-13	1976	29	2.25	1.49	0.20	39.3	207
C26	Sanno 3Chome	44-9	1978	27	3.75	2.89	0.25	37.7	190.8
C27	Sanno 3 Chome	44-4	1980	25	4.15	3.11	0.30	40.2	273
C30	Chuo 5 Chome	29-1	1980	25	3.75	2.63	0.22	44.5	254.4
C36	Chuo 6 Chome	6-5-1	2000	5	3.25	2.28	0.20	40.238	214.8
C37	Chuo 5 Chome	8-13	1974	31	3.00	1.98	0.30	36.333	178.8
C40	Minami-magome	42-18	1982	23	2.50	1.65	0.20	40.704	177.6

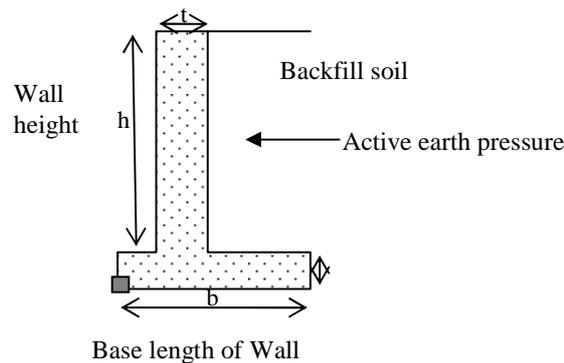


Figure 5.7: Basic configuration of cantilever type retaining wall

The height of the retaining wall was taken from field survey. The length of base was estimated to be $2/3$ of h . Thickness of base was taken to be at least 20 cm. Length from toe to column of wall is taken to be $h/8$ as a common use or at least 20 cm. There are some uncertainties on these parameters which are considered in chapter 7.

5.7. Results and Discussions

The present strength value and the age of the retaining wall shows the strength decreasing pattern (Fig. 5.8), even it may have some limitations. Since, all the retaining walls may not have the exactly same initial strength value, it may have

some uncertainty. The design strength as defined by the regulation of Building Standard Law at the time of construction is assumed to be the minimum value of the retaining walls during construction. Building Standard Laws (1986) defines the minimal value for such walls need to have minimum 120 kg/cm². Present minimal value is 180 kg/cm² since 1998.

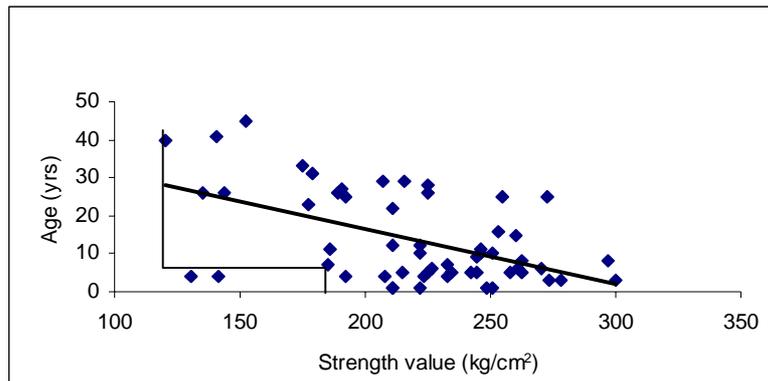


Figure 5.8: Strength of wall and Age relationship

The change in the minimum strength value defined by Building Standard Law can also be noticed from the figure 5.8. If we see the mean value for 5 year range, the strength decreasing pattern can be noticed significantly (Fig. 5.9). The strength in 40 years seems to be decreased by around 40-60 % if compared with strength of present concrete retaining walls.

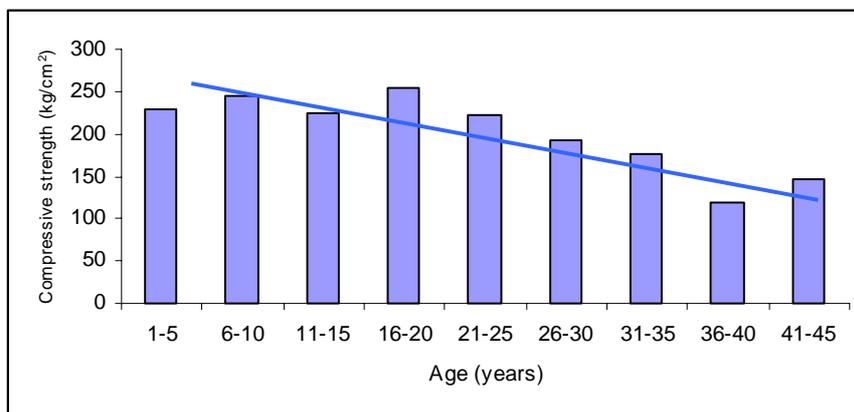


Figure 5.9: Age and compressive strength relation (in 5 years mean range)

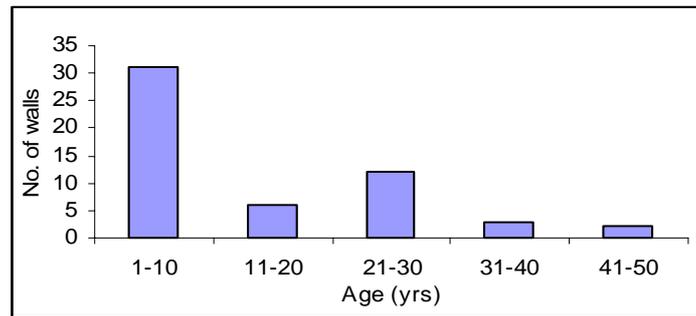


Figure 5.10: Age range of retaining wall in the study area

The most of the constructions of retaining wall are found within last 10 years (Fig. 5.10). Many constructions were also seen during the age of 21-30 and it may be because of the heavy construction in all around Japan.

From the above analysis, the Schmidt hammer strength and age has shown the strength reducing pattern of concrete walls. Recent construction wall types are mainly concrete retaining walls. So, the study of concrete retaining wall safety is important for the society.

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Chapter 6

**Probabilistic Seismic
Hazard in the Study Area**

6.1. Introduction

This chapter deals with the seismic hazard in the study area. The probabilistic seismic hazard assessment is obtained for each field site localities. This seismic hazard information will be used to estimate the probability of failure of retaining walls due to the possible earthquakes in respective field sites.

Seismic hazard can be estimated by many methods such as observational method, deterministic method, statistical method and probabilistic seismic hazard assessment method. Earliest attempts for seismic hazard were done by observational methods. It just assumes high hazard in the areas having lots of earthquake or high intensity distribution. The deterministic methods are based on following procedures.

- a) Find the nearest active fault
- b) Calculate the largest earthquake that could happen on this fault
- c) Assume the largest earthquake happens at the closest point to your site
- d) Calculate what the ground motion will be

This deterministic method will give conservative estimates. The statistical methods are based on observational data by which probability of future events can be predicted. This relies on the method of extreme values of earthquake ground motions at the interested sites. These extreme values are applied for probability distribution for certain period and hazard is obtained.

Probabilistic method for seismic hazard assessment is the one of widely used one. In this method, the source zone can be demarcated assuming that earthquakes have an equal probability of occurring at any spot in the interested area. The statistical data and probability for future are estimated to find the probability of an earthquake of a given intensity occurring in the same area in some future period. The attenuation relation, source distance, and then earthquake intensity is considered in such analysis (Cornell 1968). The seismic source zone model is defined by interpretive decisions based on both geological and seismological data. The time dependent seismic hazard method is also considered to find the probabilistic seismic hazard assessment. Limited data sets increase the uncertainty

in the hazard calculations (Petersen et. al. 2004), so larger data set is preferred to use for hazard assessment.

There are different data sources to find out the probabilistic seismic hazard assessment in Japan. Some of sources include National Research Institute for Earth Sciences and Disaster Prevention (NIED), Earthquake Research Institute in The University of Tokyo, Program for Structural Performance Evaluation of a Building (SS-Web), Japan Meteorological Agency etc. I have used the sources of National Research Institute for Earth Sciences and Disaster Prevention provided at <http://www.bosai.go.jp/jpn/jishin.htm>.

6.2. Factors Effecting on Seismic Damage

Two different scenarios can be taken to cause seismic damage: seismic hazard and performance of a structure.

The seismic hazard depends on the parameters like source of earthquake, fault characteristics, distance from the fault, intensity of earthquake, ground condition etc., whereas the performance of the structure depends on the design parameters of that structure. So, the damage or failure of a structure can have lots of parameters expressing the complex system of hazard assessment.

6.3. General Ground Condition of the Area

In the study area, there is large spatial variability in the ground condition. The ground condition of the area can have a greater influence to the probability of failure of structures on the upper most soil layer in the area. A layer of soft clay, deposited by a marine transgression during a Quaternary post-glacial stage, occurs extensively in the Kanto plain surrounding Tokyo Bay (Baba 1994).

The transgression, more widely the Yurakucho transgression, was characterized by more than 20 meters thick, forming alluvium deposits in the large plain of Tokyo. The Arakawa River had contributed for the terrace deposits in the southern part of the Tokyo, represents some part of the study area. This soft clay is also considered to be highly sensitive and may cause slope failures even in gentle sloppy areas in the region (Baba 1994).

The soil deposits in the study area are characterized as follows:

Southern part of Kanto consists of Kanto loam and its uppermost layer Tachikawa Loam Formation (Kato and Matsui 1979). Engineering properties of soils are important to prepare the maps for disaster mitigation systems. Subsurface geology, geomorphological land classifications, and agricultural soil type is used for such maps (Wakamatsu et. al. 2001). The soil data of Yokohama area consists of hills of Quaternary deposits on the top and alluvial fan or valley deposits on the lower parts of the area. But Ota ward area has four different types of soil layer, representing mostly soft soil ground as follows: A1, A2, B1, and C.

A1- It has alluvium of Yurakucho Upper layer at the top and Tokyo Layer at the bottom. Sometimes, it may consist three layers, Yurakucho Upper Layer, Yurakucho Lower Layer and Tokyo Layer respectively from top to bottom. The thickness of weak layer is less than 10 m.

A2- It has alluvium consisting Yurakucho Upper Layer, Yurakucho Lower Layer and 7th Layer from top to bottom. The thickness of weak layer ranges from 10-30 m.

B1- The diluvial upland deposits of Kanto Loam, Loam Clay Layer, and Tokyo Layer from top to bottom.

C- The soil deposits on low land river valley have humus and humic-clays. This layer is stretched in valleys.

6.4. Assessment of Seismic Hazard

The data source of NIED was used to obtain the hazard curve for each area. The source data consists of exceeding probability for seismic hazard in 50 years. It was converted into non exceeding probability for its application. The probability of earthquake occurrence and respective peak ground velocity (PGV) at the surface were available from NIED data. To find the seismic hazard curve of different field sites, required correction factor was applied to the ground velocity of the nearby reference location given by NIED. The complete data set for PGA at the seismic site in Yokohama lies at Latitude 35.4544° and Longitude 139.6414° while the reference for Ota ward lies at Latitude 35.6876° and Longitude 139.6937° . With taking reference of seismic location, there can be obtained the correction factor for the interested field site.

The field sites within the range of one square kilometer grid were assumed to have a same seismic hazard because the source data is only available on one square kilometer grid area. So, one grid area may have many field sites. All 54 site locations were grouped into 7 localities or zones (Table 6.1 and 6.2) which are named by area ID for easy and they are based on grid limitation. Each locality represents all sites lying within that grid.

The PGV data were available from NIED, which were later converted into PGA, using the relation between them. The PGA (Peak Ground Acceleration) was obtained using the relation given by AIJ Load Recommendation 1996. It was the recommendation given in 1993 (in Japanese). According to this recommendation, PGV and PGA relation comes nearly 1:15 for firm alluvial or soft diluvial while for soft soil it comes nearly 1:9. In the Ota area, the ground condition dominated by soft soil layer may have smaller ratio of PGA and PGV than that of little stiff ground of Hodogaya and Naka area. So, 1:10 ratio was taken for Ota area and 1:13 was taken for Hodogaya and Naka area.

Hodogaya ward consists of 3 localities as follows:

Table 6.1: Three localities of field sites in Hodogaya and one in Naka ward

No.	SE part of Mineoka	Area ID	No.	NW part of Mineoka	Area ID
Y11	Mineoka 3 Chome	HM3	Y61	Mineoka 1 Chome	1 – 89 HM1
Y14	Mineoka 3 Chome	306 HM3	Y62	Mineoka 1 Chome	1-94 HM1
Y15	Mineoka 3 Chome	399 HM3	Y63	Mineoka 1 Chome	94 HM1
Y16	Mineoka 3 Chome	398 HM3	Y64	Mineoka 1 Chome	HM1
Y17	Mineoka 3 Chome	398-8 HM3	Y65	Mineoka 1 Chome	HM1
Y18	Mineoka 3 Chome	401 HM3	Y66	Mineoka 1 Chome	9-1 HM1
Y19	Mineoka 3 Chome	401-7 HM3	Y67	Mineoka 1 Chome	98-15 HM1
Y20	Mineoka 3 Chome	401-6 HM3	Y68	Mineoka 1 Chome	98-1 HM1
YA1	Mineoka 3 Chome	401 HM3	Y70	Mineoka 1 Chome	100 HM1
Y22	Mineoka 3 Chome	HM3	Y73	Miyata-cho 3 Chome	307 HM1
Y23	Mineoka 3 Chome	HM3	Y112	Mineoka 2 Chome	142-10 HM1
Y24	Mineoka 2 Chome	219-7 HM3	Y113	Mineoka 2 Chome	HM1
Y25	Mineoka 2 Chome	HM3	Kamadaicho area		
Y26	Mineoka 3 Chome	HM3	Y50	Kamadaicho	8-4 Hka
Y29	Mineoka 2 Chome	188-4 HM3	Y51	Kamadaicho	8-7 Hka
YA2	Mineoka 3 Chome	438-6 HM3	Y52	Kamadaicho	8-10 Hka
Y33	Mineoka 3 Chome	414 HM3	Y58	Kamadaicho	39-22S Hka
Y34	Mineoka 3 Chome	413-2 HM3	Naka ward area		
YA3	Mineoka 3 Chome	HM3	Y86	Nishinoya-cho	114-33 NK
YA4	Mineoka 3 Chome	HM3	Y87	Mameguchi-dai	98 NK
YA5	Mineoka 3 Chome	408 HM3			

The seismic hazard curve in seismic hazard reference location in Yokohama area and Ota area are given in figure 6.1a and 6.1b respectively. Location of the reference site at Yokohama area is located at Latitude 35.4544° and Longitude 139.6414° . Nearest reference site at Ota area is located at Latitude 35.6876° and Longitude 139.6937° . These two locations are taken as the reference to find hazard curve for specific sites.

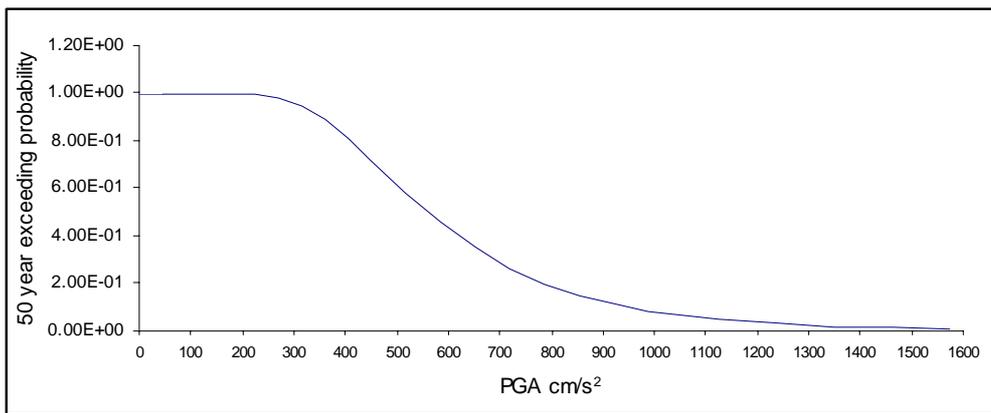


Figure 6.1a.: Seismic hazard curve at reference location in Yokohama area

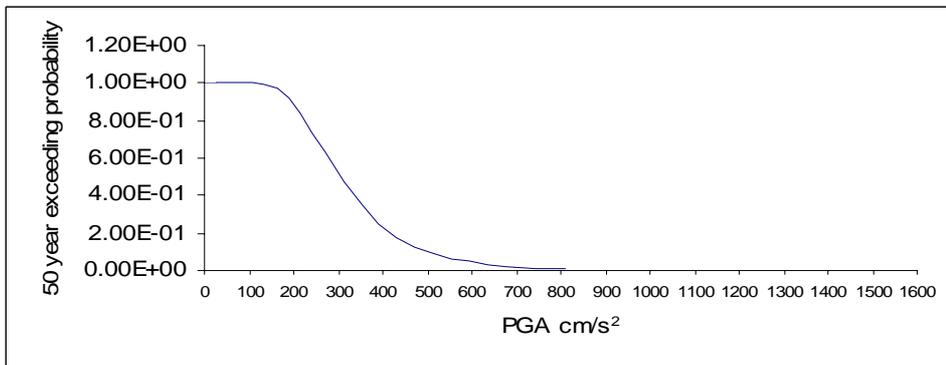


Figure 6.1b.: Seismic hazard curve at reference location in Ota area

The corresponding seismic hazard maps obtained for different localities in Hodogaya area are as follows:

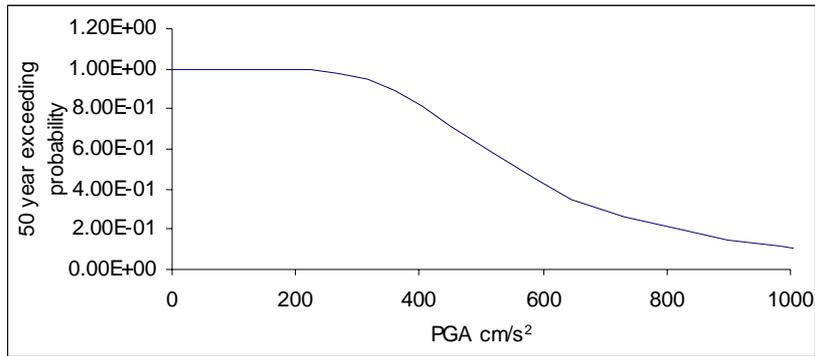


Figure 6.2: Seismic hazard curve for SE and central part of Mineoka Cho (HM1)

Hazard curve for south-east and central part of Mineoka Cho was taken at location having Latitude 35.45833°, Longitude 139.6028°. There are total of 21 field sites that were studied. Ground condition consists of quaternary deposits. The hazard curve for PGV was quite similar ground condition to the reference location in Yokohama.

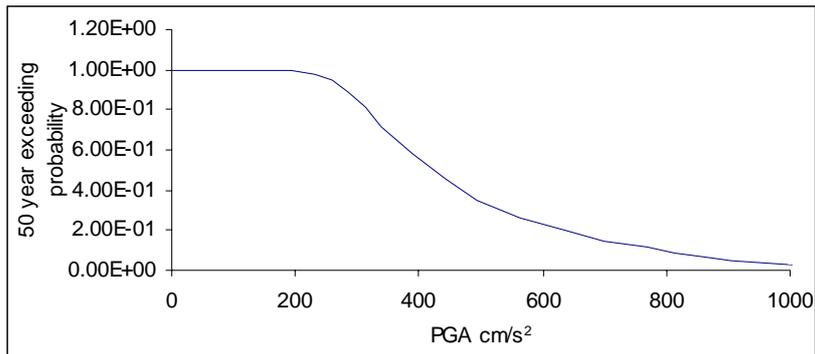


Figure 6.3 Seismic hazard curve for NW part of Mineoka Cho (HM3)

Seismic hazard curve for north-west part of Mineoka Cho consists of 12 field sites. The location of this is taken as Latitude 35.46111°, Longitude 139.5972°.

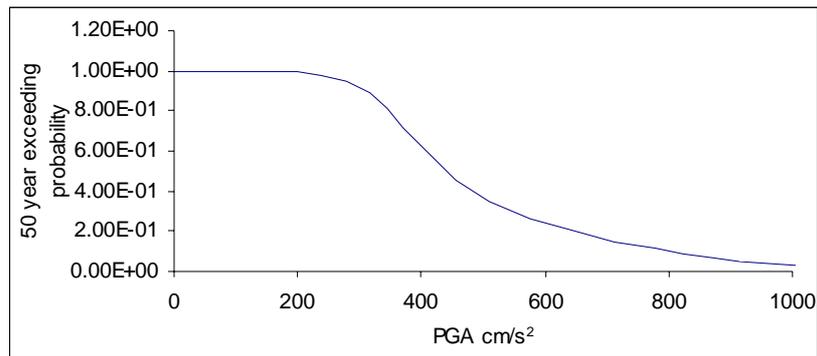


Figure 6.4 Seismic hazard curve for Kamadaicho area (Hka)

The location taken for Kamadaicho area has Latitude 35.46667° , Longitude 139.5847° . It also consists of quaternary deposits. The corresponding seismic hazard curve in PGA is given in figure 6.4

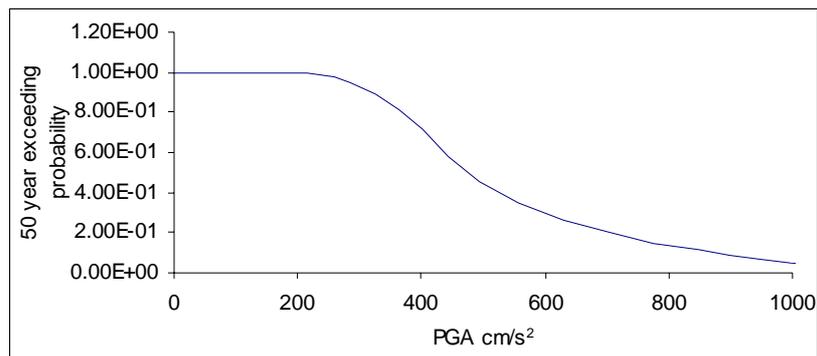


Figure 6.5 Seismic hazard curve for Naka ward area (Nk)

There are only two sites in the Naka ward. It has alluvium layer or valley deposits. The hazard curve is given in figure 6.5 The location of this area is taken at Latitude 35.42222° , Longitude 139.65° .

Ota ward also has three different localities- Ikegami area, Sanno-4Chome and 3Chome, and Minami-magome area (Table 6.2). Ikegami, and Sanno area has Yurakucho Upper Layer and Tokyo Layer where as Minami-magome area has Kanto Loam as well as Loam Clay Layer.

Table 6.2: Three localities of field sites in Ota ward area

No.	Ikegami area	Area ID	No.	Sanno area	Area ID		
C11	Ikegami 1 Chome	17-19	Olk1	C18	Sanno 4 Chome	32-11	Osan
C12	Ikegami 1 Chome	17-17	Olk1	C21	Sanno 4 Chome	32-12	Osan
C13	Ikegami 1 Chome	17-15	Olk1	C23	Sanno 4 Chome	32-13	Osan
C15	Ikegami 1 Chome		Olk1	C26	Sanno 3 Chome	44-9	Osan
C16	Ikegami 1 Chome	20-26	Olk1	C27	Sanno 3 Chome	44-4	Osan
C17	Ikegami 1 Chome	20-26	Olk1	Minami-magome area			
C30	Chuo 5 Chome	29-1	Olk1	C37	Chuo 5 Chome	8-13	OMg1
C36	Chuo 6 Chome	6-5-1	Olk1	C40	Minami-magome	42-18	OMg1

The location for Ikegami area was taken for Latitude 35.57222°, Longitude 139.7111°. It has 8 field sites in this area.

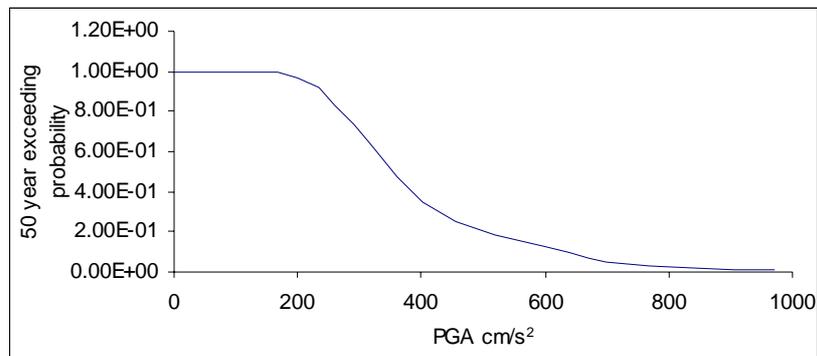


Figure 6.6 Seismic hazard curve for Ikegami area (Olk1)

Sanno area consists of 5 field sites and the location of hazard curve for this area is taken at Latitude 35.58611°, Longitude 139.725°.

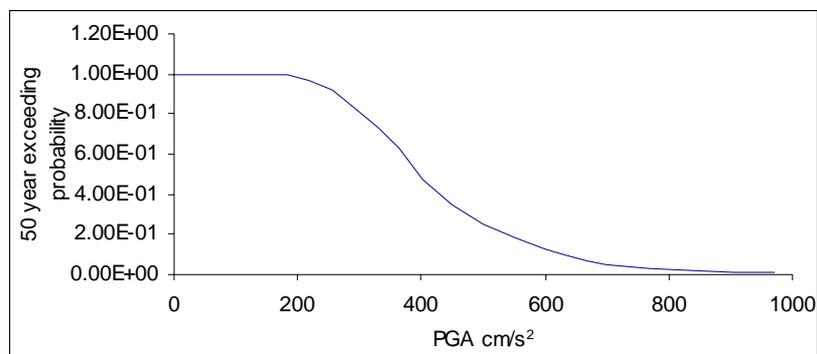


Figure 6.7: Seismic hazard curve for Sanno area

Two field sites in Minami-magome area were studied and the location for this area was taken at Latitude 35.57778°, Longitude 139.7111°. The corresponding hazard curve is shown in figure 6.8.

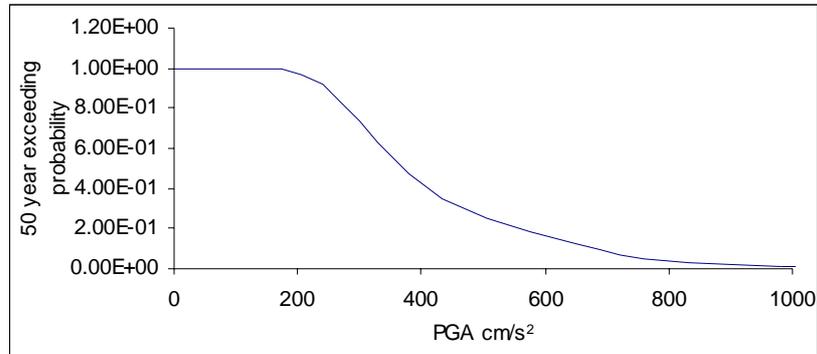


Figure 6.8 Seismic hazard curve for Minami-magome area (Omg)

The seismic hazard of the field sites is found to have some deviation from the nearest reference location. Sites in Ikegami area show relatively lower seismic hazard than in Sanno and Minami-magome area. Seismic hazard for the sites in the south-east part of Mineoka cho (HM1) shows higher probability than in other sites of Hodogaya area.

Thus, this chapter provides the seismic hazard of each site. This information will be used in finding the probability of failure of retaining wall in respective sites.

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Website links:

NIED: <http://www.bosai.go.jp/jpn/jishin.htm>

SS-Web : <http://ssweb.k.u-tokyo.ac.jp/indexe.htm>

Chapter 7

Failure Probability of Retaining Walls

7.1. Introduction

This chapter will describe the results obtained from the calculation for different mode of failures to assess the safety of retaining walls. The overturning moment checked by resisting moment, sliding force checked by resisting force, and compressive and shear stress resisted by strength of the wall are analyzed. The dominant mode of failure was found as overturning mode which is described with the probabilistic approach.

Practical design procedures are established to evaluate the seismic stability of different types of retaining walls against the applied higher seismic loads. These loads are considered in designing to ensure the safety of structure. To withhold the wall from seismic load, we need sufficient level of safety factor. The applied load can be computed precisely from pseudostatic method (Greco 2003). One of the widely used method Coulomb method was applied for static load and Mononobe Okabe Method for the load in dynamic condition.

Earthquake load has uncertainty on the occurrence probability of the earthquakes. This uncertainty was obtained as seismic hazard curve for the different site areas. When the earthquakes occur, it still has many parameters to influence the applying load to each retaining wall. The influencing parameters include the intensity of earthquake ground motion, ground soil behavior and variability etc.

Uncertainties in the resisting load are considered to be built up from the weight of the soil mass behind the retaining walls. This resisting load has less variability than the applied load.

7.2. Assumption of Wall Configuration

For the calculation of the resisting and applied load, the configuration of the wall governs key role for the safety of the structure. During the field survey, the wall parameters like wall height, strength on the surface, thickness etc were taken. Almost all walls are in similar arrangement in the exposed face although there may have differences in some properties of the wall. Many of structures are now made of reinforced concrete construction and majority of them are made into earthquake resistant construction with supporting structural calculations (AIJ 1970). Such retaining walls were assumed to have the configuration of cantilever type retaining wall with

vertical wall face and no surcharge (Fig. 5.7). The wall properties that are used in the calculation are given in Table 5.2. The design regulation can play significant role to make the similarity of construction. Sometimes, the construction company that worked for construction of housing lots was specified with similar type of construction. The construction of housing lots is more dominant in Hodogaya area.

Thus the assumption of wall configuration may have some uncertainty. In getting the some detailed information of the retaining walls, we can reduce this uncertainty. It can be assumed that this uncertainty may vary up to 10% in resulting the resisting moment or forces. In case of sliding, the resisting force is much dependent on the base length and soil upon it. So, the coefficient of variation can be considered up to 30%. But with getting all parameters precisely, we can apply this methodology to find out more accurate results. So, this method will be applicable to other retaining walls to find the probability of failure in other areas.

7.3. Analysis of Overturning and Resisting Moment

In the designing stage, the stability of retaining walls against overturning is generally evaluated through a safety factor, F , which is the ratio between the sum of resisting moments and the sum of overturning moments with respect to the toe of the wall. The calculation of the moment was taken from the toe of the retaining wall. The static component of the earth pressure was calculated by Coulomb's theory (equation 3.4). The total applied seismic load due to earthquakes was also calculated by Mononobe Okabe method (equation 3.7). It consists of both static and dynamic components. The dynamic component was also separately calculated by the methods given by Kramer (1996) (equation 3.11)) and verified with the results from Mononobe Okabe method. There was the same result for each case. There will be the redistribution of applied stress during the earthquake (Nadim et.al. 1983). Considering this, the point of act of the total thrust can be calculated as given by Seed and Whitman (1970) equation 3.12.

The overturning moment based on the effective height of the wall can be calculated using equation 3.13. The load exerted by the soil behind the wall is main load for the resisting force. It also needs the separate calculation of load of concrete wall, its base, and the soil weight on the base of the wall. So, resisting moment depends mainly upon weight of soil, height of wall and wall base or its thickness. This was calculated using

method as given in chapter 3.4.1. The backfill soil material is commonly used dry sandy soil. So, the unit weight is taken to be 18 kN/m³. The unit weight of the concrete wall is taken to be 24 kN/m³. The analysis of driving moment over resisting moment is given in table 7.1.

Table 7.1a: Analysis of driving moment

ID	M _{resist}	Driving moment at different PGA (in kN, (PGA in cm/s ²))														
		(PGA=)	100	200	250	300	350	400	450	500	550	600	700	900	1000	1200
Y11	37.44	25.07	33.14	37.17	41.21	45.24	49.27	53.31	57.34	61.38	65.41	73.48	89.61	97.68	113.82	129.96
Y14	23.85	13.73	18.15	20.36	22.57	24.78	26.99	29.20	31.41	33.62	35.83	40.24	49.08	53.50	62.34	71.18
Y15	21.38	9.64	12.75	14.30	15.85	17.40	18.95	20.51	22.06	23.61	25.16	28.27	34.47	37.58	43.78	49.99
Y16	11.54	4.07	5.38	6.03	6.69	7.34	8.00	8.65	9.31	9.96	10.62	11.92	14.54	15.85	18.47	21.09
Y17	8.58	2.35	3.11	3.49	3.87	4.25	4.63	5.01	5.39	5.76	6.14	6.90	8.42	9.17	10.69	12.20
Y18	11.54	4.07	5.38	6.03	6.69	7.34	8.00	8.65	9.31	9.96	10.62	11.92	14.54	15.85	18.47	21.09
Y19	16.83	9.64	12.75	14.30	15.85	17.40	18.95	20.51	22.06	23.61	25.16	28.27	34.47	37.58	43.78	49.99
Y20	35.98	25.07	33.14	37.17	41.21	45.24	49.27	53.31	57.34	61.38	65.41	73.48	89.61	97.68	113.82	129.96
YA1	18.70	6.46	8.54	9.58	10.62	11.66	12.70	13.74	14.78	15.82	16.86	18.94	23.09	25.17	29.33	33.49
Y22	84.87	51.68	68.32	76.63	84.95	93.27	101.58	109.90	118.22	126.53	134.85	151.48	184.75	201.38	234.65	267.92
Y23	97.25	56.24	74.34	83.39	92.44	101.49	110.54	119.59	128.64	137.69	146.74	164.84	201.04	219.14	255.34	291.54
Y24	38.78	25.07	33.14	37.17	41.21	45.24	49.27	53.31	57.34	61.38	65.41	73.48	89.61	97.68	113.82	129.96
Y25	26.96	14.67	19.39	21.75	24.11	26.47	28.83	31.19	33.55	35.91	38.27	42.99	52.43	57.15	66.59	76.03
Y26	54.81	32.55	43.02	48.26	53.50	58.73	63.97	69.21	74.45	79.68	84.92	95.40	116.34	126.82	147.77	168.72
Y112	13.38	4.07	5.38	6.03	6.69	7.34	8.00	8.65	9.31	9.96	10.62	11.92	14.54	15.85	18.47	21.09
Y113	38.78	25.07	33.14	37.17	41.21	45.24	49.27	53.31	57.34	61.38	65.41	73.48	89.61	97.68	113.82	129.96
Y29	13.25	4.94	6.53	7.32	8.12	8.91	9.70	10.50	11.29	12.09	12.88	14.47	17.65	19.24	22.42	25.60
YA2	72.02	41.38	54.70	61.36	68.02	74.68	81.33	87.99	94.65	101.31	107.97	121.29	147.92	161.24	187.87	214.51
Y33	29.34	18.84	24.90	27.93	30.96	33.99	37.02	40.05	43.08	46.11	49.14	55.21	67.33	73.39	85.51	97.64
Y34	29.34	18.84	24.90	27.93	30.96	33.99	37.02	40.05	43.08	46.11	49.14	55.21	67.33	73.39	85.51	97.64
YA3	26.38	13.73	18.15	20.36	22.57	24.78	26.99	29.20	31.41	33.62	35.83	40.24	49.08	53.50	62.34	71.18
YA4	26.38	13.73	18.15	20.36	22.57	24.78	26.99	29.20	31.41	33.62	35.83	40.24	49.08	53.50	62.34	71.18
YA5	27.54	15.64	20.68	23.20	25.71	28.23	30.75	33.27	35.78	38.30	40.82	45.85	55.92	60.96	71.03	81.10
Y50	25.32	12.84	16.97	19.03	21.10	23.16	25.23	27.29	29.36	31.42	33.49	37.62	45.88	50.01	58.28	66.54
Y51	17.07	6.46	8.54	9.58	10.62	11.66	12.70	13.74	14.78	15.82	16.86	18.94	23.09	25.17	29.33	33.49
Y52	21.38	9.64	12.75	14.30	15.85	17.40	18.95	20.51	22.06	23.61	25.16	28.27	34.47	37.58	43.78	49.99
Y58	17.88	7.03	9.29	10.42	11.56	12.69	13.82	14.95	16.08	17.21	18.34	20.61	25.13	27.39	31.92	36.44
Y61	38.78	25.07	33.14	37.17	41.21	45.24	49.27	53.31	57.34	61.38	65.41	73.48	89.61	97.68	113.82	129.96
Y62	26.38	13.73	18.15	20.36	22.57	24.78	26.99	29.20	31.41	33.62	35.83	40.24	49.08	53.50	62.34	71.18
Y63	18.66	9.64	12.75	14.30	15.85	17.40	18.95	20.51	22.06	23.61	25.16	28.27	34.47	37.58	43.78	49.99
Y64	82.53	51.68	68.32	76.63	84.95	93.27	101.58	109.90	118.22	126.53	134.85	151.48	184.75	201.38	234.65	267.92
Y65	71.50	51.68	68.32	76.63	84.95	93.27	101.58	109.90	118.22	126.53	134.85	151.48	184.75	201.38	234.65	267.92
Y66	6.06	1.21	1.59	1.79	1.98	2.18	2.37	2.56	2.76	2.95	3.15	3.53	4.31	4.70	5.47	6.25
Y67	13.38	4.07	5.38	6.03	6.69	7.34	8.00	8.65	9.31	9.96	10.62	11.92	14.54	15.85	18.47	21.09
Y68	21.38	9.64	12.75	14.30	15.85	17.40	18.95	20.51	22.06	23.61	25.16	28.27	34.47	37.58	43.78	49.99
Y70	31.32	18.84	24.90	27.93	30.96	33.99	37.02	40.05	43.08	46.11	49.14	55.21	67.33	73.39	85.51	97.64
Y73	152.54	92.54	122.32	137.21	152.10	166.99	181.88	196.77	211.66	226.55	241.44	271.23	330.79	360.57	420.13	479.69
Y86	50.84	27.91	36.89	41.38	45.87	50.36	54.85	59.34	63.83	68.32	72.81	81.79	99.75	108.73	126.69	144.65
Y87	38.78	25.07	33.14	37.17	41.21	45.24	49.27	53.31	57.34	61.38	65.41	73.48	89.61	97.68	113.82	129.96

Table 7.1b: Analysis of driving moment

ID	M _{resist}	Driving moment at different PGA (in kN m-m, (PGA in cm/s ²))														
		100	200	250	300	350	400	450	500	550	600	700	900	1000	1200	1400
C11	10.23	2.35	3.11	3.49	3.87	4.25	4.63	5.01	5.39	5.76	6.14	6.90	7.66	8.42	9.17	10.69
C12	22.34	11.16	14.76	16.55	18.35	20.15	21.94	23.74	25.54	27.33	29.13	32.72	36.31	39.91	43.50	50.68
C13	76.55	51.68	68.32	76.63	84.95	93.27	101.58	109.90	118.22	126.53	134.85	151.48	168.12	184.75	201.38	234.65
C15	15.52	5.42	7.16	8.03	8.90	9.77	10.64	11.51	12.39	13.26	14.13	15.87	17.61	19.36	21.10	24.58
C16	17.07	6.46	8.54	9.58	10.62	11.66	12.70	13.74	14.78	15.82	16.86	18.94	21.01	23.09	25.17	29.33
C17	17.07	6.46	8.54	9.58	10.62	11.66	12.70	13.74	14.78	15.82	16.86	18.94	21.01	23.09	25.17	29.33
C18	92.82	77.15	101.98	114.39	126.81	139.22	151.64	164.05	176.46	188.88	201.29	226.12	250.95	275.78	300.61	350.26
C21	28.50	18.84	24.90	27.93	30.96	33.99	37.02	40.05	43.08	46.11	49.14	55.21	61.27	67.33	73.39	85.51
C23	23.85	13.73	18.15	20.36	22.57	24.78	26.99	29.20	31.41	33.62	35.83	40.24	44.66	49.08	53.50	62.34
C26	106.80	63.57	84.03	94.26	104.49	114.72	124.94	135.17	145.40	155.63	165.86	186.32	206.78	227.24	247.69	288.61
C27	160.26	86.16	113.89	127.75	141.62	155.48	169.34	183.21	197.07	210.93	224.80	252.53	280.25	307.98	335.71	391.17
C30	94.53	63.57	84.03	94.26	104.49	114.72	124.94	135.17	145.40	155.63	165.86	186.32	206.78	227.24	247.69	288.61
C36	64.57	41.38	54.70	61.36	68.02	74.68	81.33	87.99	94.65	101.31	107.97	121.29	134.60	147.92	161.24	187.87
C37	53.52	32.55	43.02	48.26	53.50	58.73	63.97	69.21	74.45	79.68	84.92	95.40	105.87	116.34	126.82	147.77
C40	32.16	18.84	24.90	27.93	30.96	33.99	37.02	40.05	43.08	46.11	49.14	55.21	61.27	67.33	73.39	85.51

This analysis shows the overturning moment exerted by the earth pressure during earthquake ground motion and the resisting moment of the wall. The total moment generated due to different intensity of earthquake ground motion has been analyzed so that we can find out which intensity has possibility to damage the retaining wall. When the overturning moment will overcome the resisting moment we assume the failure probability of the retaining walls. In the designing, there would have required factor of safety of 1.5. But in the analysis of failure probability of existing retaining wall, the factor of safety is not considered. Rather than the factor of safety, the equilibrium of the moment or forces will be considered for the stability. The horizontal coefficient of earth pressure increases with the change of earthquake ground motion intensity. The interface frictional angle between wall and soil is taken to be 20.

In case of Hodogaya and ward, the failure distribution in different ground motion intensity of earthquake is found as:

Table 7.2: Failure distribution in Hodogaya and Naka ward area

Intensity (PGA, cm/s ²)	200	250	300	350	400	450	500	550	600	700	900	1000	1200	1400
No of Failure cases	0	2	9	8	6	1	3	-	1	5	2	1	-	1
Total failures		2	11	19	25	26	29	29	30	35	37	38		39

The failure has started from 250 gal which is not so large scale of intensity. Most of the damage cases are observed from 300 to 400 gal. The highly resistive wall are seemed to be damaged by 1400 gal.

The Naka ward has two sites, which will face the failure at 300 gal by Y87 no wall and at 400 gal by Y86. In Ota ward, the failure distribution in different intensity of ground motion is found as:

Table 7.3: Failure distribution in different intensity of earthquake in Ota wards area

Intensity (PGA,cm/s ²)	200	250	300	350	400	450	500	550	600	700	900	1000	1200	1400
No of Failure cases	1	1	3	4	1	1	-	-	-	3	-	-	1	-
Total failures	1	2	5	9	10	11	11	11	11	14	14	14	15	-

One retaining wall was found to fail with the intensity of 200 gal. Most of failure cases are observed in 300-400 gal though total number of walls is also small. Four retaining walls seem to sustain the intensity of 600 gal and will be failing by 700 gal, and another one by 1200 gal.

7.4. Analysis of Sliding and Resisting Forces

The sliding force is developed at the base of the retaining walls. The horizontal component of earth pressure tends to slide the retaining wall along the plane of its base. This force is resisted by the shear resisting force created between the foundation ground and base. The resisting force is dependent upon the vertical load applied on the base of the wall and adhesion between base and foundation ground.

The cohesive force of the base soil is different in sites of Ota ward and Hododaya ward. Kanto Loam has ranges of cohesive force from 30-40 kN/m². In general case, the two third of cohesive force is taken as the adhesive force between the base of wall and soil. In case of soft soil as in Ota area like Kanto Loam, the cohesive force 35 kN/m² is taken while in Hodogaya area, 25 kN/m² is taken for having some silts and fines. The resisting force is calculated using the equation (3.14) and sliding force was the acting earth pressure force at the base of the wall. The results are given in table 7.4.

Table 7.4a: Analysis of sliding failure

ID	F _{resist}	Analysis of sliding and resisting force (in kN,(PGA in cm/s ²))											
	(PGA->)	350	400	450	500	550	600	700	800	900	1000	1100	1400
Y11	43.90	40.70	43.69	46.68	49.68	52.67	55.67	61.65	67.64	73.63	79.62	85.61	103.57
Y14	33.34	27.67	29.73	31.80	33.86	35.93	37.99	42.12	46.25	50.38	54.51	58.64	71.04
Y15	29.70	22.10	23.76	25.43	27.09	28.76	30.42	33.75	37.08	40.41	43.74	47.07	57.06
Y16	20.66	12.83	13.83	14.82	15.82	16.81	17.80	19.79	21.78	23.77	25.76	27.75	33.71
Y17	16.84	9.14	9.86	10.59	11.31	12.03	12.76	14.20	15.65	17.10	18.54	19.99	24.33
Y18	20.66	12.83	13.83	14.82	15.82	16.81	17.80	19.79	21.78	23.77	25.76	27.75	33.71
Y19	28.06	22.10	23.76	25.43	27.09	28.76	30.42	33.75	37.08	40.41	43.74	47.07	57.06
Y20	41.51	40.70	43.69	46.68	49.68	52.67	55.67	61.65	67.64	73.63	79.62	85.61	103.57
YA1	27.15	17.15	18.46	19.77	21.08	22.38	23.69	26.31	28.92	31.54	34.16	36.77	44.62
Y22	71.39	64.94	69.65	74.36	79.07	83.78	88.49	97.91	107.33	116.75	126.17	135.59	163.85
Y23	64.55	68.60	73.57	78.54	83.50	88.47	93.44	103.38	113.31	123.25	133.18	143.12	172.93
Y24	44.21	40.70	43.69	46.68	49.68	52.67	55.67	61.65	67.64	73.63	79.62	85.61	103.57
Y25	35.00	28.86	31.01	33.16	35.31	37.46	39.61	43.91	48.21	52.51	56.81	61.11	74.01
Y26	57.23	48.15	51.67	55.20	58.72	62.24	65.77	72.81	79.86	86.90	93.95	100.99	122.13
Y112	21.32	12.83	13.83	14.82	15.82	16.81	17.80	19.79	21.78	23.77	25.76	27.75	33.71
Y113	44.21	40.70	43.69	46.68	49.68	52.67	55.67	61.65	67.64	73.63	79.62	85.61	103.57
Y29	22.37	14.48	15.60	16.71	17.83	18.94	20.06	22.29	24.52	26.75	28.98	31.20	37.89
YA2	59.52	56.23	60.33	64.42	68.52	72.61	76.71	84.90	93.09	101.28	109.47	117.66	142.23
Y33	38.06	33.87	36.38	38.88	41.39	43.90	46.41	51.42	56.44	61.46	66.47	71.49	86.54
Y34	38.06	33.87	36.38	38.88	41.39	43.90	46.41	51.42	56.44	61.46	66.47	71.49	86.54
YA3	34.25	27.67	29.73	31.80	33.86	35.93	37.99	42.12	46.25	50.38	54.51	58.64	71.04
YA4	34.25	27.67	29.73	31.80	33.86	35.93	37.99	42.12	46.25	50.38	54.51	58.64	71.04
YA5	35.76	30.07	32.31	34.55	36.78	39.02	41.26	45.73	50.21	54.68	59.16	63.63	77.05
Y50	33.32	26.50	28.49	30.47	32.45	34.43	36.41	40.38	44.34	48.30	52.27	56.23	68.12
Y51	25.39	17.15	18.46	19.77	21.08	22.38	23.69	26.31	28.92	31.54	34.16	36.77	44.62
Y52	29.70	22.10	23.76	25.43	27.09	28.76	30.42	33.75	37.08	40.41	43.74	47.07	57.06
Y58	26.24	18.09	19.47	20.84	22.22	23.59	24.97	27.72	30.47	33.23	35.98	38.73	46.99
Y61	44.21	40.70	43.69	46.68	49.68	52.67	55.67	61.65	67.64	73.63	79.62	85.61	103.57
Y62	34.25	27.67	29.73	31.80	33.86	35.93	37.99	42.12	46.25	50.38	54.51	58.64	71.04
Y63	30.05	22.10	23.76	25.43	27.09	28.76	30.42	33.75	37.08	40.41	43.74	47.07	57.06
Y64	70.54	64.94	69.65	74.36	79.07	83.78	88.49	97.91	107.33	116.75	126.17	135.59	163.85
Y65	66.57	64.94	69.65	74.36	79.07	83.78	88.49	97.91	107.33	116.75	126.17	135.59	163.85
Y66	13.18	6.08	6.57	7.07	7.56	8.06	8.55	9.55	10.54	11.53	12.52	13.51	16.48
Y67	21.32	12.83	13.83	14.82	15.82	16.81	17.80	19.79	21.78	23.77	25.76	27.75	33.71
Y68	29.70	22.10	23.76	25.43	27.09	28.76	30.42	33.75	37.08	40.41	43.74	47.07	57.06
Y70	37.05	33.87	36.38	38.88	41.39	43.90	46.41	51.42	56.44	61.46	66.47	71.49	86.54
Y73	108.04	94.83	101.65	108.46	115.27	122.08	128.90	142.52	156.15	169.77	183.40	197.02	237.90
Y86	49.65	43.60	46.80	50.00	53.20	56.40	59.60	66.01	72.41	78.81	85.21	91.61	110.81
Y87	44.21	40.70	43.69	46.68	49.68	52.67	55.67	61.65	67.64	73.63	79.62	85.61	103.57

Table 7.4b: Analysis of sliding failure

ID	F _{resist} (PGA->)	Analysis of sliding and resisting force (in kN,(PGA in cm/s ²))											
		350	400	450	500	550	600	700	800	900	1000	1100	1400
C11	22.93	9.14	9.86	10.59	11.31	12.03	12.76	14.20	15.65	17.10	18.54	19.99	24.33
C12	40.38	24.25	26.07	27.89	29.71	31.53	33.35	36.99	40.63	44.27	47.91	51.55	62.47
C13	85.78	64.94	69.65	74.36	79.07	83.78	88.49	97.91	107.33	116.75	126.17	135.59	163.85
C15	31.00	15.35	16.53	17.70	18.88	20.06	21.24	23.59	25.95	28.30	30.66	33.01	40.07
C16	33.09	17.15	18.46	19.77	21.08	22.38	23.69	26.31	28.92	31.54	34.16	36.77	44.62
C17	33.09	17.15	18.46	19.77	21.08	22.38	23.69	26.31	28.92	31.54	34.16	36.77	44.62
C18	98.75	84.24	90.31	96.38	102.45	108.52	114.59	126.72	138.86	151.00	163.14	175.27	211.69
C21	51.70	33.87	36.38	38.88	41.39	43.90	46.41	51.42	56.44	61.46	66.47	71.49	86.54
C23	43.24	27.67	29.73	31.80	33.86	35.93	37.99	42.12	46.25	50.38	54.51	58.64	71.04
C26	105.82	74.28	79.65	85.01	90.38	95.75	101.12	111.85	122.59	133.32	144.06	154.80	187.00
C27	130.32	90.52	97.03	103.54	110.05	116.56	123.07	136.09	149.11	162.13	175.15	188.17	227.23
C30	97.06	74.28	79.65	85.01	90.38	95.75	101.12	111.85	122.59	133.32	144.06	154.80	187.00
C36	76.33	56.23	60.33	64.42	68.52	72.61	76.71	84.90	93.09	101.28	109.47	117.66	142.23
C37	68.44	48.15	51.67	55.20	58.72	62.24	65.77	72.81	79.86	86.90	93.95	100.99	122.13
C40	50.08	33.87	36.38	38.88	41.39	43.90	46.41	51.42	56.44	61.46	66.47	71.49	86.54

The distribution of failure is obtained as follows in the study area:

Table 7.5: Sliding Failure distribution in study area

Intensity (PGA- cm/s ²)	350	400	450	500	550	600	700	800	900	1000	1100	1400
No of failure case	0	4	12	5	5	7	6	9	1	2	2	1
Total failure		4	16	21	26	33	39	48	49	51	53	54

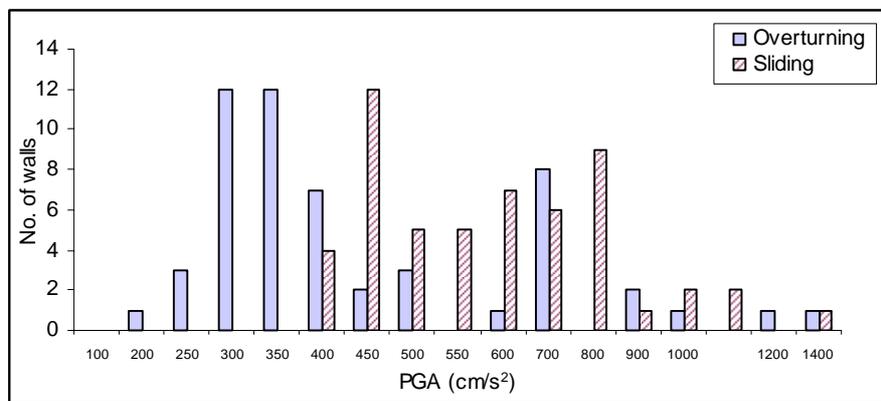


Figure 7.1: Distribution of overturning and sliding cases in the study area

Sliding force can cause failure if the intensity of earthquake ground intensity will be higher than or nearly that of 400 gal (Fig 7.1). If we compare it with the overturning mode of failure, the sliding is not dominant mode. Prior to occur the sliding, overturning will cause the failure of retaining walls. The all failure seems to have occurred at the same intensity of earthquake at 1400 gal. The failure at a particular

point provides deterministic value in terms of PGA. Overturning case is described on deterministic resisting force against continuous dragging force. In case of sliding, both of resisting force and dragging force are taken in continuous probabilities.

7.5. Analysis of Compressive and Tensile Stress with Wall Strength

This failure mode could be described as the material failure rather than whole structural failure though it will cause damage to retaining walls.

The bending moment of the wall was calculated to find out the compressive and tensile stress applied to the wall. To find out the bending moment the following common relation was used

$$\frac{M}{I} = \frac{F}{y}$$

where, M is the bending moment generated at wall base and

I = moment of inertia

F = Shear force applied to the bottom of wall height

y = effective thickness of wall for tensile or compressive force

Table 7.6a: Compressive stress and shear stress compared with strength

ID	Strength measured (kPa)	At PGA 500 cm ² /s		At PGA 600 cm ² /s		At PGA cm ² /s 600
		Bend.Mo (kN.m)	Stress(kPa)	Bend.Mo (kN.m)	Stress(kPa)	Shear stress (kPa)
Y11	21182.4	41.9	6354.8	46.7	7077.5	169.9
Y14	24124.4	23.0	3480.6	25.6	3876.4	170.6
Y15	18240.4	16.1	2444.5	18.0	2722.5	134.8
Y16	21770.8	6.8	1031.3	7.6	1148.6	75.8
Y17	21770.8	3.9	596.8	4.4	664.7	52.6
Y18	14121.6	6.8	1031.3	7.6	1148.6	75.8
Y19	22065.0	32.3	4889.1	35.9	5445.1	134.8
Y20	18828.8	41.9	6354.8	46.7	7077.5	254.8
YA1	18534.6	10.8	1637.6	12.0	1823.9	103.2
Y22	22065.0	86.5	13101.2	96.3	14591.1	412.7
Y23	25301.2	94.1	14256.5	104.8	15877.8	436.7
Y24	26478.0	41.9	6354.8	46.7	7077.5	254.8
Y25	23947.8	24.5	3717.8	27.3	4140.6	178.2
Y26	22947.6	54.5	8250.3	60.6	9188.6	303.2
Y112	24124.4	6.8	1031.3	7.6	1148.6	75.8
Y113	24595.1	41.9	6354.8	46.7	7077.5	254.8
Y29	22829.9	8.3	1251.6	9.2	1393.9	86.3
YA2	22065.0	69.2	10489.5	77.1	11682.4	355.9
Y33	25595.4	31.5	4774.5	35.1	5317.4	210.6
Y34	20711.6	31.5	4774.5	35.1	5317.4	210.6
YA3	23771.3	23.0	3480.6	25.6	3876.4	170.6
YA4	22829.9	23.0	3480.6	25.6	3876.4	170.6
YA5	20696.9	26.2	3965.6	29.2	4416.6	186.1

Y50	12827.1	21.5	3253.7	23.9	3623.7	163.1
Y51	20358.6	10.8	1637.6	12.0	1823.9	103.2
Y52	21888.4	16.1	2444.5	18.0	2722.5	134.8
Y58	26831.0	11.8	1782.1	13.1	1984.7	109.2
Y61	11768.0	41.9	6354.8	46.7	7077.5	254.8
Y62	13886.2	23.0	3480.6	25.6	3876.4	170.6
Y63	22947.6	16.1	2444.5	18.0	2722.5	134.8
Y64	25771.9	86.5	13101.2	96.3	14591.1	412.7
Y65	29125.8	86.5	13101.2	96.3	14591.1	412.7
Y66	18828.8	2.0	305.6	2.2	340.3	33.7
Y67	24006.7	6.8	1031.3	7.6	1148.6	75.8
Y68	18122.7	16.1	2444.5	18.0	2722.5	134.8
Y70	13768.5	31.5	4774.5	35.1	5317.4	210.6
Y73	20711.6	154.8	11848.5	172.4	13195.9	405.7
Y86	25536.5	46.7	7073.6	52.0	7878.0	273.7
Y87	24830.4	41.9	6354.8	46.7	7077.5	254.8

The wall may have some variation on installing the reinforcement bar, so there may be some differences for each wall to find the effective thickness of acting force. Even though this, it is assumed that 0.1 to 0.2 cm is commonly applicable thickness of the wall to install reinforcement.

The results obtained from this computation are given in Table 7.6.

Table 7.6b: Compressive stress and shear stress compared with strength

ID	Strength measured (kPa)	At 500 g		At 600 g		At 600 g Shear stress (kPa)
		Bend.Mo (kN.m)	Stress(kPa)	Bend.Mo (kN.m)	Stress(kPa)	
C11	29420.0	3.9	596.8	4.4	664.7	52.6
C12	27301.7	18.7	2829.9	20.8	3151.7	148.6
C13	20696.9	86.5	13101.2	96.3	14591.1	412.7
C15	24589.2	9.1	1372.6	10.1	1528.7	91.7
C16	24342.1	10.8	1637.6	12.0	1823.9	103.2
C17	21747.2	10.8	1637.6	12.0	1823.9	103.2
C18	14945.3	129.1	19556.2	143.8	21780.3	359.4
C21	17181.3	31.5	4774.5	35.1	5317.4	210.6
C23	20299.8	23.0	3480.6	25.6	3876.4	170.6
C26	18711.1	106.4	16113.9	118.4	17946.4	473.8
C27	26772.2	144.1	21839.9	160.5	24323.6	464.2
C30	24948.1	106.4	16113.9	118.4	17946.4	473.8
C36	21064.7	69.2	10489.5	77.1	11682.4	355.9
C37	17534.3	54.5	8250.3	60.6	9188.6	303.2
C40	17416.6	31.5	4774.5	35.1	5317.4	189.07

The compressive strength of concrete is much higher than that of compressive stress. The material concrete itself has a higher value of strength. From the result, only one will be damaged with 600 gal. The results obtained (Table 7.6) suggests that possibility of failure of wall due to material damage is very low.

The acting force is taken to be compressive stress on the face side of the wall while tensile stress on the back side of the wall. In this case, the tensile stress will be equivalent to the compressive strength of the wall. Mainly the reinforcement will resist the tensile strength of the wall. In this case, there is very rare chance to overcome the compressive strength to cause tensile failure. In having the details reinforcement bars of each retaining wall it can be determined effectively though it is sufficient to sustain the wall in general case.

The shear force acting on the wall is also analyzed but it will not cause the failure even in the intensity of 600 g. This force also can be checked with the standard shear resistance of concrete that is commonly used.

7.6. Dominant Failure Mode and Probability Analysis

The dominant failure mode was found to be the overturning mode among overturning, sliding, compressive or tensile analysis. The overturning mode is further analyzed for the probability of failure with view point of occurrence of earthquake for each site. This will give us the overall probability of the concrete retaining walls due to the effect of dynamic load during earthquakes.

The probability of failure is obtained using the seismic hazard of specific sites. The log normal distribution may be useful for the cases that use strength of material (Ang and Tang 1975). The safety index or reliability index can be obtained from the probability density function using equation 7.1 for the continuous data set.

$$\beta = \frac{\lambda_R - \lambda_D}{\sqrt{\zeta_R^2 + \zeta_D^2}} \quad (7.1)$$

where, λ_R and λ_D are mean log value of resistance and load respectively while ζ_R and ζ_D are log normal standard deviations.

In case of deterministic resistance value, we can find β by,

$$\beta = \left(\frac{\ln x - \lambda}{\zeta_D} \right) \quad (7.2)$$

where, x is the mean value of log of dragging load and λ is mean log value of load

The failure probability will be

$$P_f = 1 - \Phi(\beta) \quad (7.3)$$

Table 7.7: Failure Probability of each retaining wall

ID	Location		Locality ID	Overturn, P_i	Slide, P_i
C11	Ikegami 1 Chome	17-19	Olk1	0.0322	0.0281
C12	Ikegami 1 Chome	17-17	Olk1	0.3264	0.119
C13	Ikegami 1 Chome	17-15	Olk1	0.648	0.2119
C15	Ikegami 1 Chome		Olk1	0.1401	0.056
C16	Ikegami 1 Chome	20-26	Olk1	0.1401	0.0708
C17	Ikegami 1 Chome	20-26	Olk1	0.1401	0.0708
C30	Chuo 5 Chome	29-1	Olk1	0.5477	0.2119
C36	Chuo 6 Chome	6-5-1	Olk1	0.5477	0.2119
C18	Sanno 4 Chome	32-11	Osan	0.7967	0.3086
C21	Sanno 4 Chome	32-12	Osan	0.587	0.1272
C23	Sanno 4 Chome	32-13	Osan	0.4961	0.1272
C26	Sanno 3Chome	44-9	Osan	0.4961	0.1686
C27	Sanno 3 Chome	44-4	Osan	0.4208	0.1686
Y11	Mineoka 3 Chome		HM3	0.6103	0.3975
Y14	Mineoka 3 Chome	306	HM3	0.5239	0.3446
Y15	Mineoka 3 Chome	399	HM3	0.3336	0.2644
Y16	Mineoka 3 Chome	398	HM3	0.1848	0.1563
Y17	Mineoka 3 Chome	398-8	HM3	0.0808	0.1231
Y18	Mineoka 3 Chome	401	HM3	0.1848	0.1563
Y19	Mineoka 3 Chome	401-7	HM3	0.5239	0.3016
Y20	Mineoka 3 Chome	401-6	HM3	0.7019	0.4563
YA1	Mineoka 3 Chome	401	HM3	0.1848	0.1563
Y22	Mineoka 3 Chome		HM3	0.6103	0.3975
Y23	Mineoka 3 Chome		HM3	0.5239	0.4563
Y24	Mineoka 2 Chome	219-7	HM3	0.6103	0.3975
Y25	Mineoka 2 Chome		HM3	0.4523	0.3446
Y26	Mineoka 3 Chome		HM3	0.5239	0.3446
Y29	Mineoka 2 Chome	188-4	HM3	0.1848	0.1563
YA2	Mineoka 3 Chome	438-6	HM3	0.5239	0.4563
Y33	Mineoka 3 Chome	414	HM3	0.6103	0.3975
Y34	Mineoka 3 Chome	413-2	HM3	0.6103	0.3975
YA3	Mineoka 3 Chome		HM3	0.4523	0.3016
YA4	Mineoka 3 Chome		HM3	0.4523	0.3016
YA5	Mineoka 3 Chome	408	HM3	0.5239	0.3446
Y50	Kamadaicho	8-4	Hka	0.4602	0.3632
Y51	Kamadaicho	8-7	Hka	0.2327	0.2515
Y52	Kamadaicho	8-10	Hka	0.4052	0.3192
Y58	Kamadaicho	39-22S	Hka	0.3086	0.2515
Y61	Mineoka 1 Chome	1-89	HM1	0.7257	0.5159
Y62	Mineoka 1 Chome	1-94	HM1	0.5832	0.4169
Y63	Mineoka 1 Chome	94	HM1	0.5832	0.3745
Y64	Mineoka 1 Chome		HM1	0.7257	0.5159
Y65	Mineoka 1 Chome		HM1	0.8023	0.5763
Y66	Mineoka 1 Chome	9-1	HM1	0.0643	0.1357
Y67	Mineoka 1 Chome	98-15	HM1	0.1841	0.2451
Y68	Mineoka 1 Chome	98-1	HM1	0.4642	0.3745
Y70	Mineoka 1 Chome	100	HM1	0.6517	0.5159
Y73	Miyata-cho 3 Chome	307	HM1	0.6517	0.5159
Y112	Mineoka 2 Chome	142-10	HM1	0.1841	0.2451
Y113	Mineoka 2 Chome		HM1	0.7257	0.5159
Y86	Nishinoya-cho	114-33	NK	0.5674	0.5
Y87	Mameguchi-dai	98	NK	0.719	0.5
C37	Chuo 5 Chome	8-13	OMg1	0.5792	0.1841
C40	Minami-magome	42-18	OMg1	0.5792	0.1841

The parameters ζ and λ for the PGA distribution were obtained from seismic hazard data. The probability of the failure of retaining walls at specified PGA was obtained from these parameters. The obtained probability of failure can be seen in table 7.7 and fig 7.2 to 7.31. The probabilities of failure for each retaining wall were obtained (Fig. 7.3 to 7.31, bold line is drawn for overturning and other lines for sliding case).

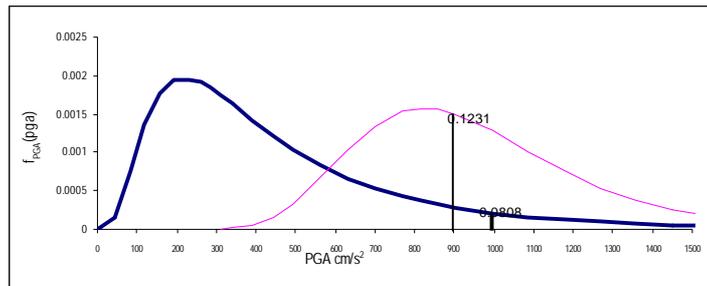


Figure 7.2: Probability of failure for retaining wall-Y17

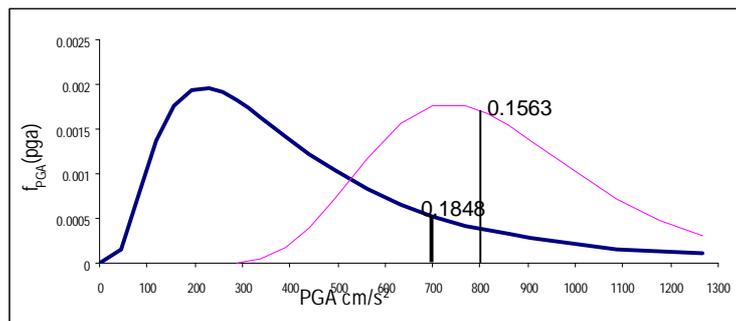


Figure 7.3: Probability of failure for retaining wall-Y16, Y18, YA1, Y29

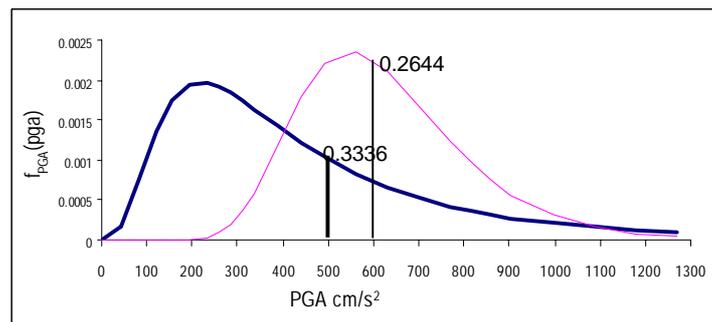


Figure 7.4: Probability of failure for retaining wall- Y15

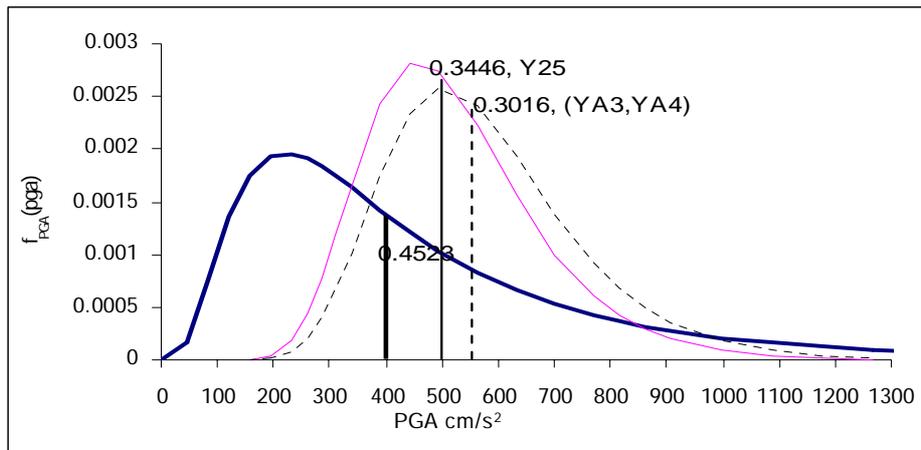


Figure 7.5: Probability of failure for retaining wall- Y25, YA3, YA4

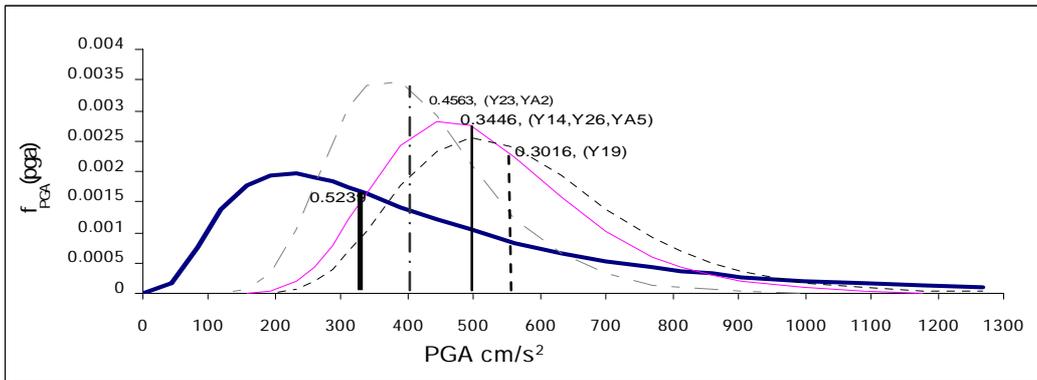


Figure 7.6: Probability of failure for retaining wall- Y14, Y19, Y23, Y26, YA2, YA5

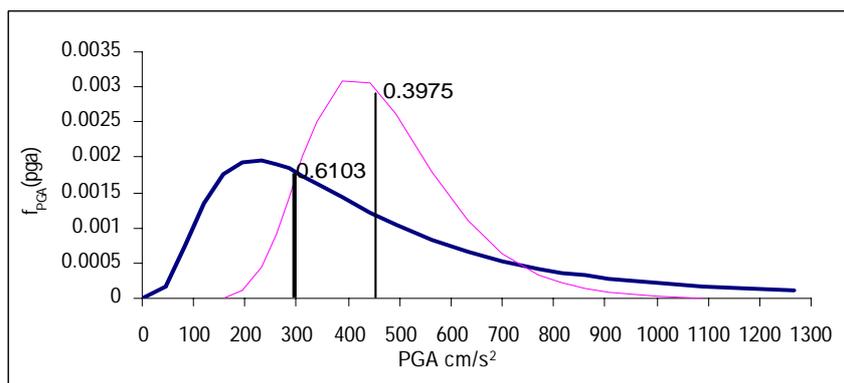


Figure 7.7: Probability of failure for retaining wall- Y11, Y22, Y24, Y33, Y34

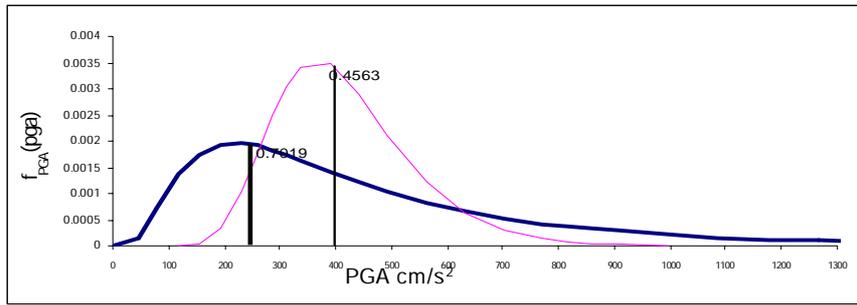


Figure 7.8: Probability of failure for retaining wall- Y20

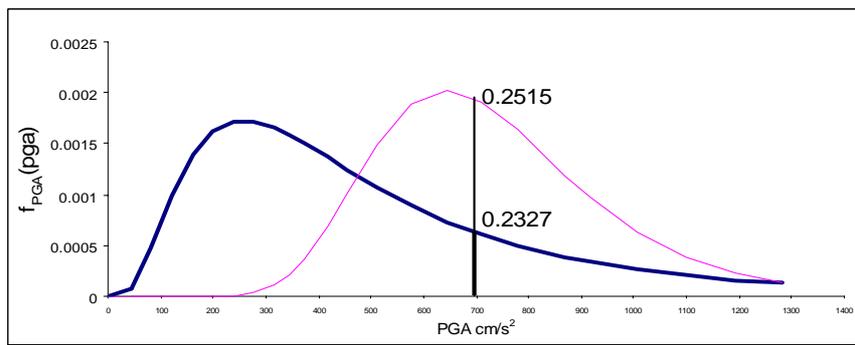


Figure 7.9: Probability of failure for retaining wall- Y51

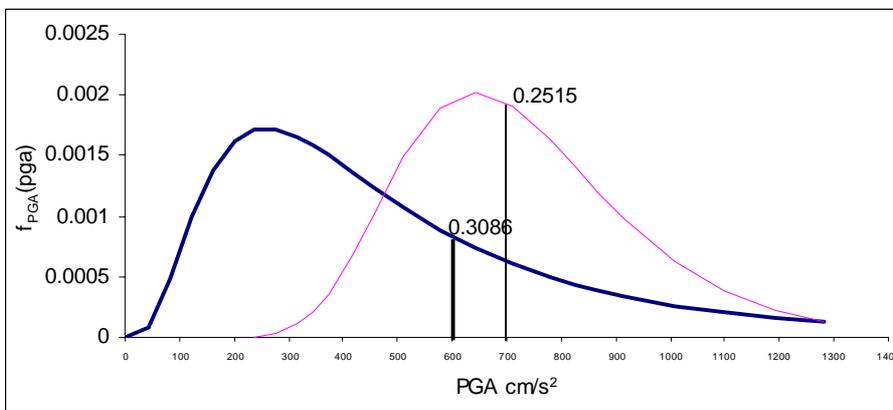


Figure 7.10: Probability of failure for retaining wall- Y58

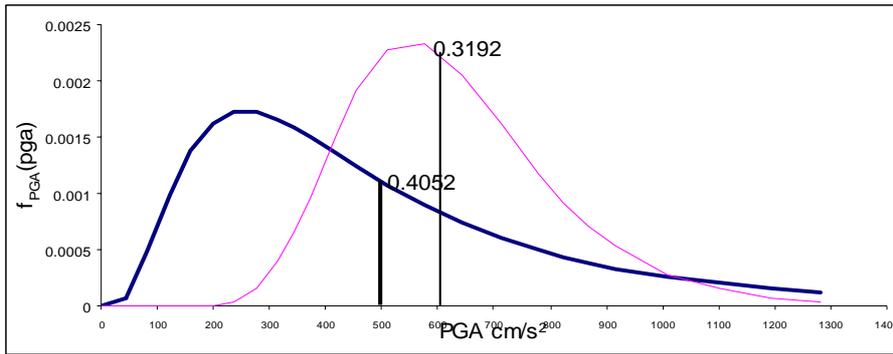


Figure 7.11: Probability of failure for retaining wall- Y52

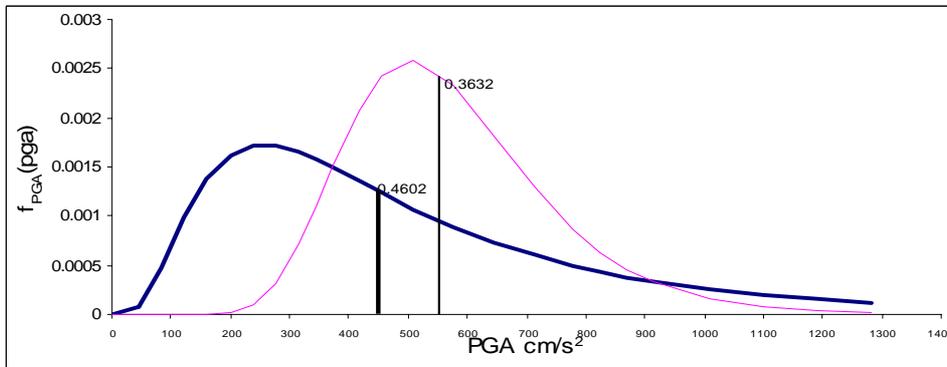


Figure 7.12: Probability of failure for retaining wall- Y50

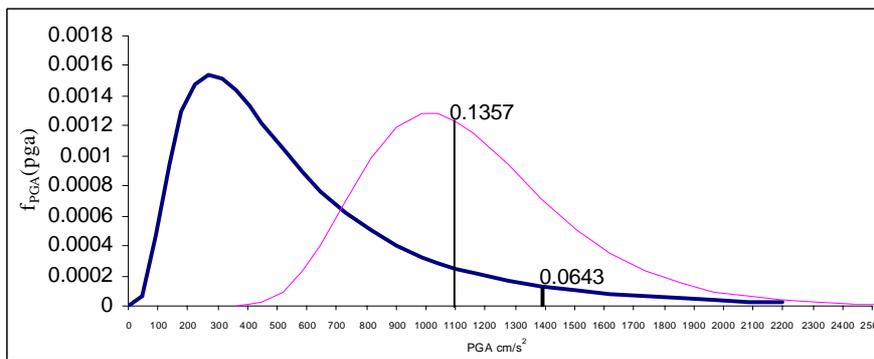


Figure 7.13: Probability of failure for retaining wall- Y66

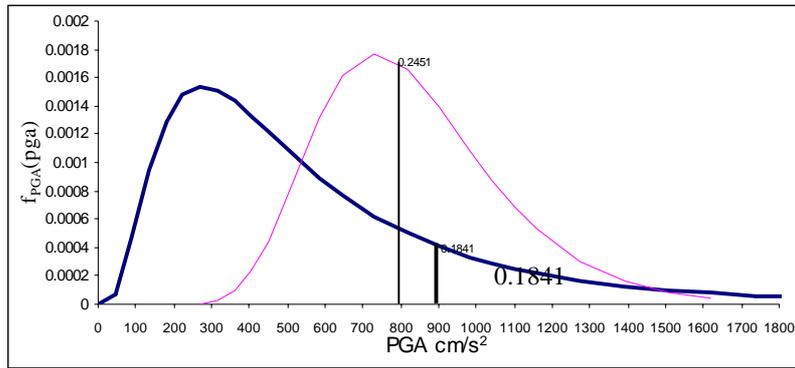


Figure 7.14: Probability of failure for retaining wall- Y67, Y112

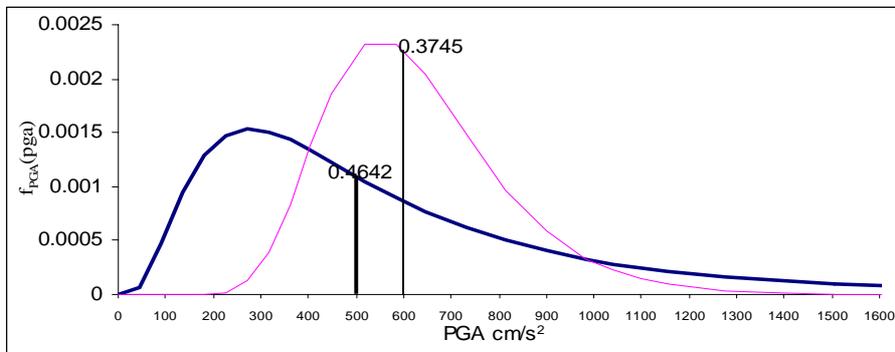


Figure 7.15: Probability of failure for retaining wall- Y68

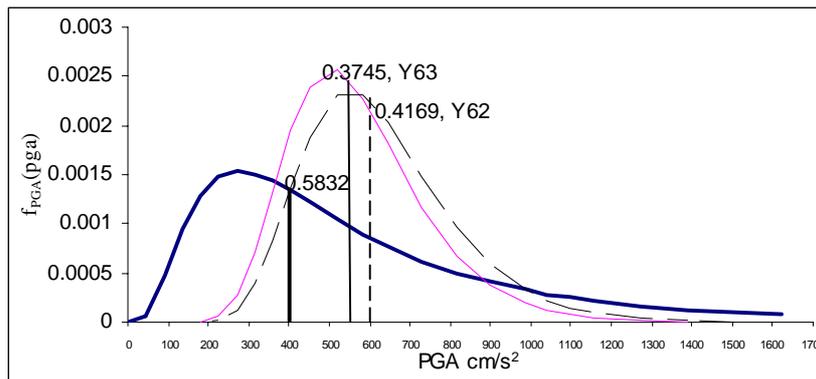


Figure 7.16: Probability of failure for retaining wall- Y62, Y63

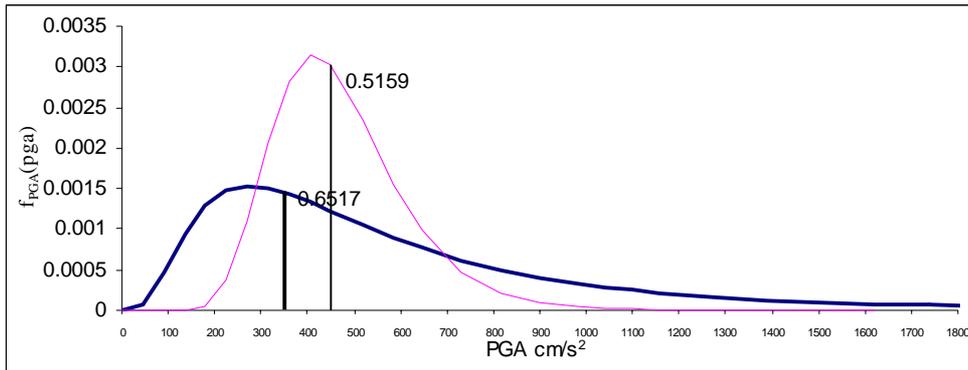


Figure 7.17: Probability of failure for retaining wall- Y70, Y73

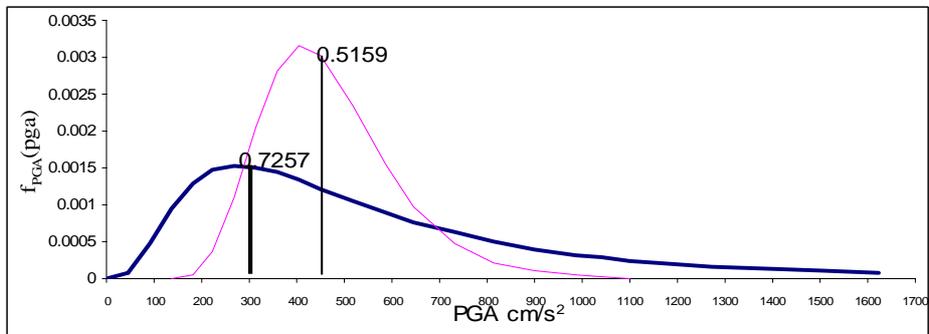


Figure 7.18: Probability of failure for retaining wall- Y61, Y64, Y113

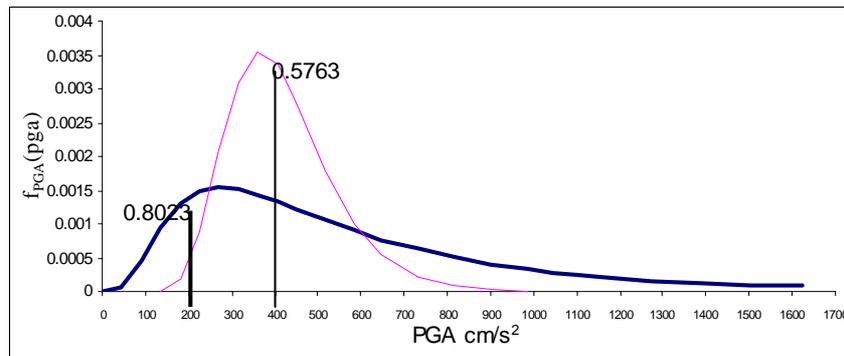


Figure 7.19: Probability of failure for retaining wall- Y65

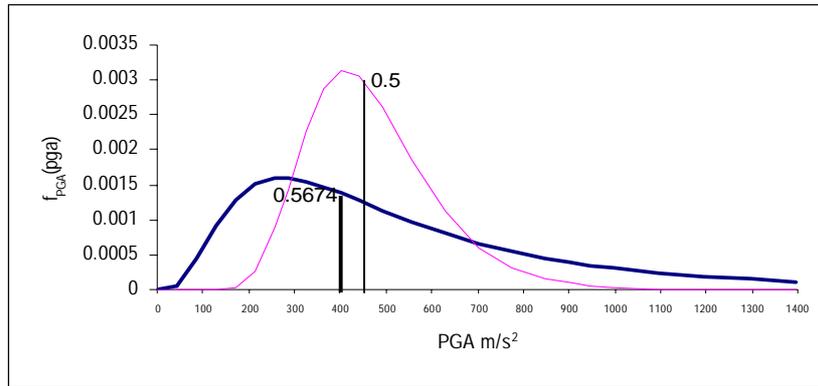


Figure 7.20: Probability of failure for retaining wall- Y86

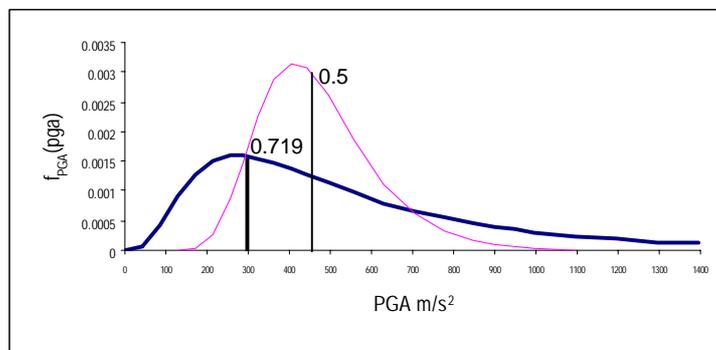


Figure 7.21: Probability of failure for retaining wall- Y87

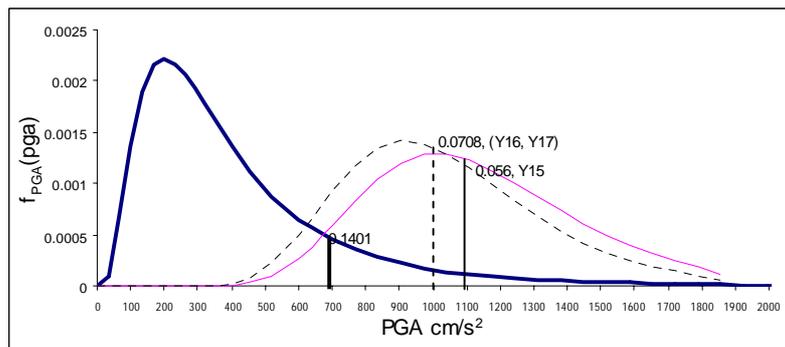


Figure 7.22: Probability of failure for retaining wall- C15, C16, C17

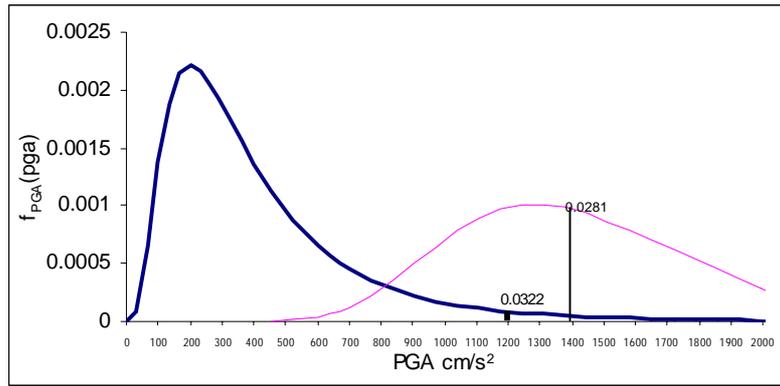


Figure 7.23: Probability of failure for retaining wall- C11

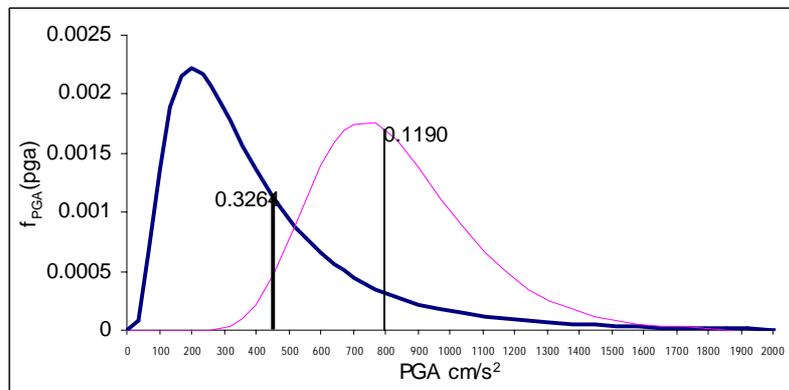


Figure 7.24: Probability of failure for retaining wall- C12

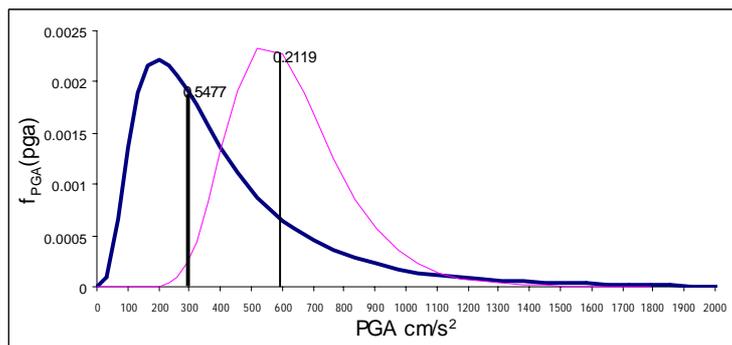


Figure 7.25: Probability of failure for retaining wall- C30, C36

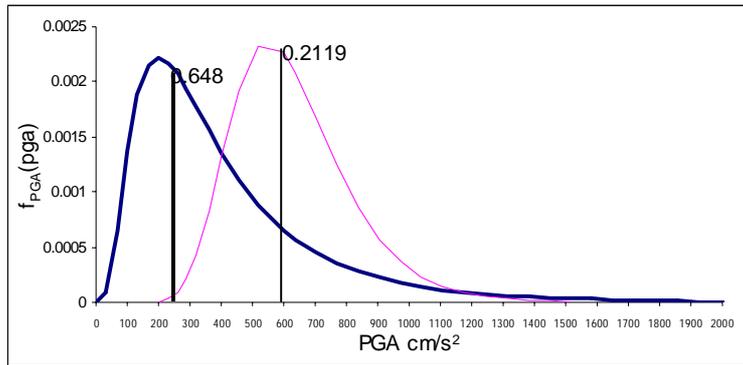


Figure 7.26: Probability of failure for retaining wall- C13

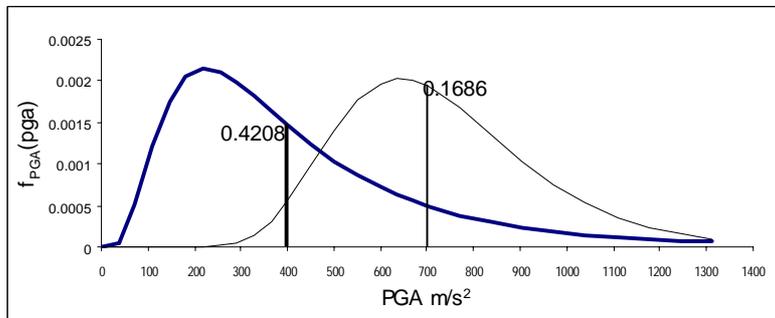


Figure 7.27: Probability of failure for retaining wall- C27

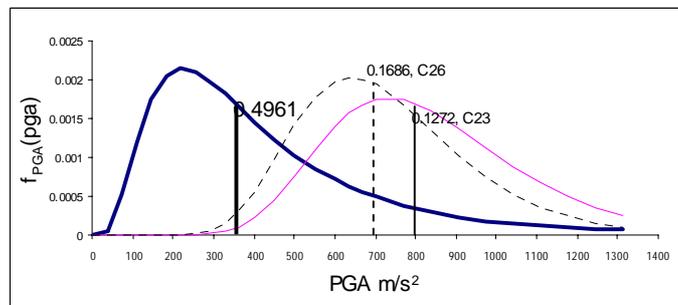


Figure 7.28: Probability of failure for retaining wall- C23, C26

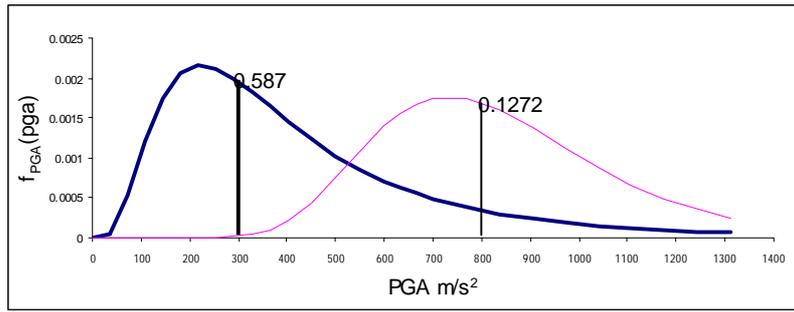


Figure 7.29: Probability of failure for retaining wall- C21

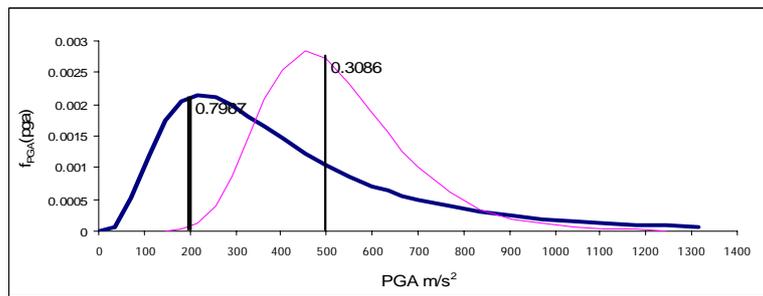


Figure 7.30: Probability of failure for retaining wall- C18

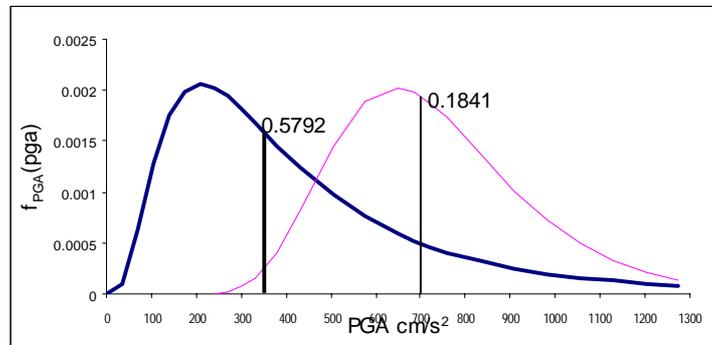


Figure 7.31: Probability of failure for retaining wall- C37, C40

The loads and resistance phenomena are inherently random in nature (Aoki et. al. 2000); therefore probability assessment has become significant to measure safety of retaining walls in terms of probability of failure. In case of building safety, reliability for 50 years for failure of structural elements are considered to be the reliability index 2.0 according to Aoki et. al. (2000). The seismic hazard curve in terms of PGA was used to find out the probability of failure. According to the results obtained from the analysis, we can consider the walls having high probability of failure (more than 70%), medium with range of 30-70% and lower probability of failure having less than 30%. The distribution of probability failure is shown in figure 7.32. Twelve retaining walls have less than 30% probability and seven walls have more than 70% probability. Most of the walls are in the medium range having large mode in between 50-60%.

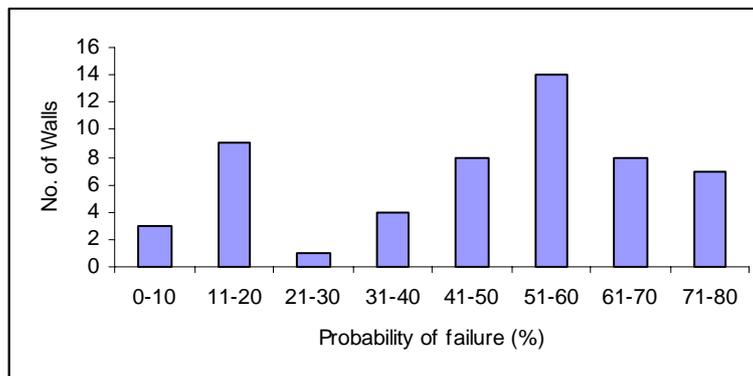


Figure 7.32a: Distribution of probability of failure by overturning

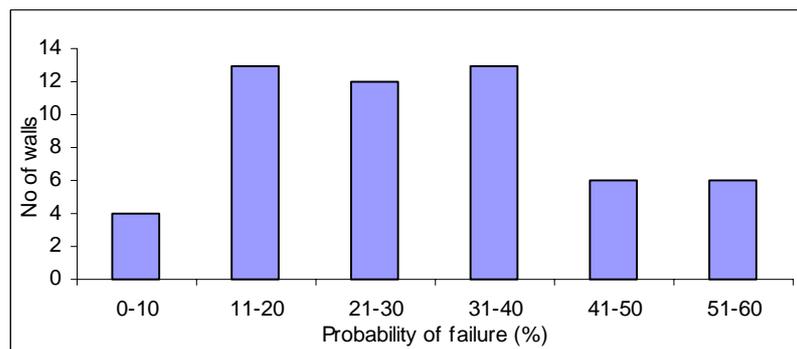


Figure 7.32a: Distribution of probability of failure by sliding

The probability of failure in different area varies with many factors. In Sanno area of Ota ward, most of walls have high probability for overturning failure ranging from 42 to 79 %. It may be due to relatively taller walls and ground condition of low land clay

deposit. In this area, sliding probability ranges only from 12-30 %. North-west part and southeast part of Hodogaya ward also shows varying of probability of failure for overturning case. Kamadaicho area in Hodogaya ward has nearly middle range of probability of failure for both overturning and sliding (for overturning 23-52% and for sliding 25-36%) as compared to walls in other places. From results, clay deposited area has relatively low sliding probability while overturning probability is dependent mainly on wall height and backfill weight on it.

This chapter has provided the details of method for estimating the failure probability of concrete retaining walls in the area. Most of the natural phenomena have some uncertainty and sometimes they show the aleatory behavior. Our model and some assumptions can increase uncertainty level. So, the probabilistic methods of reliability or failure assessment are quite significant tool to use in the failure probability of retaining walls. The procedure approached here for the assessment of failure probability of retaining walls is important in this case. In having the précised detail information about retaining walls and appropriate model, we can predict the failure of probability more accurately.

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Chapter 8

Conclusions and Recommendations

8.1. Conclusions

The study has focused on safety of retaining walls which will contribute to the safety of our social and physical environment. The study area, Ota ward in Tokyo Metropolitan and Hodogaya and Naka ward in Yokohama Municipality, includes fragile ground condition and some sloppy areas.

Retaining wall damages are dependent on ground condition, wall types and age. The results from large earthquakes show that the ground condition is the most important factor to all types of retaining walls, which is also associated with slope failures. The retaining wall damages in housing lots or individual houses have shown that their damage pattern and types are sometimes different from highway or railway retaining structures. Preparation of contour map for ground motion intensity of earthquake will be an appropriate method to estimate the hazard and it is also useful to predict damages.

In recent years, construction of concrete retaining walls is increasing in the study area. More than 60% of such walls were found to be built within last 10 years. The relationship between strength measured by Schmidt hammer and age of retaining wall shows the strength decreasing pattern.

Seismic hazard in the study area was found in terms of peak ground acceleration. South-east part of Mineoka cho shows higher seismic hazard as median probability for 50 year exceeding to be about 600 gal while other parts have nearly 400-450 gal range. The resistance of the retaining walls against earthquake loading was computed for different failure modes- overturning, sliding, shearing, and compressive failure. The most dominant failure mode was found to be the overturning mode. Overturning mode could cause high failure with the intensity of 300-400 gal as found in the study area. Sliding failure in the study area may happen by an earthquake with intensity of 400 gal or more, however this is not dominated mode of failure. Compressive or tensile and shearing failure, a type of material failure, seems to have very low possibility to cause failure.

From the dominant failure mode, failure probability of retaining walls was obtained. Most of the retaining walls have failure probability from 40-60 % for overturning and from 10-40% for sliding in 50 years period. It may have some uncertainty on resisting force which can be reduced by using very detailed information on wall properties, model and seismic hazard. It can be assumed that conservative estimates came from Mononobe-Okabe method.

Proposed procedure in this study to find the probability can be widely applied to walls in other places. This procedure of probabilistic approach is useful to estimate the reliability and safety of retaining walls and consequently of our environment.

8.2. Recommendations for Further Research

Although the procedure of failure probability of retaining walls has been provided, there are many aspects and complex parameters that increase the uncertainty in the probabilistic hazard assessment of structures. More specific information on the retaining wall behavior during the earthquake would play a key role to find the stability and reliability more accurately.

In static condition, there will be some background triggering factors produced by simple natural phenomena or due to human activities. Therefore, earth pressure may not be absolutely static in nature. There may small scale dynamic behavior. However, Mononobe Okabe method gives the dynamic thrust due to earthquake loading; the point of act of the dynamic thrust will be fluctuating due to uncertain earthquake behavior. If we know very small scale background, then we can estimate earth pressure more accurately. Furthermore, we can analyze the existing retaining structures response to this behavior. It is recommended to use Microtremor for this analysis to find out such dynamic behavior and to analyze safety. It will also provide subsurface behavior in different scale of ground motion intensity.

Most of the models for computing earth pressure are based on the safety calculation during the design of retaining wall and safety of wall is usually described in terms of factor of safety. But safety or reliability evaluation of existing retaining walls will need different approach with more précised information about ground behavior. Therefore, it is recommended that the evaluation of existing retaining structures should be analyzed from view point of their reliability by evaluating their current response behavior.

The failure of retaining walls is associated with the slope failures. In this case, the slope stability analysis should be assembled in the safety evaluation of retaining walls.